

**PRELIMINARY
GEOTECHNICAL INVESTIGATION**

**SUMMIT ESTATES
SAN DIEGO COUNTY, CALIFORNIA**



GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

**2510 SUMMIT, LLC
IRVINE, CALIFORNIA**

**JANUARY 11, 2019
PROJECT NO. G2279-42-01**



Project No. G2279-42-01

January 11, 2019

2510 Summit, LLC
19782 MacArthur Boulevard, Suite 300
Irvine, California 92612

Attention: Mr. Oscar Uranga

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
SUMMIT ESTATES
SAN DIEGO COUNTY, CALIFORNIA

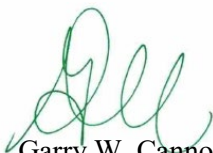
Dear Mr. Uranga:

In accordance with your authorization and our proposal (LG-18-090 dated October 11, 2018) we herein submit our preliminary geotechnical investigation for the subject project. The accompanying report presents the findings, conclusions, and recommendations pertinent to the project. Based on the results of our study, it is our opinion that the subject site can be developed as proposed, provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON INCORPORATED


Garry W. Cannon
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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical investigation for the property located northeast of the intersection of Mary Lane and Summit Drive, San Diego County, California (see *Vicinity Map*, Figure 1). The purpose of this investigation was to evaluate site geology; observe and sample the prevailing soil conditions at the site; and to provide recommendations pertinent to the geotechnical aspects of constructing the proposed improvements.

The scope of our investigation included a review of relevant published reports, a site reconnaissance, a field investigation, laboratory testing, engineering analyses, and preparation of this report.

The field investigation was performed on December 7 and 13, 2018. The investigation consisted of drilling 10, air-percussion, borings and excavating fourteen, shallow, exploratory pits at the approximate locations shown on the *Site Plan*, Figure 2. Logs of the exploratory trenches, borings and other details of the field investigation are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained from the borings to evaluate their pertinent physical and chemical properties for engineering analyses. A discussion pertaining to the laboratory testing and results is presented in Appendix B.

Six infiltration tests were performed in general conformance with guidelines presented Geosyntec (2018) at the approximate locations shown on Figure 2. The results and conclusions of the infiltration testing are presented in Appendix C.

The recommendations presented herein are based on our analysis of the data obtained from the exploratory boring, laboratory tests and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The site consists of approximately 22 acres of land currently occupied by one, single-family residence. The existing residence sits on a hill in the western portion of the property. Site elevations range from near 858 feet Mean Sea Level (MSL) at the top of the hill to near 780 feet MSL at the northwest corner, 790 feet MSL at the south side, and near 700 feet MSL at the southeast end of the property.

Based on preliminary concept plans, the site will be developed into 20, single-family, approximately 1-acre, lots. Cuts and fills up to approximately 15 feet are planned to construct individual building

pads and create roadways. Fill slopes up to 35 feet and cut slopes up to 25 feet are planned on the property. Cul-de-sac streets extending from Summit Drive into the property provide access to the residential lots. Storm water BMP basins are planned at three locations on the property. A pressurized drip disposal system is planned for the septic system.

3. SOIL AND GEOLOGIC CONDITIONS

Geology at the site consists of Cretaceous age granitic rock covered by up to topsoil. The upper portion of the granitic rock is moderately to highly weathered. Geologic cross sections are provided on Figure 3.

3.1 Topsoil (unmapped)

Topsoil was observed to depths of approximately 2 feet. The topsoil generally consisted of loose, silty, fine to medium sand. The topsoil, in its natural state, is not suitable for the support of settlement-sensitive structures or structural fill and should be removed and replaced as compacted fill in lots, slopes, and street improvement areas.

3.2 Weathered Granitic Rock (Kgr)

Deeper weathered Cretaceous age granitic rock was observed in the area near Trench T-2 and could be present at other areas. The weathered granitic rock excavated as silty sand. The highly weathered granitic rock is considered compressible and should be removed and replaced as compacted fill in areas of settlement-sensitive structures and structural fill.

3.3 Granitic Rock (Kgr)

Cretaceous age granitic rock was observed in all trenches. The upper portion of the granitic rock is moderately weathered and excavatable with conventional heavy-duty equipment. Weathering decreases with depth and the formation becomes non-rippable at depths around 10 feet below ground surface. The undisturbed granitic rock is suitable for the support of settlement-sensitive structures and structural fill.

4. GROUNDWATER

No groundwater was encountered during our investigation. Groundwater is not expected to significantly affect project development as presently proposed; however, it is not uncommon for groundwater or seepage conditions to develop where none previously existed. Proper surface drainage of irrigation and rainwater will be critical to future performance of the project.

5. GEOLOGIC HAZARDS

5.1 Ground Rupture

No evidence of faulting was observed during our investigation. The 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 nearest active fault is the Elsinore Fault, which lies approximately 17 miles west of the site. The risk associated with ground rupture hazard is low.

5.2 Seismicity

We performed a deterministic seismic hazard analysis using Risk Engineering (2015). Seven known active faults were located within a search radius of 50 miles from the property. We used the 2008 USGS fault database, which provides several models and combinations of fault data, to evaluate the fault information. Based on this database, the Elsinore Fault, located approximately 16.7 miles northeast of the site, is the nearest known active fault and is the dominant source of potential ground motion. Earthquakes that might occur on the Elsinore Fault 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 Elsinore Fault are 7.85 and 0.21g, respectively. The table below lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relation to the site. We calculated peak ground acceleration (PGA) using acceleration-attenuation relationships by: Boore and Atkinson (2008); Campbell and Bozorgnia (2008); and Chiou and Youngs (2008).

**TABLE 5.2.1
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)
Elsinore	16.7	7.85	0.21	0.15	0.20
Newport-Inglewood/Rose Canyon	17.2	7.5	0.18	0.13	0.16
Rose Canyon	17.2	6.9	0.14	0.11	0.11
Earthquake Valley	27.0	6.8	0.10	0.07	0.06
Coronado Bank	32.0	7.4	0.11	0.08	0.09
Palos Verdes Connected	32.0	7.7	0.13	0.09	0.11
San Jacinto	38.6	7.88	0.12	0.08	0.10

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. The risk at this site is comparable to others in the general vicinity with respect to seismic shaking hazard.

We performed a probabilistic seismic hazard analysis for the site using Risk Engineering (2015). The computer program assumes that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the fault 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 used acceleration-attenuation relationships suggested by Boore-Atkinson (2008), Campbell-Bozorgnia (2008), and Chiou-Youngs (2008) in the analysis. Table 5.2.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

**TABLE 5.2.2
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	0.39	0.38	0.44
5% in a 50 Year Period	0.30	0.28	0.31
10% in a 50 Year Period	0.23	0.22	0.23

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 soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the California Building Code (CBC) guidelines.

5.3 Liquefaction and Seismically Induced Settlement

Due to the dense subsurface soils and the lack of permanent, near-surface groundwater, the risk associated with seismically induced soil liquefaction hazard is low.

5.4 Landslides

No evidence of landsliding was encountered at the site during the geotechnical investigation or in our review of historic, stereoscopic aerial photographs (USDA, 1953).

The risk associated with ground movement hazard due to landsliding is low.

5.5 Subsidence

Based on the subsurface soil conditions encountered during our field investigation, the risk associated with ground subsidence hazard is low.

5.6 Seiches and Tsunamis

The site is not located within a tsunami inundation zone as defined by California Geological Survey. Elevation at the site is approximately 700 feet MSL and higher. There are no lakes or reservoirs located near the site. The risk associated with inundation hazard due to tsunamis or seiches is low.

5.7 Flooding

The Federal Emergency Management Agency (FEMA 2012) locates the site within a Flood Zone X area, indicating a minimal risk to inundation by 100-year and 500-year floods.

6. ROCK RIPPABILITY

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 25 spf indicates marginally to non-rippable material, and greater than 25 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 on the air-track logs 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. 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.

Very difficult ripping and/or blasting may be required for excavations that extend beyond the rippable weathered mantle. Based on an air-track penetration rate of 20 spf, the thickness of the rippable rock mantle varies between 5 to 28 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.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

- 7.1.1 From a geotechnical standpoint, it is our opinion that the site is suitable for the proposed development, provided the recommendations presented herein are implemented in design and construction of the project.
- 7.1.2 Once an approved grading plan is available and the development plans are prepared, a final geotechnical investigation report should be prepared. Updated grading and foundation recommendations specific to the project can be provided at that time. Preliminary recommendations are provided in this report.
- 7.1.3 Subsurface conditions, as observed in our trenches, are expected to be relatively consistent across the site; however, variations in subsurface conditions are possible.
- 7.1.4 Our field investigation indicates that the site is underlain by topsoil, weathered granitic rock and granitic rock. Topsoil and highly weathered granitic rock are not adequate for support of settlement-sensitive structures and should be removed and replaced as compacted fill.
- 7.1.5 With the exception of the possibility of strong seismic shaking, no significant geologic hazards were observed or are known to exist at the site or other locations that could adversely affect the proposed project.
- 7.1.6 Based on our research, no active, potentially active, or activity unknown faults are known to cross the site or are trending toward the site.
- 7.1.7 It is our opinion that the proposed development will not destabilize or result in settlement of adjacent properties.
- 7.1.8 The risks associated with liquefaction, ground rupture, landslides, and flooding hazards are low.
- 7.1.9 The planned structures can be supported on a conventional, shallow-footing system founded on properly compacted fill.

- 7.1.10 In general, cut slopes composed of the granitic rock or properly compacted fill should have a factor of safety of at least 1.5 at inclinations of 2:1 (horizontal:vertical), or flatter.
- 7.1.11 Proper drainage should be maintained. Recommendations for site drainage are provided herein.

7.2 Soil and Excavation Characteristics

- 7.2.1 Excavation of the topsoil and weathered granitic rock should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavations beyond the weathered mantel in the granitic rock will require very heavy effort and possible blasting to excavate.
- 7.2.2 We expect on-site soil to be both “expansive” (expansion index [EI] greater than 20) and “non-expansive” (EI of 20 or less) as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 7.2 presents soil classifications based on the expansion index. The on-site soils possess a “very low” to “low” expansion potential.

TABLE 7.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	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	

- 7.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site materials at the locations tested possess “Not Applicable” and “S0” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-08 Sections 4.2 and 4.3. 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.

- 7.2.4 Geocon Incorporated does not practice in the field of corrosion engineering. If improvements that could be susceptible to corrosion are planned, further evaluation by a corrosion engineer may be needed.

7.3 Preliminary Grading Recommendations

- 7.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix D and the County of San Diego Grading Ordinance. The recommendations presented in this section take precedence over those presented in Appendix D.
- 7.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the city inspector, 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.
- 7.3.3 Earthwork should be observed and compacted fill tested by representatives of Geocon Incorporated.
- 7.3.4 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. Existing utilities and foundations should be abandoned and completely removed. Material generated during stripping and/or site demolition should be exported from the site.
- 7.3.5 All compressible soil deposits, including topsoil and weathered granitic rock within areas where structural improvements and/or structural fill are planned, should be removed to expose firm competent Granitic 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.
- 7.3.6 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.

- 7.3.7 Grading operations should be scheduled to permit the placement of oversize material 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).
- 7.3.8 The upper 3 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.
- 7.3.9 Cut pads exposing hard rock and cut/fill transition building pads should be undercut at least 3 feet and replaced with properly compacted “very low” to “low” expansive soil. The base of the undercuts should be sloped towards the down-gradient portion of the lot.
- 7.3.10 Undercutting of street areas and utilities should be performed in cut areas or areas where utilities will extend through the fill into non-rippable granitic rock to facilitate excavation of underground utilities. Undercuts should extend to at least 2 feet below the bottom of the utility. If subsurface improvements or landscape zones are planned outside these areas, consideration should be given to undercutting these areas as well.
- 7.3.11 The areas of the proposed on-site septic fields should be left in their natural condition. Grading or disturbance should be prohibited in these areas as it could invalidate the area for use as a pressurized drip disposal system.
- 7.3.12 Oversize material (defined as material greater than 12 inches in nominal dimension) may be generated during excavation of Granitic Rock. Placement of oversize material within fills should be conducted in accordance with the recommendations in Appendix D.
- 7.3.13 Capping material for building pads should be at least three 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 $\frac{3}{4}$ -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 3 feet below design grade and 15 feet from the face of slopes. The grading contractor should take necessary steps to manage the available soils to cap the project.

- 7.3.14 Based on our field investigation, we expect the on-site surficial soils and decomposed granite from excavations within the weathered granitic rock mantel will be suitable for capping and use as wall backfill.
- 7.3.15 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.
- 7.3.16 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.
- 7.3.17 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.

7.4 Slopes

- 7.4.1 Slope stability analyses were performed for proposed fill slopes utilizing shear strength parameters based on laboratory testing performed for this investigation. These analyses indicate that the proposed 2:1 fill slopes 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 35 feet. Slope stability calculations and graphical printouts for both deep-seated and surficial slope stability for fill slopes are presented on Figures 4 and 5.
- 7.4.2 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. The strength parameters used are considered conservative for Granitic Rock. Based on our analysis and experience with similar rock conditions, 2:1

cut slopes to the planned heights of up to 25 feet possess a factor of safety of at least 1.5 with respect to global stability, if free of adversely oriented joints or fractures.

- 7.4.3 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.
- 7.4.4 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.
- 7.4.5 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.
- 7.4.6 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.

7.5 Seismic Design Criteria

- 7.5.1 We used SEAOC (2018) to summarize site-specific design criteria obtained (Table 7.5.1) from the 2016 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. Building pads underlain by 15 feet of fill or less should be designed using a Site Class C. Building pads underlain by fills thicker than 15 feet should be designed using a Site Class D. We evaluated the Site Class based our experience for the site subsurface soils and exploratory boring information in accordance with Section 1613.3.2 of the 2016 CBC, and Table 20.3-1 of ASCE 7-10. The values presented in Table 7.5.1 are for the risk-targeted maximum considered earthquake (MCE_R).

**TABLE 7.5.1
2016 CBC SEISMIC DESIGN PARAMETERS**

Parameter	Value		2016 CBC Reference
Site Class	C	D	Table 1613.3.2
Fill Thickness, T (feet)	$T \leq 15$	$T > 15$	--
MCE _R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.021	1.021	Figure 1613.3.1(1)
MCE _R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.392	0.392	Figure 1613.3.1(2)
Site Coefficient, F_A	1.000	1.091	Table 1613.3.3(1)
Site Coefficient, F_V	1.408	1.615	Table 1613.3.3(2)
Site Class Modified MCE _R Spectral Response Acceleration (short), S_{MS}	1.021	1.115	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE _R Spectral Response Acceleration – (1 sec), S_{M1}	0.552	0.634	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.681	0.743	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.368	0.423	Section 1613.3.4 (Eqn 16-40)

7.5.2 Table 7.5.2 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 7.5.2
2016 CBC SEISMIC DESIGN PARAMETERS**

Parameter	Site Class C	Site Class D	ASCE 7-10 Reference
Mapped MCE _G Peak Ground Acceleration, PGA	0.381	0.381	Figure 22-7
Site Coefficient, F_{PGA}	1.019	1.119	Table 11.8-1
Site Class Modified MCE _G Peak Ground Acceleration, PGA_M	0.388	0.426	Section 11.8.3 (Eqn 11.8-1)

7.5.3 Conformance to the criteria in Table 7.5.1 and 7.5.2 does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.6 Preliminary Foundation Recommendations

- 7.6.1 The preliminary foundation recommendations that follow are for one- to three-story residential structures and are separated into categories dependent on the thickness and geometry of the underlying fill soils as well as the expansion index of the prevailing subgrade soils. Final foundation categories should be determined for each lot after grading and finish pads have been established and laboratory expansion index testing performed.

**TABLE 7.6.1
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	$T \geq 50$	$D \geq 20$	$90 < EI \leq 130$

- 7.6.2 Table 7.6.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

**TABLE 7.6.2
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	6x6-10/10 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

- 7.6.3 The embedment depths presented in Table 7.6.2 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. Figure 7 presents a wall/column footing dimension detail.
- 7.6.4 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.

- 7.6.5 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. 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). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements, and in a manner that prevents puncture. The project architect or developer should specify the vapor retarder based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.
- 7.6.6 The project foundation engineer, architect, and/or developer should determine the thickness of bedding sand below the slab. In general, 3 to 4 inches of sand bedding is typically used. Geocon should be contacted to provide recommendations if the bedding sand is thicker than 6 inches.
- 7.6.7 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. The foundation design engineer should specify the concrete mix design and proper curing methods on the foundation plan. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plan.
- 7.6.8 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 2016 CBC has updated the design requirements for post-tensioned foundation systems. 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), Third Edition, as required by the 2016 CBC (Section 1805.8). Although this procedure was developed for expansive soil conditions, we understand 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 7.6.3 for the particular Foundation Category designated. The parameters presented in Table 7.6.3 are based on the guidelines presented in the PTI, Third Edition design manual.

TABLE 7.6.3
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

7.6.9 If the structural engineer proposes a post-tensioned foundation design method other than the 2016 CBC:

- The criteria presented in Table 7.6.3 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.

7.6.10 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 at least 6 inches below the clean sand or crushed rock layer.

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

7.6.12 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.

- 7.6.13 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. Differential settlement is estimated to occur over a span of 40 feet.
- 7.6.14 Isolated footings, including PT foundation systems where footings are not reinforced with PT cables, should have the minimum embedment depth and width recommended for conventional foundations (see Section 7.6.1 through 7.6.3) 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.
- 7.6.15 For Foundation Category III, consideration should be given to using interior stiffening beams and connecting isolated footings and/or increasing the slab thickness. In addition, consideration should be given to connecting patio slabs, which exceed five feet in width, to the building foundation to reduce the potential for future separation to occur.
- 7.6.16 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 appropriate in any such concrete placement.
- 7.6.17 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal:vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.
- For fill slopes less than 20 feet high 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.
 - For fill slopes greater than 20 feet high, 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 help reduce potential foundation distress associated with slope creep 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 that 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 that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

7.6.18 The exterior flatwork recommendations provided herein assumes that the near surface soils are very low to low expansive ($EI \leq 50$). Exterior slabs not subjected to vehicular traffic should be a minimum of four inches thick, and when panels are in excess of 8 feet wide, reinforced with 6 x 6-6/6 welded wire mesh. The mesh should be placed in the middle of the slab. Proper mesh positioning is critical to future performance of the slabs. The contractor should take extra measures to provide proper mesh placement. Prior to construction of slabs, the upper 12 inches of subgrade soils should be moisture conditioned at or slightly above optimum moisture content and compacted to at least 90 percent of the laboratory maximum dry density per ASTM 1557.

7.6.19 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. The occurrence may be reduced and/or controlled by: (1) limiting the slump of the concrete, (2) proper concrete placement and curing, and by (3) the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

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

7.7 Retaining Walls and Lateral Loads

- 7.7.1 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 soil should not be used as backfill material behind retaining walls. Soil placed for retaining wall backfill should have an Expansion Index less than 50.
- 7.7.2 Where walls are restrained from movement at the top, an additional uniform pressure of $8H$ psf (where H equals the height of the retaining wall portion of the wall in feet) should be added to the active soil pressure where the wall possesses a height of 8 feet or less and $12H$ where the wall is greater than 8 feet. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to two feet of fill soil should be added.
- 7.7.3 Soil to be used as backfill should be stockpiled and samples obtained for laboratory testing to evaluate its suitability for use as wall backfill. Modified lateral earth pressures will be required if backfill soils do not meet the required expansion index. County standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. On-site soils might not meet the design values used for County standard wall design. Geocon Incorporated should be consulted if County standard wall designs will be used to assess the suitability of on-site soil for use as wall backfill.
- 7.7.4 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The wall designer should provide appropriate lateral deflection quantities for planned retaining walls structures, if applicable. These lateral values should be considered when planning types of improvements above retaining wall structures.
- 7.7.5 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) free-draining backfill material with no hydrostatic forces or imposed surcharge load. A typical retaining wall drainage detail is presented on Figure 8. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

- 7.7.6 In general, wall foundations having a minimum depth and width of 1 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 less than 90. 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. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon Incorporated should be consulted where such a condition is expected.
- 7.7.7 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 $15H$ should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M , of 0.426 g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.
- 7.7.8 For resistance to lateral loads, an allowable 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 allowable passive pressure assumes a horizontal surface extending away from the base of the wall at least 5 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.
- 7.7.9 An allowable friction coefficient of 0.4 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

- 7.7.10 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of eight feet. In the event that walls higher than eight feet or other types of walls (i.e., soil nail, MSE walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

7.8 Preliminary Pavement Recommendations

- 7.8.1 The following preliminary pavement design sections are based on our experience with soil conditions within the surrounding area and preliminary R-value test results. The preliminary sections presented herein are for budgetary estimating purposes only and are not for construction. Final pavement sections should be determined after the grading operations are completed, subgrade soils are exposed, and additional R-Value tests are performed on actual pavement subgrade samples. For preliminary design, we used a resistance value (R-Value) of 40 for subgrade soils and 78 for aggregate base.
- 7.8.2 Asphalt concrete pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans).
- 7.8.3 The project civil engineer or traffic engineer should determine the actual road classification and the appropriate Traffic Index (TI) for the project. Table 7.8 provides preliminary pavement design sections for a residential road.

**TABLE 7.8
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS**

Road Classification	Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Residential Road	5	3	4

- 7.8.4 Class 2 aggregate base materials should conform to Section 26-1.02B of the *Standard Specifications of the State of California, Department of Transportation* (Caltrans) or Sections 400-2 and 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*. The aggregate base specifications are found in the *Regional Supplemental to Greenbook*.
- 7.8.5 Pavement subgrade soils should be scarified, moisture conditioned as necessary, and compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. The

depth of compaction should be at least 12 inches. Base course material should be moisture conditioned near to slightly above optimum moisture content and compacted to a dry density of at least 95 percent of the laboratory maximum dry density. Asphalt concrete pavement should be compacted to at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

- 7.8.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Allowing water to pond on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent pavement distress. Where landscape or planter islands are planned adjacent to pavement surfaces, the perimeter curb should extend at least 6 inches below the bottom of the Class 2 aggregate base and into the underlying subgrade. Drainage from landscaped areas should be directed to controlled drainage structures.

7.9 Storm Water Management

- 7.9.1 If storm water management devices are not properly designed and constructed, there is a risk for distress to improvements and property 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.
- 7.9.2 We performed an infiltration study on the property. A summary of our study and storm water management recommendations are provided in Appendix C.

7.10 Site Drainage and Moisture Protection

- 7.10.1 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.

- 7.10.2 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.
- 7.10.3 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.
- 7.10.4 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.

7.11 Slope Maintenance

- 7.11.1 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.

7.12 Grading and Foundation Plan Review

- 7.12.1 Geocon Incorporated should review the final grading and foundation plans for the project prior to final design submittal to evaluate if additional analysis and/or recommendations are required.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.
2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.
3. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.
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 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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NO SCALE

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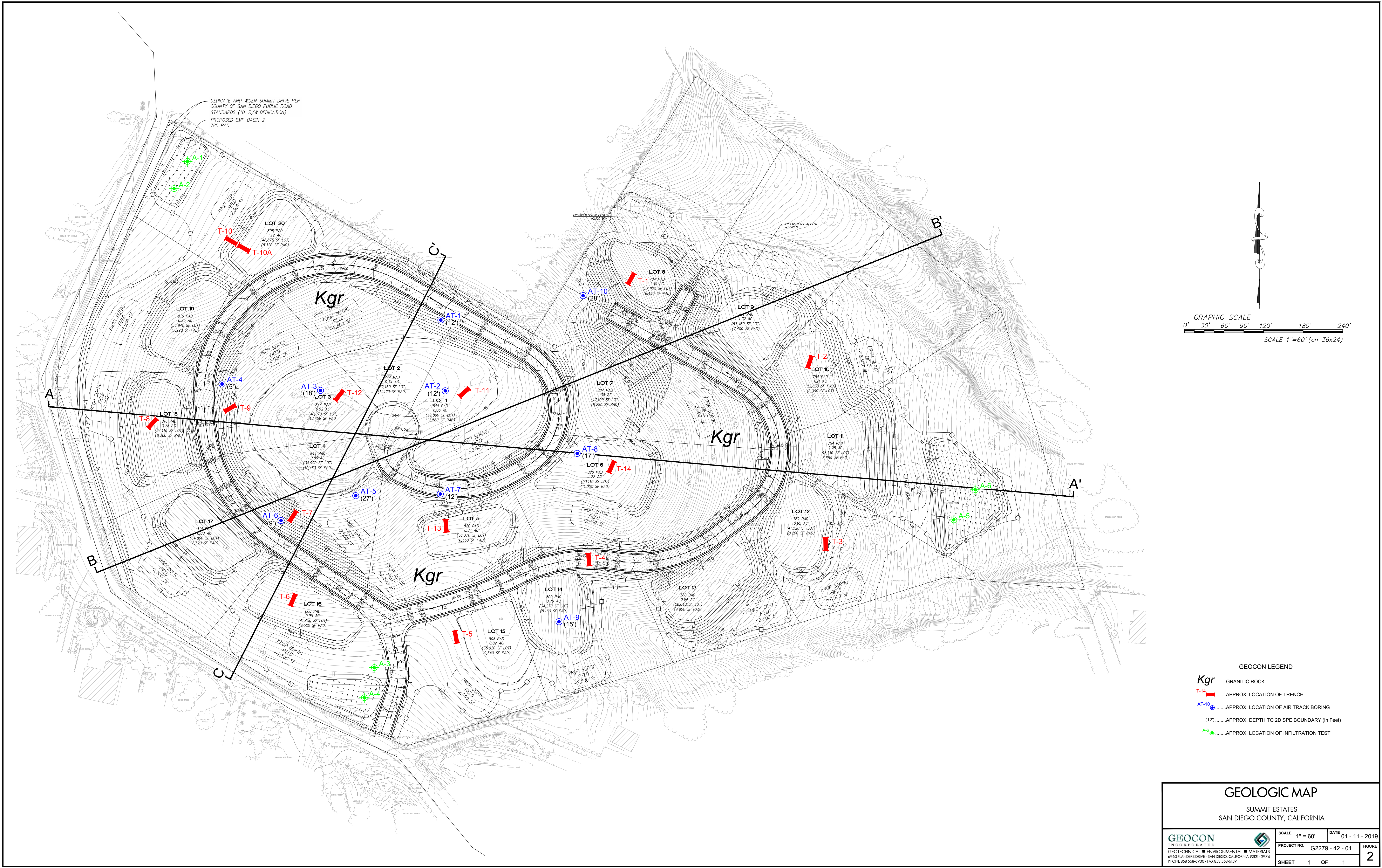
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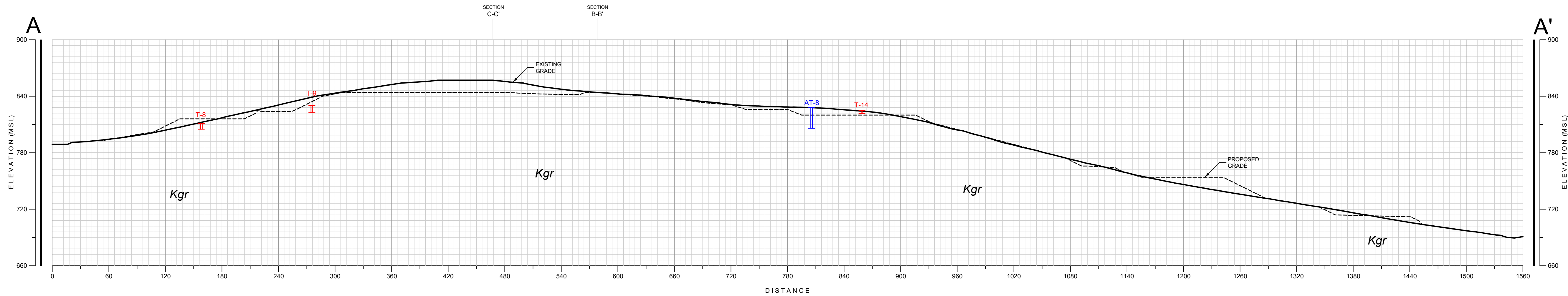
FIG. 1



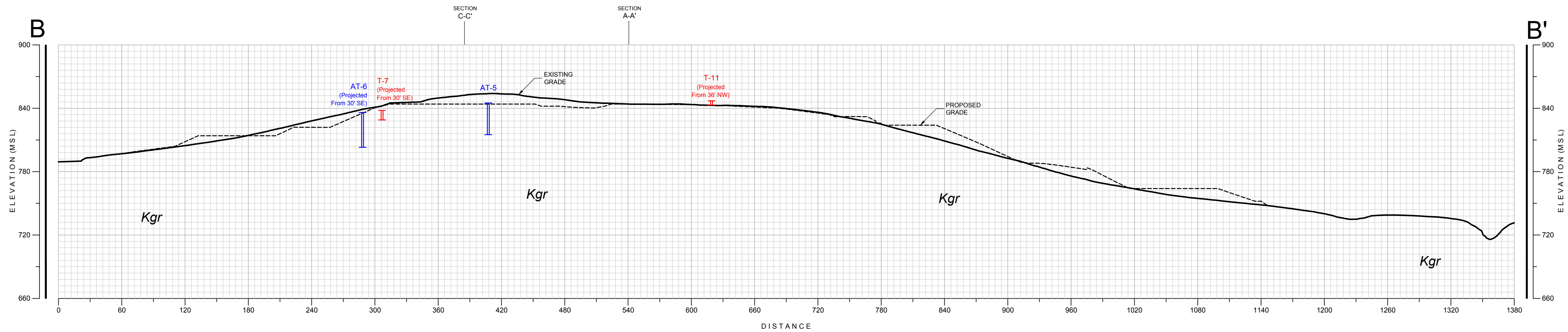
GRAPHIC SCALE
0' 30' 60' 90' 120' 180' 240'
SCALE 1"=60' (on 36x24)

GEOCON INCORPORATED			SCALE 1" = 60'	DATE 01 - 11 - 2019
GEOLOGIC MAP SUMMIT ESTATES SAN DIEGO COUNTY, CALIFORNIA			PROJECT NO. G2279 - 42 - 01	FIGURE 2
SHEET 1 OF 1				

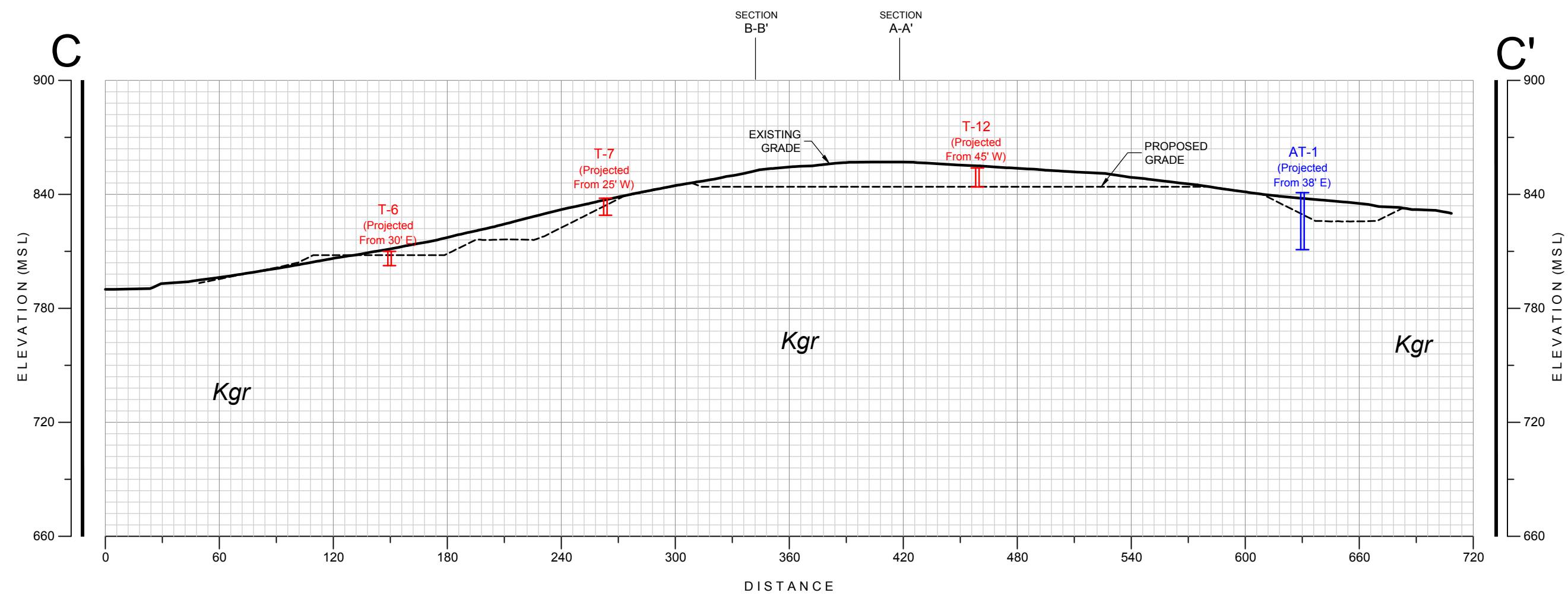
Plotted 01/11/2019 2:24PM | By: ALVIN LACRELLON | File Location: Y:\PROJECTS\G2279-42-01 (The Summit)\SHEETS\G2279-42-01 Geo Map.dwg



GEOLOGIC CROSS-SECTION A-A'
SCALE: 1" = 60' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION B-B'
SCALE: 1" = 60' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION C-C'
SCALE: 1" = 60' (Vert. = Horiz.)

GEOCON LEGEND

KgrGRANITIC ROCK

T-14APPROX. LOCATION OF TRENCH

AT-8APPROX. LOCATION OF AIR TRACK BORING

GEOLOGIC CROSS SECTION				
SUMMIT ESTATES SAN DIEGO COUNTY, CALIFORNIA				
GEOCON INCORPORATED GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS 6950 TANDERS DRIVE, SAN DIEGO, CALIFORNIA 92121 - 2974 PHONE 858.558.4900 - FAX 858.558.4159	SCALE 1" = 60'	DATE 01 - 11 - 2019	FIGURE 3	
	PROJECT NO. G2279 - 42 - 01			
	SHEET 1 OF 1			

ASSUMED CONDITIONS :

SLOPE HEIGHT	H = 35 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 :

$\lambda_{c\phi}$	=	$\frac{\gamma_t H \tan \phi}{C}$	EQUATION (3-3), REFERENCE 1
FS	=	$\frac{NcfC}{\gamma_t H}$	EQUATION (3-2), REFERENCE 1
$\lambda_{c\phi}$	=	13.1	CALCULATED USING EQ. (3-3)
Ncf	=	38	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	1.7	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 - FILL SLOPES

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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 = 300 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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FIG. 5

ASSUMED CONDITIONS :

SLOPE HEIGHT	H = 25 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t = 135 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ = 40 degrees
APPARENT COHESION	C = 100 pounds per square foot
NO SEEPAGE FORCES	

ANALYSIS :

$\lambda_{c\phi}$	=	$\frac{\gamma_t H \tan \phi}{C}$	EQUATION (3-3), REFERENCE 1
FS	=	$\frac{NcfC}{\gamma_t H}$	EQUATION (3-2), REFERENCE 1
$\lambda_{c\phi}$	=	28	CALCULATED USING EQ. (3-3)
Ncf	=	70	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	2.1	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 - CUT SLOPES

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SUMMER ESTATES
SAN DIEGO COUNTY, CALIFORNIA

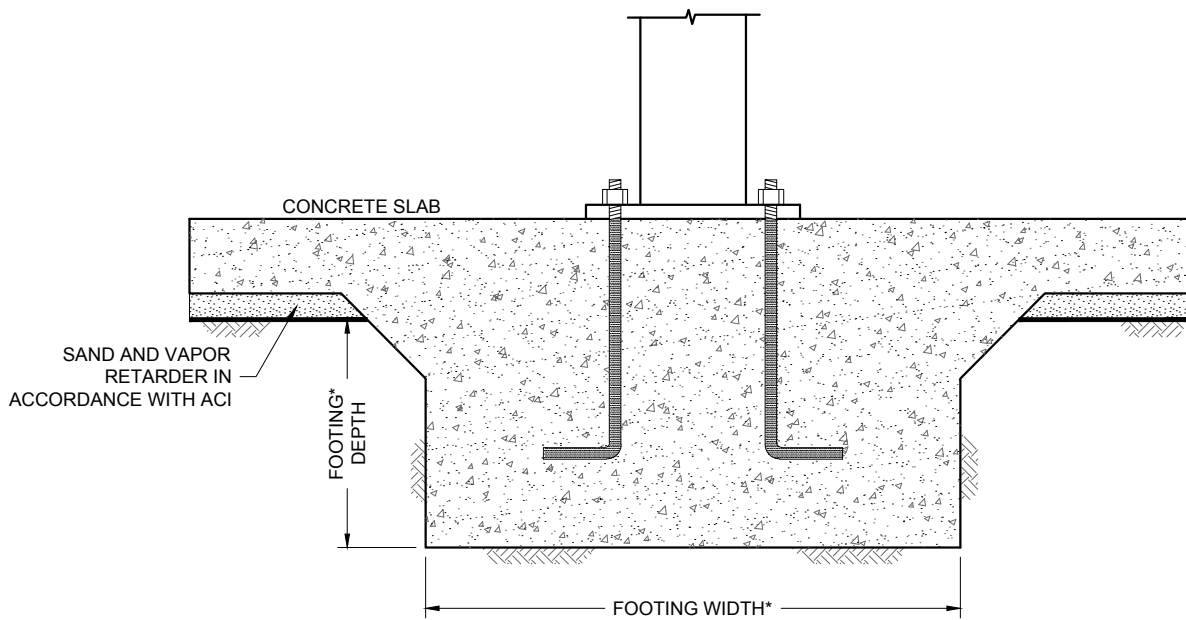
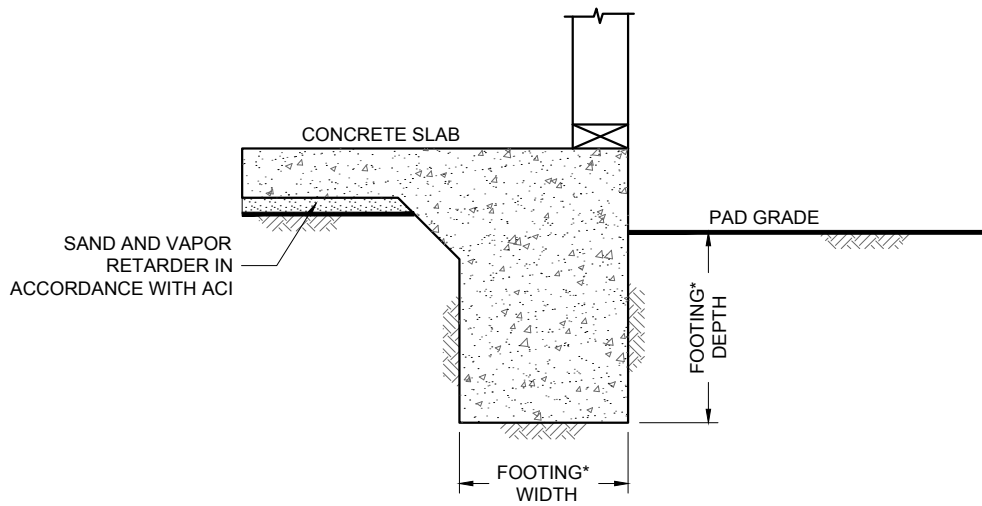
RM / AML

DSK/GTYPD

DATE 01 - 11 - 2019

PROJECT NO. G2279 - 42 - 01

FIG. 6



*SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

GEOCON
INCORPORATED



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

RM / AML

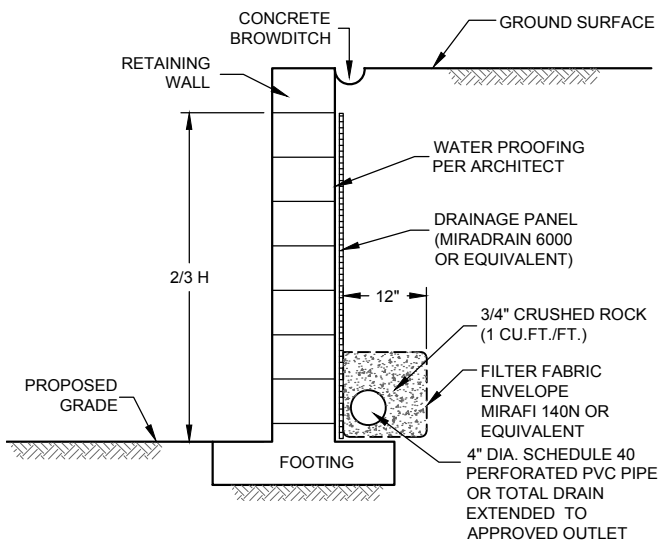
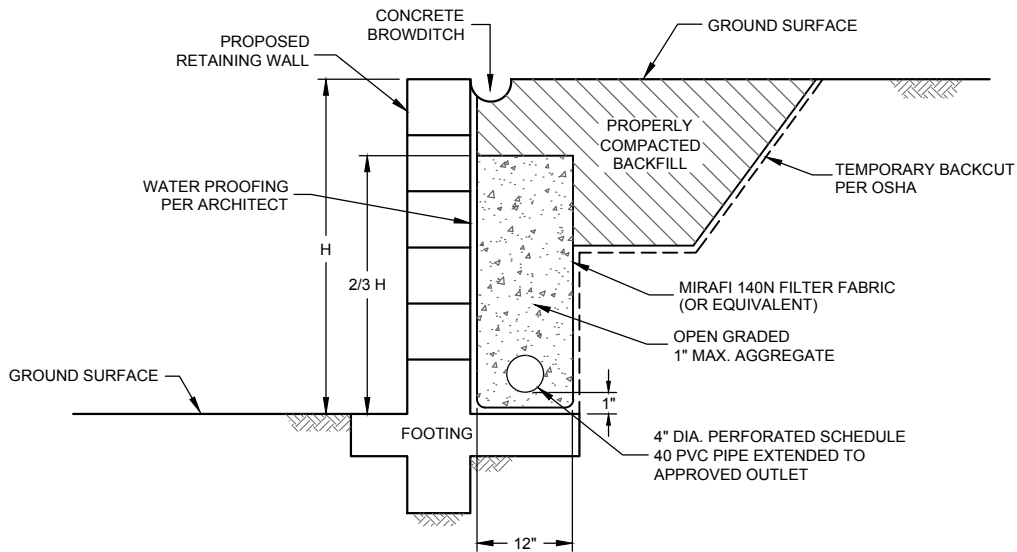
DSK/GTYPD

SUMMER ESTATES
SAN DIEGO COUNTY, CALIFORNIA

DATE 01 - 11 - 2019

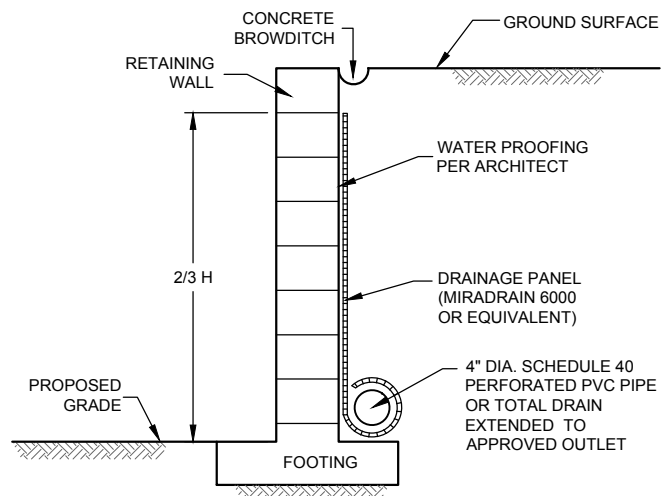
PROJECT NO. G2279 - 42 - 01

FIG. 7



NOTE :

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



NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

GEOCON
INCORPORATED



GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 297 4
PHONE 858 558-6900 - FAX 858 558-6159

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SUMMER ESTATES
SAN DIEGO COUNTY, CALIFORNIA

DATE 01 - 11 - 2019

PROJECT NO. G2279 - 42 - 01

FIG. 8

APPENDIX

A







APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on December 7 and 13, 2018. The investigation consisted of drilling 10, air-percussion, borings and excavating fourteen, shallow, exploratory pits at the approximate locations shown on the *Site Plan*, Figure 2. The soil conditions encountered in the trenches were visually examined, classified and logged in general conformance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). The log of the exploratory test pits are presented on Figures A-1 through A-14. The log depicts the various soil types encountered and indicate the depths at which samples were taken. Logs of the air-track percussion borings are shown on Figures A-15 through A-24.

G2279-42-01.GPJ

SAMPLE SYMBOLS

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 2 ELEV. (MSL.) 747' DATE COMPLETED 12-13-2018 EQUIPMENT JD 410 BACKHOE BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
2					TOPSOIL Loose, moist, dark brown, Silty, fine to medium SAND; trace gravel			
4	T2-1				WEATHERED GRANITIC ROCK Weak, damp, olive brown, Silty, fine to medium SAND; excavates with chunks of granitic rock			
6								
8					GRANITIC ROCK Moderately weathered, weak, damp, yellowish brown, GRANITIC ROCK; excavates as Silty, fine to coarse SAND			
10					TRENCH TERMINATED AT 10 FEET Groundwater not encountered			

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







G2279-42-01.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.

G2279-42-01.GPJ

SAMPLE SYMBOLS




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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	<div>TRENCH T 4</div> <div>ELEV. (MSL.) 796' DATE COMPLETED 12-13-2018</div> <div>EQUIPMENT JD 410 BACKHOE BY: N. BORJA</div>	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
2					TOPSOIL Loose, moist, dark brown, Silty, fine to medium SAND; few cobble up to 12-inch diameter			
4					GRANITIC ROCK Moderately weathered, moderately weak, damp, light reddish brown and brown, GRANITIC ROCK; excavates as Silty, fine to coarse SAND			
					REFUSAL AT 4 FEET Groundwater not encountered			

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

G2279-42-01.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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 5 ELEV. (MSL.) 806' DATE COMPLETED 12-13-2018 EQUIPMENT JD 410 BACKHOE BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
2					TOPSOIL Loose, moist, dark brown, Silty, fine to medium SAND; trace gravel			
4	T5-1				GRANITIC ROCK Moderately weathered, weak, dry, light reddish brown, GRANITIC ROCK; excavates as Silty, fine to coarse SAND -Becomes olive brown			
6					REFUSAL AT 6 FEET Groundwater not encountered			

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







G2279-42-01.GPJ

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

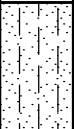
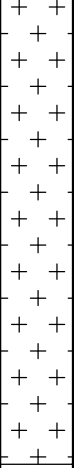
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

G2279-42-01.GPJ

SAMPLE SYMBOLS

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







GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 7		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) <u>838'</u>	DATE COMPLETED <u>12-13-2018</u>			
					EQUIPMENT <u>JD 410 BACKHOE</u> BY: <u>N. BORJA</u>				
					MATERIAL DESCRIPTION				
0					TOPSOIL Loose, damp to moist, dark brown, Silty, fine to medium SAND				
2					GRANITIC ROCK Moderately weathered, weak, damp, reddish brown, GRANITIC ROCK; excavates as Silty, fine to coarse SAND -Becomes yellowish brown -Hard digging below 7 feet				
8					PRACTICAL REFUSAL AT 9 FEET Groundwater not encountered				

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

G2279-42-01.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.


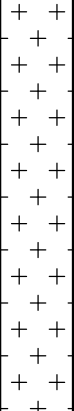
GEOCON

G2279-42-01.GPJ

SAMPLE SYMBOLS

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







GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 9		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) <u>830'</u>	DATE COMPLETED <u>12-13-2018</u>			
					EQUIPMENT <u>JD 410 BACKHOE</u> BY: <u>N. BORJA</u>				
					MATERIAL DESCRIPTION				
0					TOPSOIL Loose, moist, dark brown, Silty, fine to medium SAND; trace gravel				
2					GRANITIC ROCK Moderately, weak, damp, light brown, GRANITIC ROCK; excavates as Silty, fine to coarse SAND -Becomes olive brown				
6					-Hard digging at 7 feet TRENCH TERMINATED AT 7.5 FEET Groundwater not encountered				

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

G2279-42-01.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.

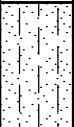
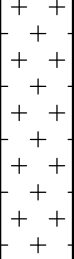
DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	<div>TRENCH T 10</div> <div>ELEV. (MSL.) 811' DATE COMPLETED 12-13-2018</div> <div>EQUIPMENT JD 410 BACKHOE BY: N. BORJA</div>	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
					TOPSOIL Loose, moist, dark brown, Silty, fine to medium SAND; trace gravel			
2					GRANITIC ROCK Moderately weak to strong, damp, light grayish brown, GRANITIC ROCK			
					REFUSAL AT 2.5 FEET Groundwater not encountered			

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

G2279-42-01.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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 10A		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) <u>812'</u>	DATE COMPLETED <u>12-13-2018</u>			
					EQUIPMENT <u>JD 410 BACKHOE</u>	BY: <u>N. BORJA</u>			
					MATERIAL DESCRIPTION				
0					TOPSOIL Loose, moist, dark brown, Silty, fine to medium SAND				
2					GRANITIC ROCK Moderately weathered, weak, light brown, GRANITIC ROCK; excavates as Silty, fine to coarse SAND -Becomes olive brown, fine- to medium-grained				
4									
6					TRENCH TERMINATED AT 6 FEET Groundwater not encountered				

**Figure A-11,
Log of Trench T 10A, Page 1 of 1**

G2279-42-01.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.

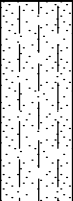




DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 11		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 847' DATE COMPLETED 12-13-2018	EQUIPMENT JD 410 BACKHOE BY: N. BORJA			
0					MATERIAL DESCRIPTION				
					TOPSOIL Loose, damp to moist, dark brown, Silty, fine to medium SAND				
2					-Becomes dry; porous				
					GRANITIC ROCK Moderately, weak to strong, dry, light brown, GRANITIC ROCK; excavates as Silty, fine to coarse SAND				
4					REFUSAL AT 4 FEET Groundwater not encountered				

Figure A-12,
Log of Trench T 11, Page 1 of 1







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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.

G2279-42-01.GPJ

SAMPLE SYMBOLS



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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	<div>TRENCH T 13</div> <div>ELEV. (MSL.) 822' DATE COMPLETED 12-13-2018</div> <div>EQUIPMENT JD 410 BACKHOE BY: N. BORJA</div>	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
					TOPSOIL Loose, moist, dark brown, Silty, fine to medium SAND; trace gravel			
2					GRANITIC ROCK Moderately weathered, weak to moderately weak, damp, light brown and gray, GRANITIC ROCK; excavates as Silty, fine to coarse SAND; some gravel			
4					TRENCH TERMINATED AT 4 FEET Groundwater not encountered			

Figure A-14,
Log of Trench T 13, Page 1 of 1

G2279-42-01.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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	<div>TRENCH T 14</div> <div>ELEV. (MSL.) 824' DATE COMPLETED 12-13-2018</div> <div>EQUIPMENT JD 410 BACKHOE BY: N. BORJA</div>	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
					TOPSOIL Loose, moist, dark brown, Silty, fine to medium SAND			
2					GRANITIC ROCK Moderately weathered, weak, damp, light brown to yellowish brown, GRANITIC ROCK; excavates as Silty, fine to coarse SAND			
					TRENCH TERMINATED AT 3.5 FEET Groundwater not encountered			

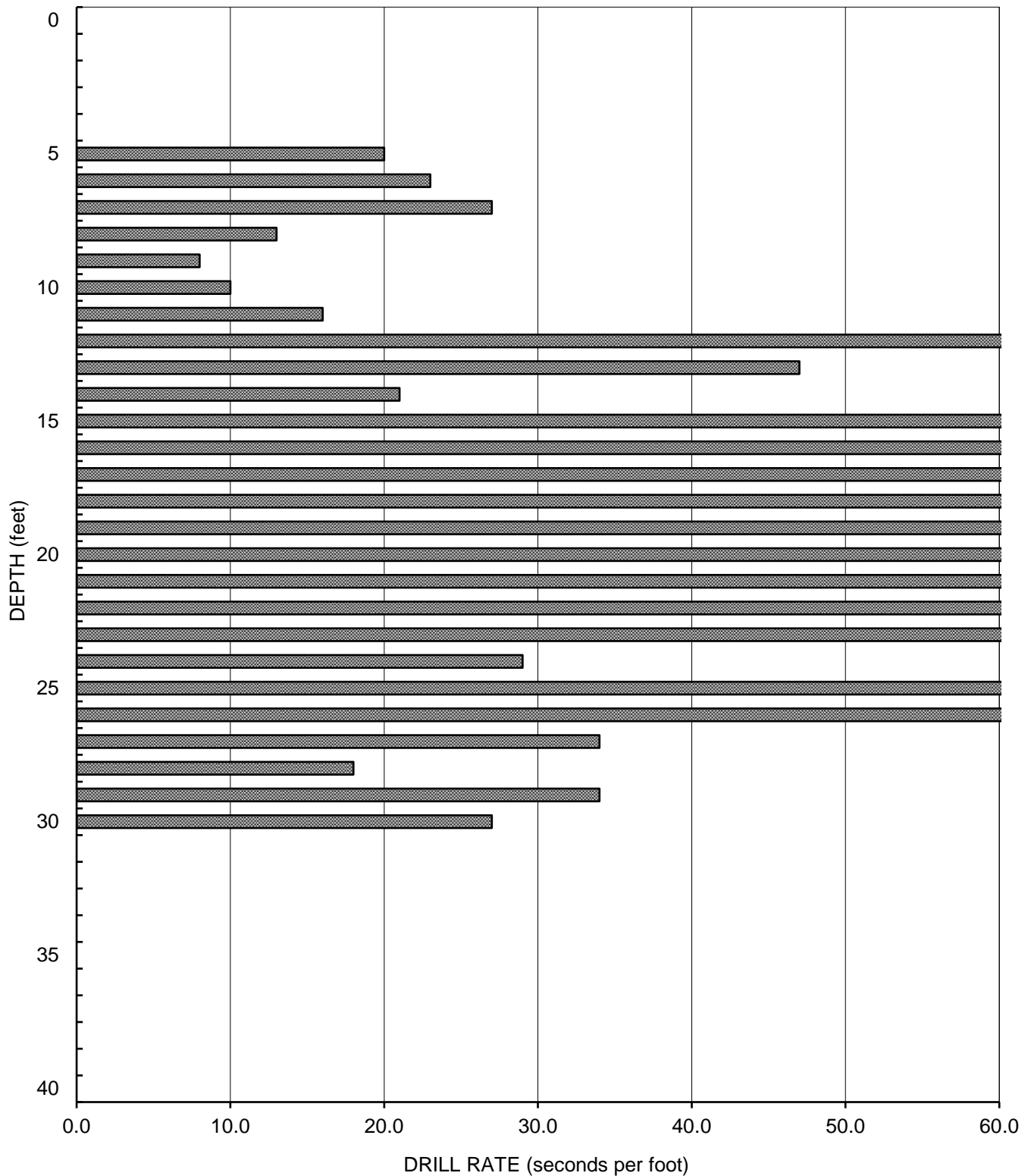
Figure A-15,
Log of Trench T 14, Page 1 of 1

G2279-42-01.GPJ

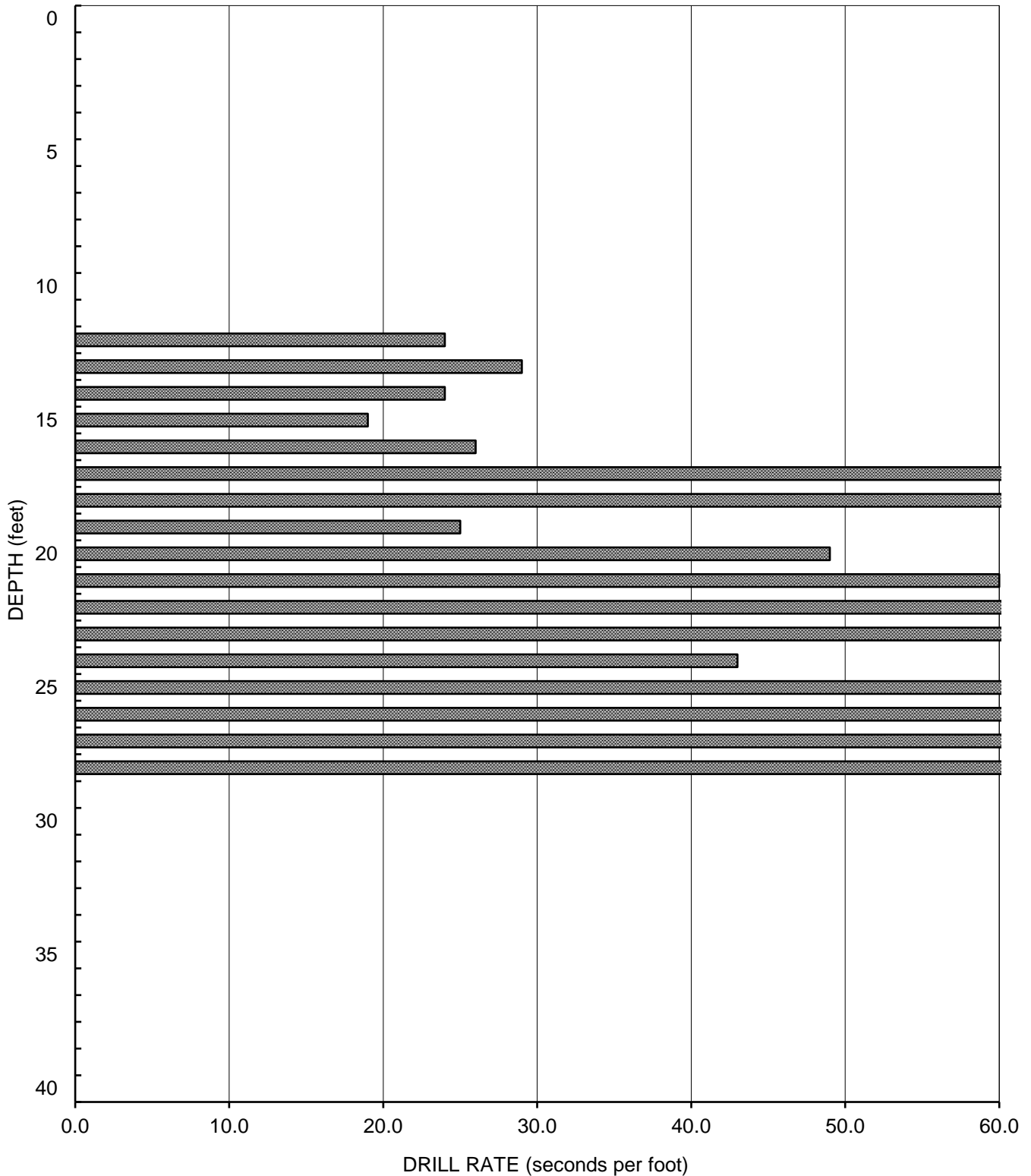
SAMPLE SYMBOLS		... SAMPLING UNSUCCESSFUL		... STANDARD PENETRATION TEST		... DRIVE SAMPLE (UNDISTURBED)
		... DISTURBED OR BAG SAMPLE		... CHUNK SAMPLE		... WATER TABLE OR SEEPAGE

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.

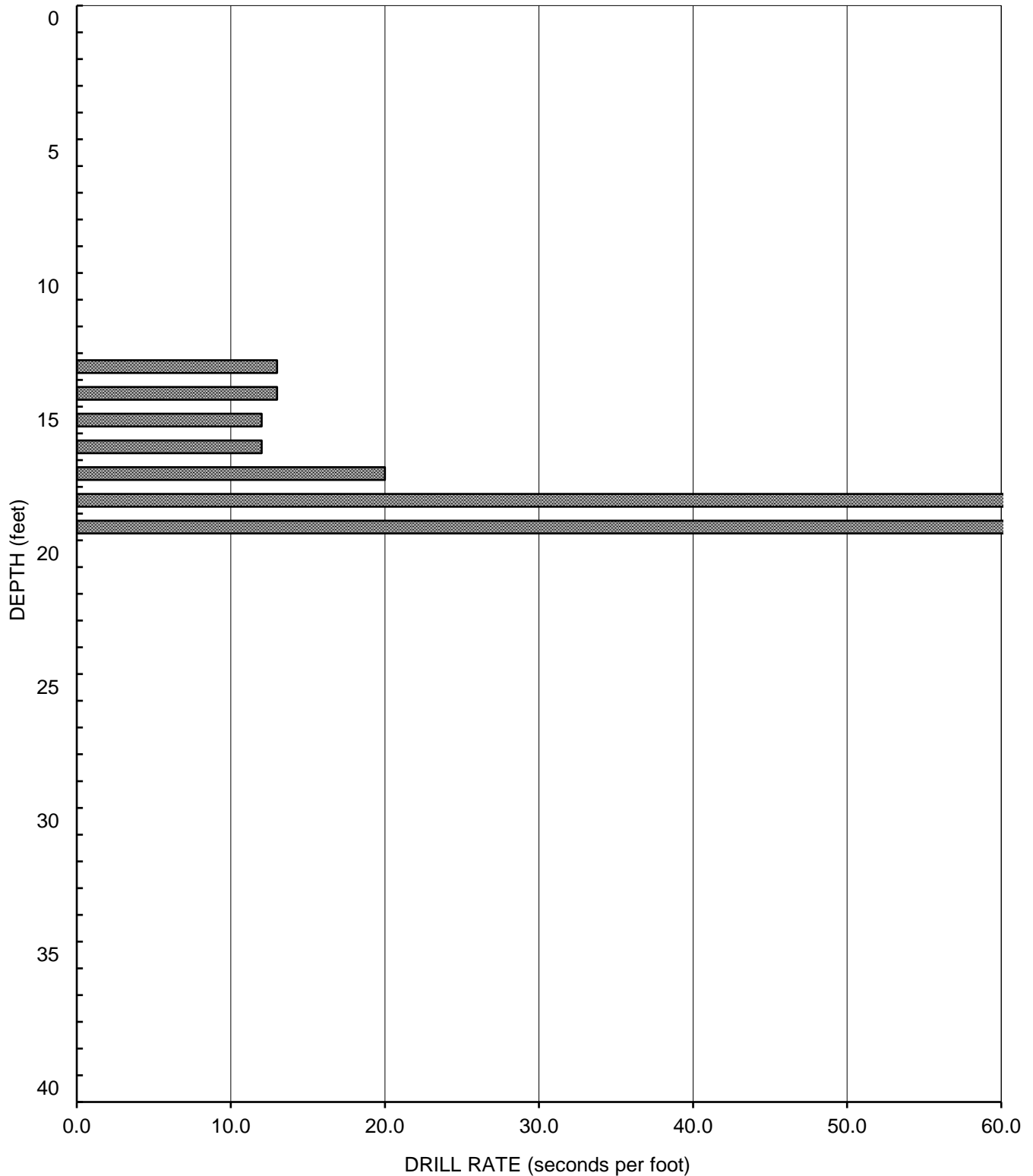
AIR TRACK BORING AT-1
Elevation - 841 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



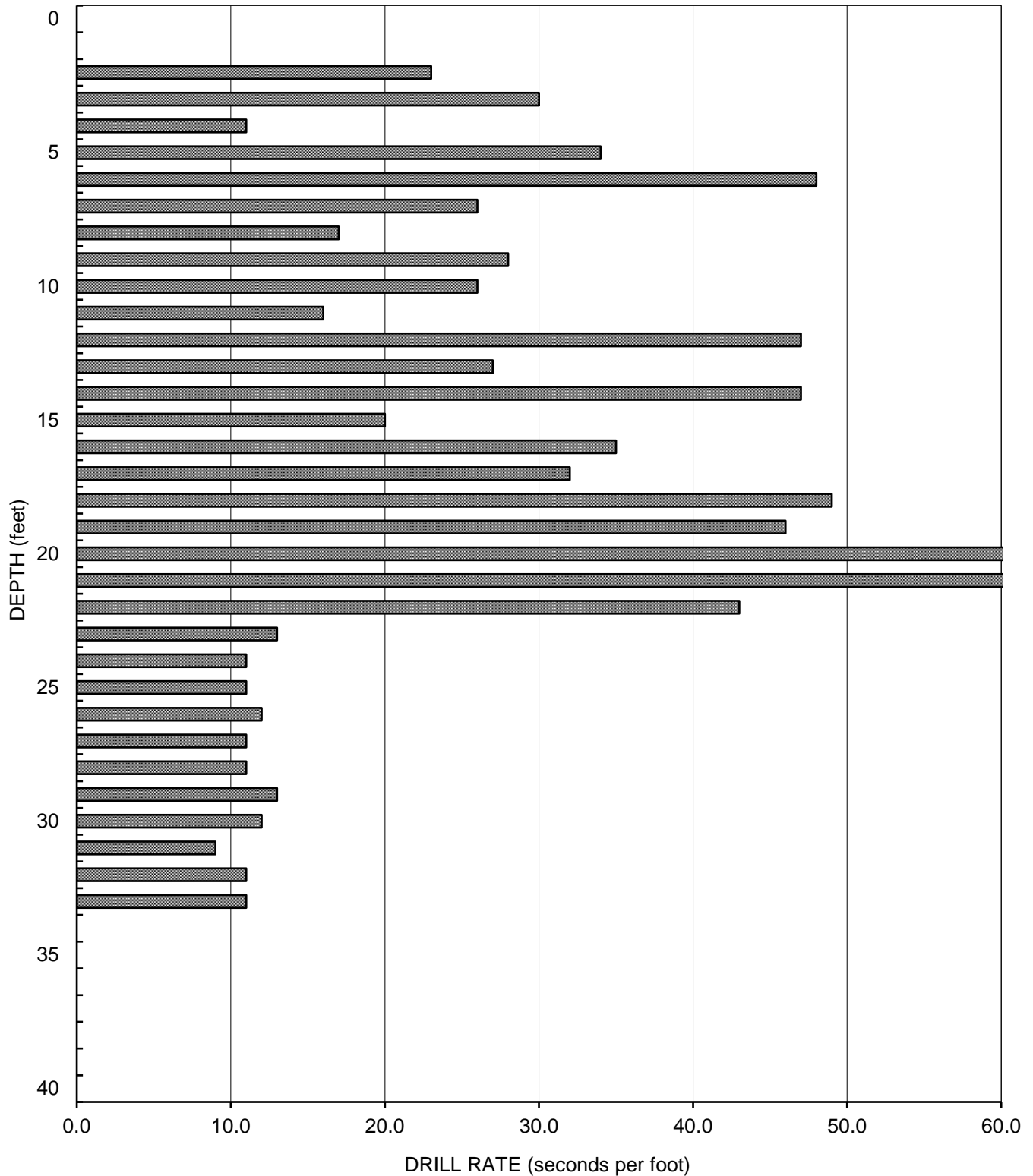
AIR TRACK BORING AT-2
Elevation - 849 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



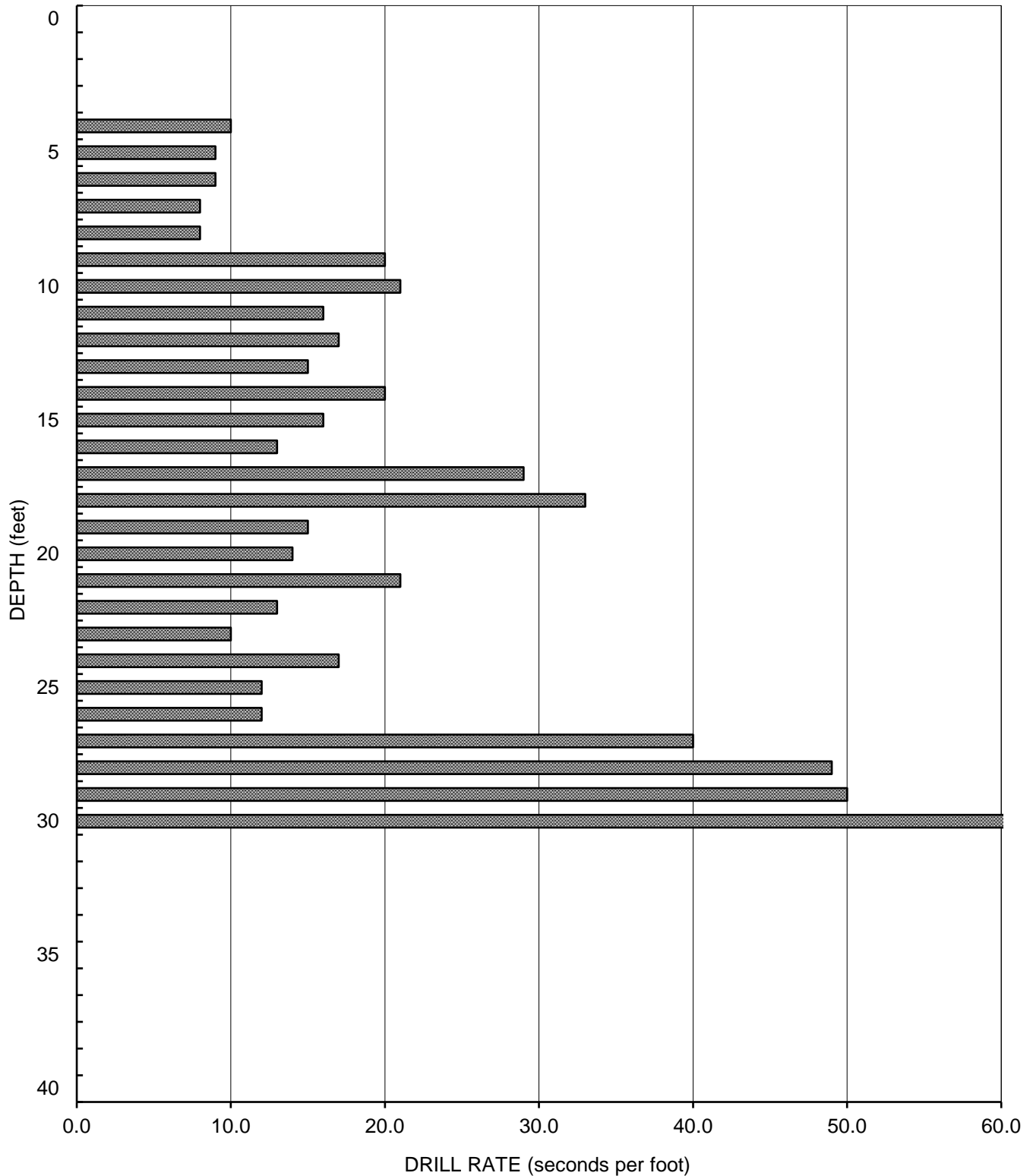
AIR TRACK BORING AT-3
Elevation - 852 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



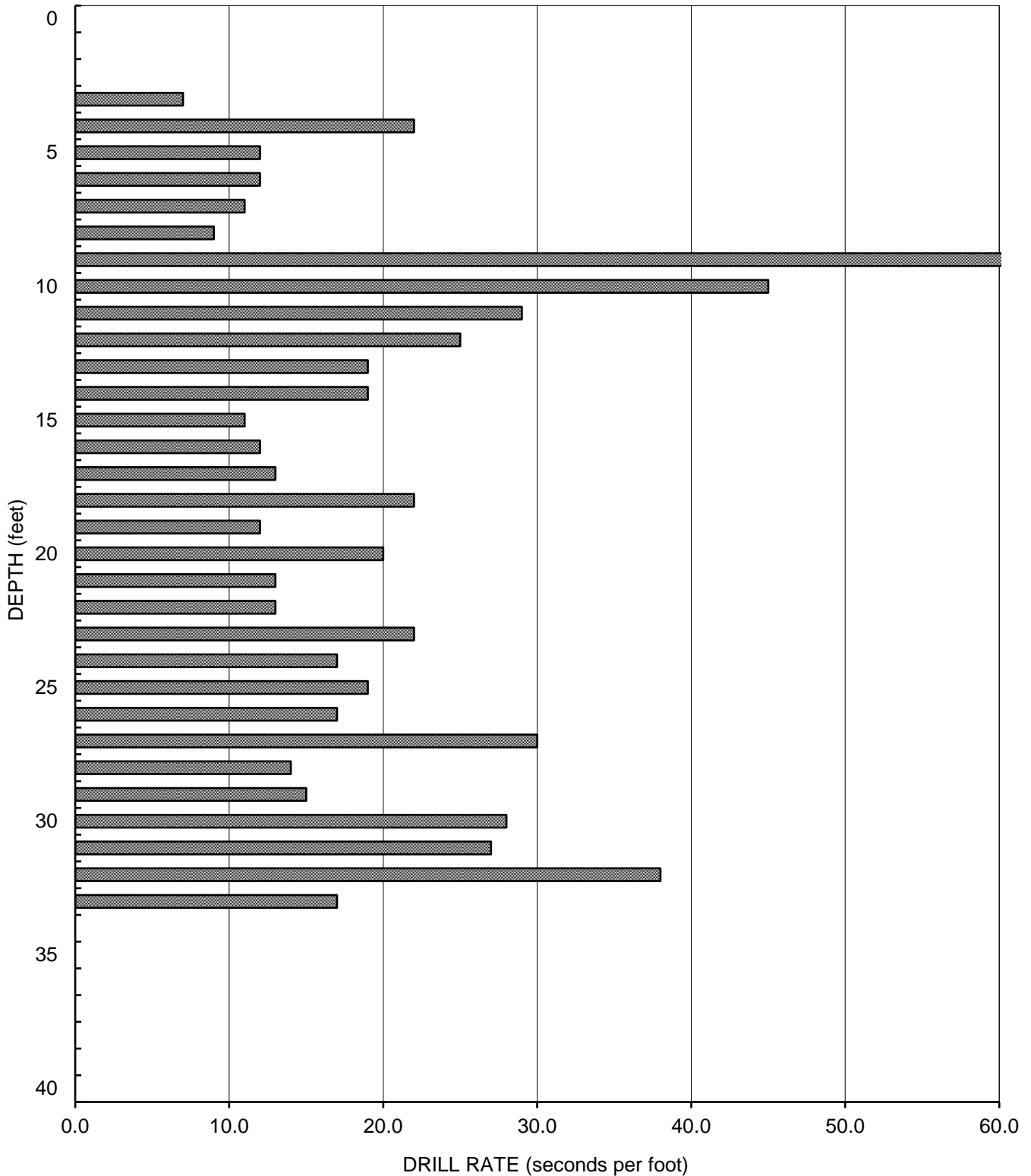
AIR TRACK BORING AT-4
Elevation - 830 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



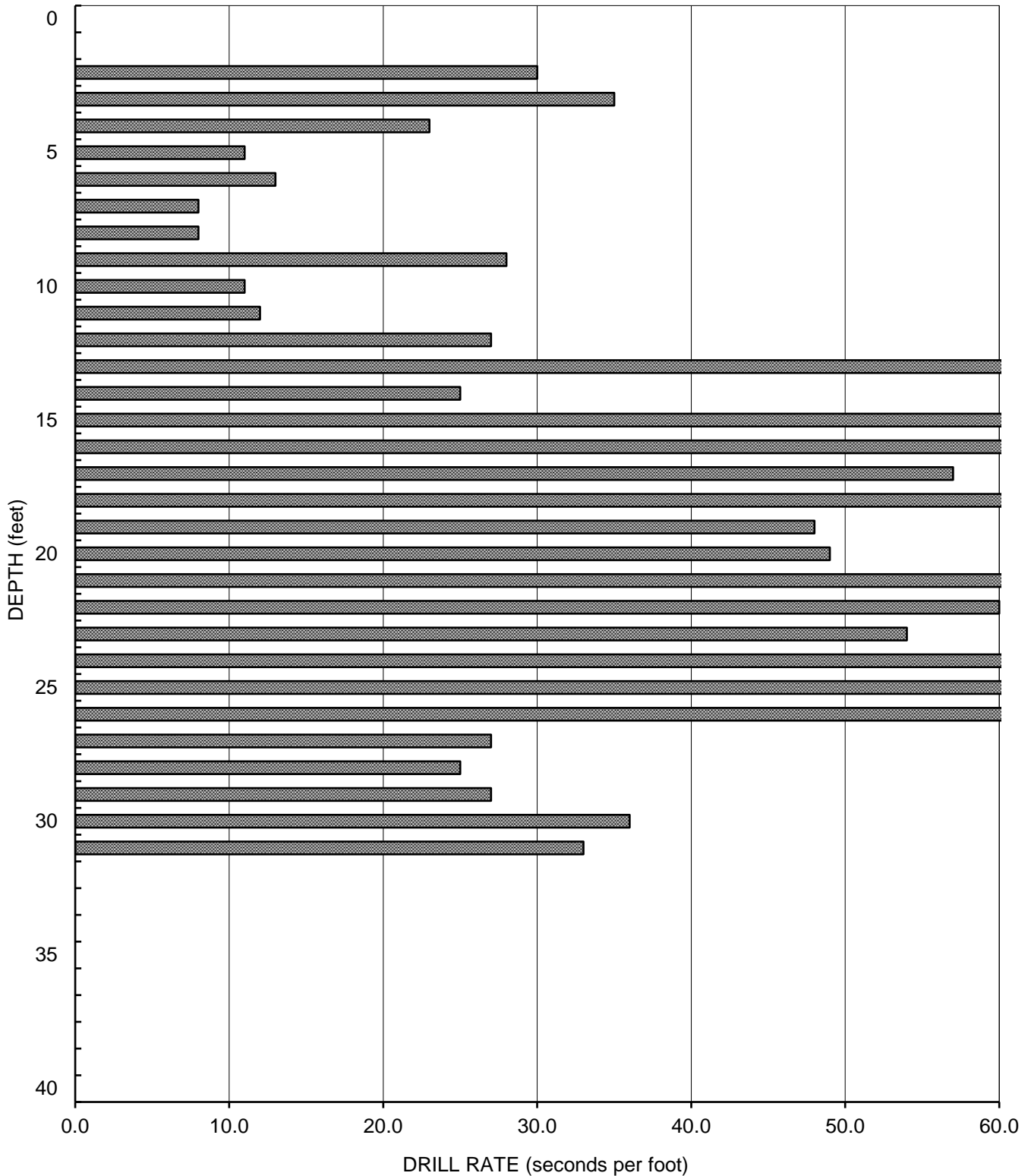
AIR TRACK BORING AT-5
Elevation - 845 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



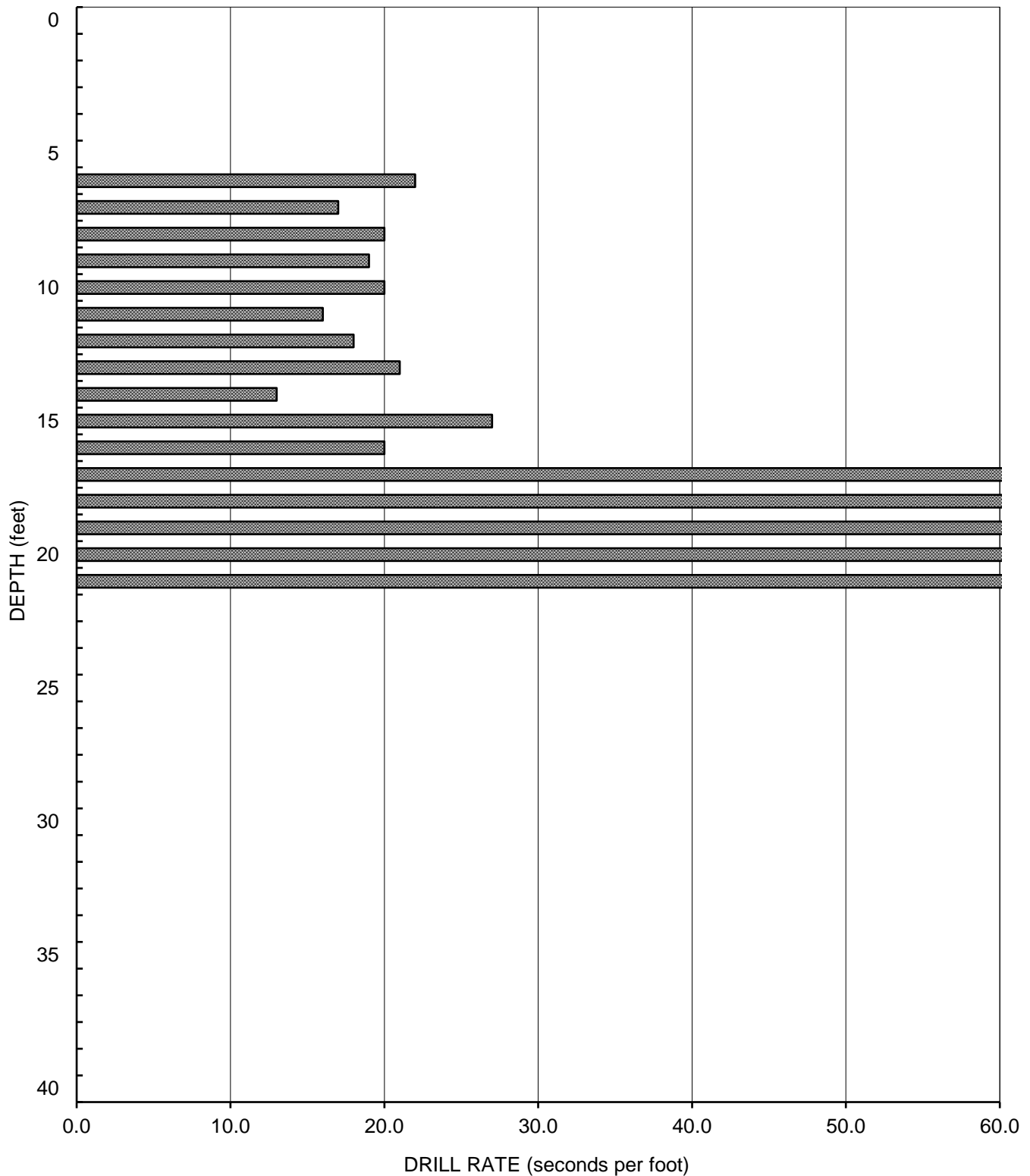
AIR TRACK BORING AT-6
Elevation - 836 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



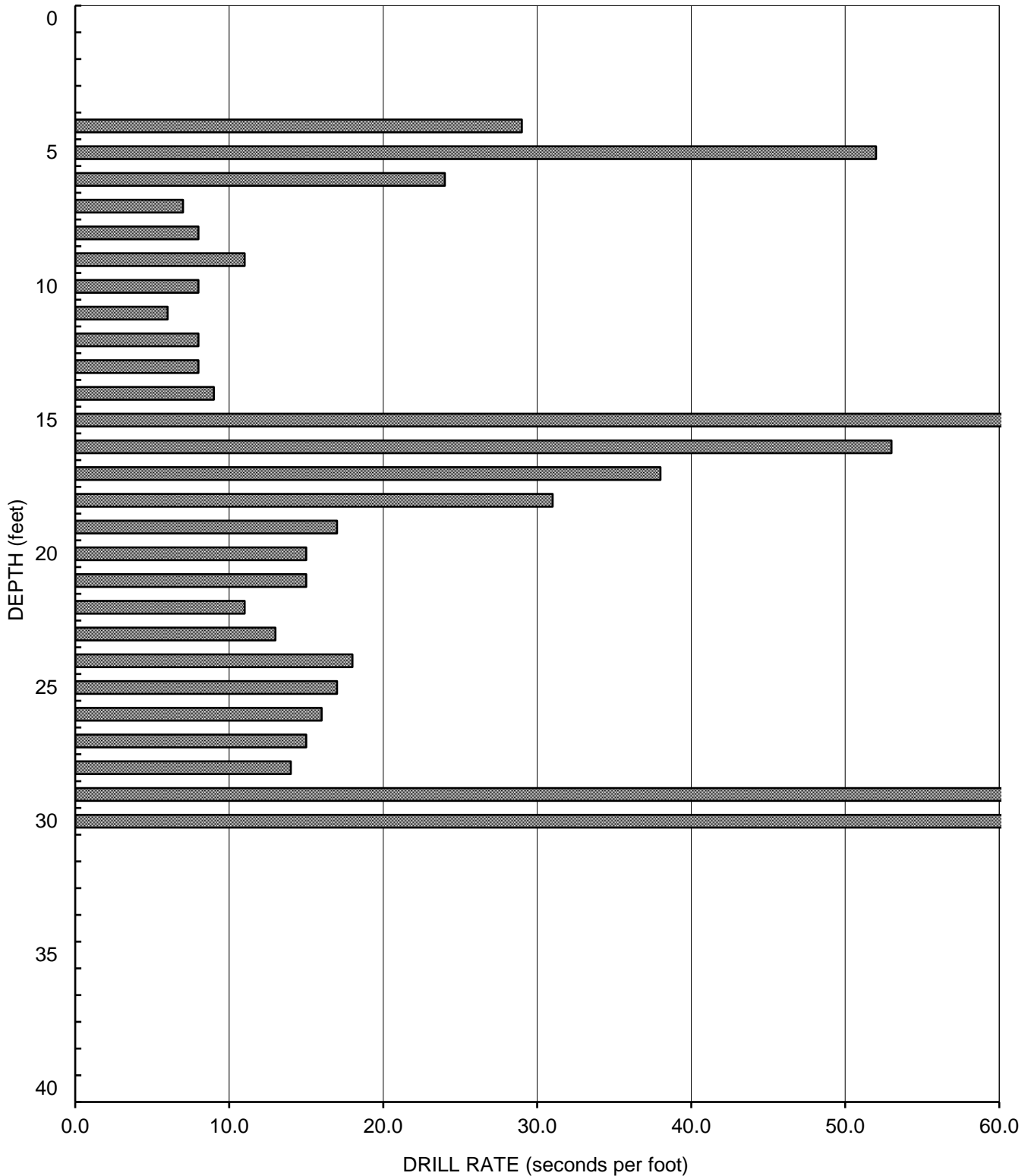
AIR TRACK BORING AT-7
Elevation - 831 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



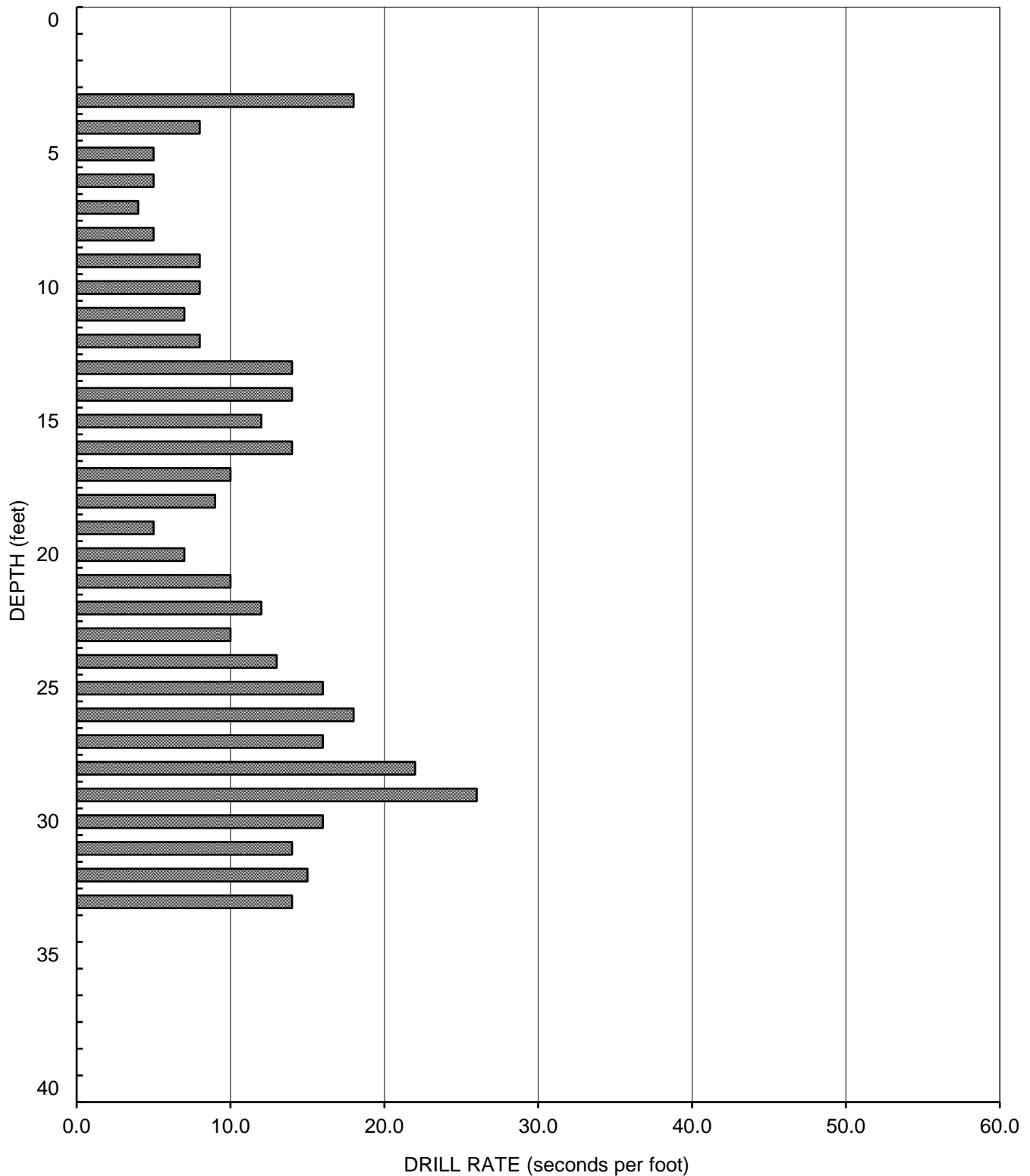
AIR TRACK BORING AT-8
Elevation - 828 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



AIR TRACK BORING AT-9
Elevation - 801 Feet (MSL)
Date 12-07-2018 - Equipment: ECM 370



AIR TRACK BORING AT-10
Elevation - 812 Feet (MSL)
Date 12-07-2018 - 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: maximum dry density and optimum moisture content; shear-strength; expansion index; water-soluble sulfate content; chloride ion content; resistance value; and grain size distribution. The results of our laboratory tests are presented on the following tables and Figures.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY
AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T5-1	Brown, silty, fine to coarse SAND	133.2	8.2
T12-2	Brown, silty, fine to coarse SAND	130.4	10.3

TABLE B-II
SUMMARY OF LABORATORY REMOLDED DIRECT SHEAR TEST RESULTS
ASTM D3080-98

*Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
T5-1	120.1	7.8	660	30
T12-2	118.9	8.5	350	42

*Samples remolded to approximately 90 percent relative compaction near optimum moisture content.

TABLE B-III
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D4829-95

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T12-1	8.1	15.2	118.3	8

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE CONTENT TEST RESULTS
CALIFORNIA TEST METHOD NO. 417

Sample No.	Water Soluble Sulfate %	Sulfate Exposure
T12-1	0.002	S0
T12-2	0.001	S0

TABLE B-V
SUMMARY OF LABORATORY CHLORIDE ION CONTENT TEST RESULTS
AASHTO T291

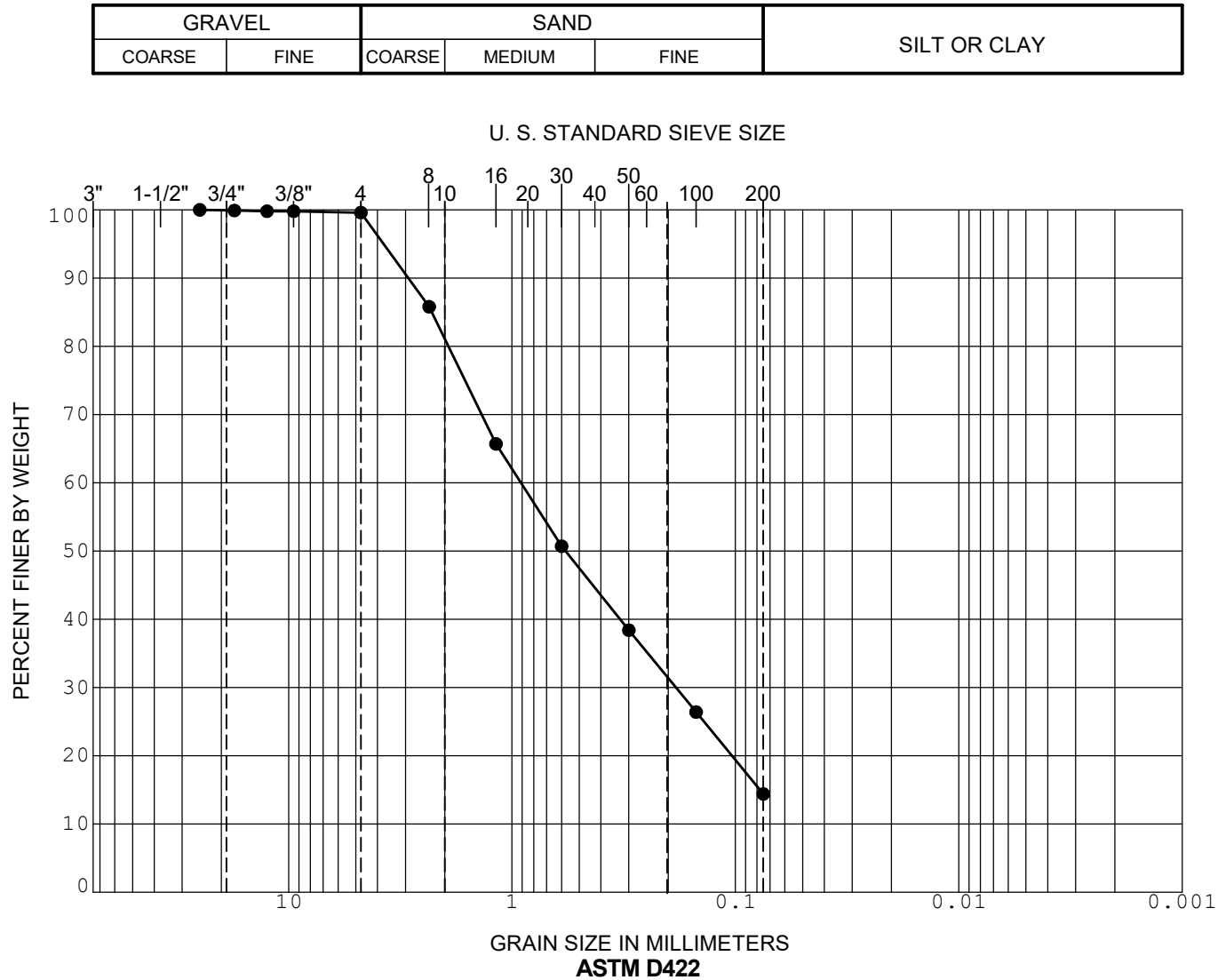
Sample No.	Chloride Ion Content %	PPM
T5-1	0.009	91
T12-2	0.009	92

TABLE B-VI
SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS

Sample No.	R-Value
T12-2	67

TABLE B-VII
SUMMARY OF LABORATORY SAND EQUIVALENT TEST RESULTS
ASTM 2419

Sample No.	San Equivalent
T5-1	24



-
-
- ▲

SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
T5-1	3.0	SM - Silty SAND				

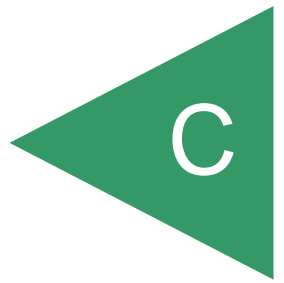
GRADATION CURVE

SUMMIT ESTATES

SAN DIEGO COUNTY, CALIFIORNIA

Figure B-1

APPENDIX



APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the *2016 City of County of San Diego Design Manual*. If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, provides general information regarding the existing 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.

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 property is underlain by granitic rock. Table C-2 presents the information from the USDA website for the subject property.

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

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group
Cieneba coarse sandy loam, 15 to 30 percent slopes, eroded	CIE2	82	D
Fallbrook sandy loam, 9 to 15 percent slopes, eroded	FaD2	16	C
Steep gullied land	StG	2	NA

In-Situ Testing

We performed 6 borehole infiltration tests at the locations presented on the Figure 2. Table C-3 presents the results of the saturated hydraulic conductivity testing.

**TABLE C-3
UNFACTORED, FIELD-SATURATED, INFILTRATION TEST RESULTS
USING THE SOILMOISTURE CORP AARDVARK PERMEAMETER**

Test No.	Depth (inches)	Surficial Soil or Geologic Unit	Field Infiltration Rate, I (in/hr)	Factored* Field Infiltration Rate, I (in/hr)
A-1	50	Kgr	0.007	0.0035
A-2	52	Kgr	0.17	0.085
A-3	61	Kgr	0.34	0.17
A-4	59	Kgr	0.007	0.0035
A-5	49	Kgr	0.053	0.03
A-6	51	Kgr	0.045	0.023

*Factor of Safety of 2.0 for feasibility determination.

Soil permeability values from in-situ tests can vary significantly from one location to another due to the non-homogeneous characteristics inherent to most soil. For this project and for storm water purposes, the test results presented herein should be considered approximate values.

STORM WATER MANAGEMENT CONCLUSIONS

Soil Types

Granitic Bedrock – Granitic Bedrock underlies the site. The Granitic Bedrock encountered during our investigation was weathered and excavates as silty, fine to coarse sand.

Groundwater Elevations

We did not encounter groundwater during our field investigation. Groundwater is expected to be at depths in excess of 10 feet below the bottom of basins.

Soil or Groundwater Contamination

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

Existing and Proposed Utilities

There are existing utilities that serve the existing residence. However, we expect these utilities to be abandoned during grading. Based on the current site plan, we do not expect new utilities will be present within 10 feet of the proposed BMP basins.

Existing and Planned Structures

Water should not be allowed to infiltrate in areas where it could affect the neighboring properties and existing adjacent structures, improvements and roadway. Based on the site plan, we do not expect existing or planned structures will be located within 10 feet of the proposed BMP basins. Water infiltration should not be allowed within a lateral distance of 10 feet from new or existing structures.

Slopes

A slope is planned adjacent to the easternmost basin. Other than side slopes constructed for the basins, no slopes are planned adjacent to the western basins.

Storm Water Management Devices

Liners and subdrains should be incorporated into the design and construction of the planned storm water devices. The liners should be impermeable (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent Polyvinyl Chloride, PVC) to prevent lateral water migration. The subdrains should be perforated within the liner area, installed at the base and above the liner, be at least 3 inches in diameter and consist of Schedule 40 PVC pipe. The subdrains outside of the liner should consist of solid pipe. The penetration of the liners at the subdrains should be properly waterproofed. The subdrains should be connected to a proper outlet. The devices should also be installed in accordance with the manufacturer's recommendations.

Storm Water Standard Worksheets

We have evaluated the proposed basins with respect to the infiltration restrictions contained in Table C.1-1 in Appendix C of the County of San Diego BMP Design Manual (DRAFT). Table C-4 below provides the information.

TABLE C-4
INFILTRATION RESTRICTIONS FOR BASIC INFILTRATION ANALYSIS
(TABLE C.1-1 OF APPENDIX C DRAFT)

Restriction Element		Is Element Applicable? (Yes/No)
Mandatory Considerations	BMP is within 100' of Contaminated Soils	No
	BMP is within 100' of Industrial Activities Lacking Source Control	No
	BMP is within 100' of Well/Groundwater Basin	No
	BMP is within 50' of Septic Tanks/Leach Fields	Yes
	BMP is within 10' of Structures/Tanks/Walls	No
	BMP is within 10' of Sewer Utilities	No
	BMP is within 10' of Seasonal High Groundwater	No
	BMP is within Hydric Soils	No
	BMP is within Highly Liquefiable Soils and has Connectivity to Structures	No
	BMP is within 1.5 Times the Height of Adjacent Steep Slopes ($\geq 25\%$)	Yes (East Basin) No (West and South Basins)
	County Staff has Assigned "Restricted" Infiltration Category	No
Optional Considerations	BMP is within Predominantly Type D Soil	Yes
	BMP is within 5' of Property Line	No
	BMP is within Fill Depths of $\geq 5'$ (Existing or Proposed)	Yes (East Basin) No (West and South Basins)
	BMP is within 10' of Underground Utilities	No
	BMP is within 250' of Ephemeral Stream	No
Result	Based on examination of the best available information, I have not identified any restrictions above.	
	Based on examination of the best available information, I have identified one or more restrictions above.	Restricted

Based on the information in Table C-4, BMP Basins should be considered "Restricted" due to the presence of Type D soils, adjacent proposed septic fields, and proposed compacted fill.

Using Section C.1-2 of the County Draft Guidelines, the basins should be designed for minimal retention. The average infiltration rate for each basin is provided in Table C-5. Due to the proposed fill, the eastern basin should be fully lined.

**TABLE C-5
FIELD-SATURATED, INFILTRATION TEST RESULTS**

Basin Location	Average Field Infiltration Rate, I (in/hr)	Average Factored* Field Infiltration Rate, I (in/hr)
West Basin (Infiltration Tests A-1 and A-2)	0.09	0.045
Southern Basin (Infiltration Tests A-3 and A-4)	0.17	0.09
Eastern Basin (Infiltration Tests A-5 and A-6)	0.05	0.025

*Factor of Safety of 2.0 for feasibility determination.

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. A separate worksheet has been prepared for the eastern basin where a full liner is recommended.

CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that partial infiltration is feasible for the western and southern basins. A “no infiltration” condition should be used for the eastern basin. Our evaluation included the soil and geologic conditions, settlement and volume change of the underlying soil, slope stability, utility considerations, groundwater mounding, structures and foundations, and estimated groundwater elevations.

Categorization of Infiltration Feasibility Condition		Form I-8	
Part 1 - Full Infiltration Feasibility Screening Criteria Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?			
Criteria	Screening Question	Yes	No
1	Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
<u>WESTERN AND SOUTHERN BASINS</u> The average results of the field infiltration tests for each basin are: West Basin: 0.09 in/hr (0.045 in/hr using a factor of 2.0 for screening purposes) South Basin: 0.17 in/hr (0.09 in/hr using a factor of 2.0 for screening purposes) The rates are less than 0.5 inches/hour. Therefore, full infiltration is not feasible.			
2	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
Provide basis: <u>WESTERN AND SOUTHERN BASINS</u> We do not believe slope stability, groundwater mounding, or impacts to existing utilities or improvements would occur if infiltration greater than 0.5 inches per hour was allowed considering the location of the proposed basins with respect to site soil and geologic conditions.			

Worksheet I-8 Page 2 of 4

Criteria	Screening Question	Yes	No
3	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
<u>WESTERN AND SOUTHERN BASINS</u> <p>Groundwater was not encountered in any of our trenches or borings. The groundwater elevation is assumed to be in excess of 10 feet below proposed basins grades. It is our opinion that there is not a significant increase in risk of groundwater contamination due to infiltration.</p>			
4	Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
<u>WESTERN AND SOUTHERN BASINS</u> <p>We do not expect infiltration will cause water balance issues such as seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters.</p>			
Part 1 Result*	<p>If all answers to rows 1 - 4 are “Yes” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration</p> <p>If any answer from row 1-4 is “No”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2</p>	No	

Worksheet I-8 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

WESTERN AND SOUTHERN BASINS

The average results of the field infiltration tests for each basin are:

West Basin: 0.09 in/hr (0.045 in/hr using a factor of 2.0 for screening purposes)

South Basin: 0.17 in/hr (0.09 in/hr using a factor of 2.0 for screening purposes)

The soil conditions allow for an appreciable rate.

6	Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
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WESTERN AND SOUTHERN BASINS

We do not believe slope stability, groundwater mounding, or impacts to existing utilities or improvements would occur if an appreciable quantity of infiltration was allowed considering the location of the proposed basins with respect to site soil and geologic conditions.

Worksheet I-8 Page 4 of 4

Criteria	Screening Question	Yes	No
7	Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
<u>WESTERN AND SOUTHERN BASINS</u> Groundwater was not encountered in any of our trenches or borings. The groundwater elevation is assumed to be in excess of 10 feet below proposed basins grades. It is our opinion infiltration should not pose a significant risk for groundwater related concerns.			
8	Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
<u>WESTERN AND SOUTHERN BASINS</u> We did not provide a study regarding water rights. However, for a partial infiltration condition, violation of downstream water rights is not anticipated.			
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration . If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration .		Partial Infiltration Feasible

Categorization of Infiltration Feasibility Condition		Form I-8	
Part 1 - Full Infiltration Feasibility Screening Criteria Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?			
Criteria	Screening Question	Yes	No
1	Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
<u>EASTERN BASIN</u> The average results of the field infiltration tests for the basin are: Eastern Basin: 0.05 in/hr (0.025 in/hr using a factor of 2.0 for screening purposes) The rates are less than 0.5 inches/hour. Therefore, full infiltration is not feasible.			
2	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: <u>EASTERN BASIN</u> Proposed grading will create a fill slope along the southern side of the basin. Additionally, a portion of the basin is underlain by compacted fill. Infiltration into the fill could cause settlement and daylight seepage to the slope..			

Worksheet I-8 Page 2 of 4

Criteria	Screening Question	Yes	No
3	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
<u>EASTERN BASIN</u> Groundwater was not encountered in any of our trenches or borings. The groundwater elevation is assumed to be in excess of 10 feet below proposed basins grades. It is our opinion that there is not a significant increase in risk of groundwater contamination due to infiltration.			
4	Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
<u>EASTERN BASIN</u> We do not expect infiltration will cause water balance issues such as seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters.			
Part 1 Result*	If all answers to rows 1 - 4 are “ Yes ” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration If any answer from row 1-4 is “ No ”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2	No	

Worksheet I-8 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

EASTERN BASIN

The average results of the field infiltration tests for the basin are:

Eastern Basin: 0.05 in/hr (0.025 in/hr using a factor of 2.0 for screening purposes)

The factored rate is less than 0.05 inches/hour. Therefore, partial infiltration is not feasible.

6	Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
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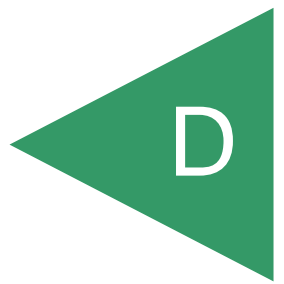
EASTERN BASIN

Proposed grading will create a fill slope along the southern side of the basin. Additionally, a portion of the basin is underlain by compacted fill. Infiltration into the fill could cause settlement and daylight seepage to the slope..

Worksheet I-8 Page 4 of 4

Worksheet I-8 Page 4 of 4			
Criteria	Screening Question	Yes	No
7	Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
<u>EASTERN BASIN</u> Groundwater was not encountered in any of our trenches or borings. The groundwater elevation is assumed to be in excess of 10 feet below proposed basins grades. It is our opinion infiltration should not pose a significant risk for groundwater related concerns.			
8	Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
<u>EASTERN BASIN</u> We did not provide a study regarding water rights. However, violation of downstream water rights is not anticipated.			
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration . If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration .		No Infiltration

APPENDIX



APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

SUMMIT ESTATES
SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2279-42-01

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 $\frac{3}{4}$ 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 $\frac{3}{4}$ 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.

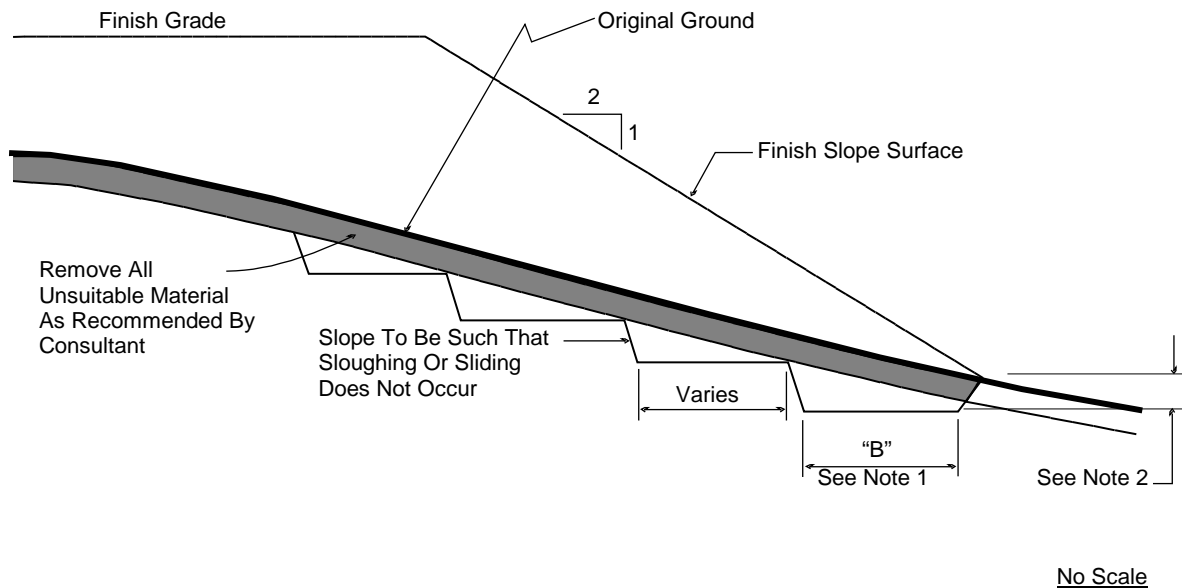
- 3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker 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

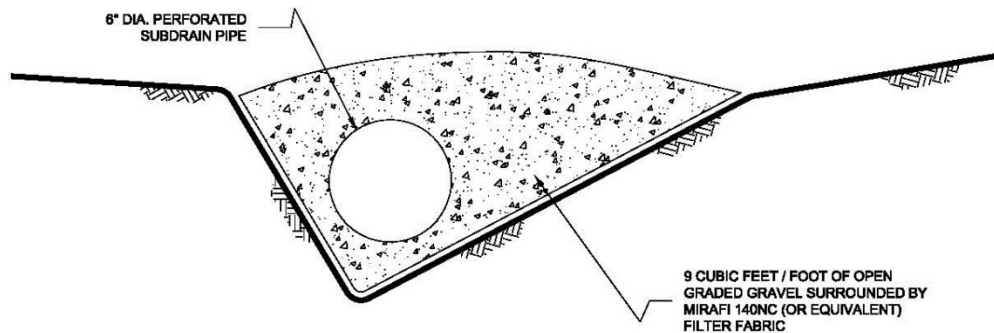
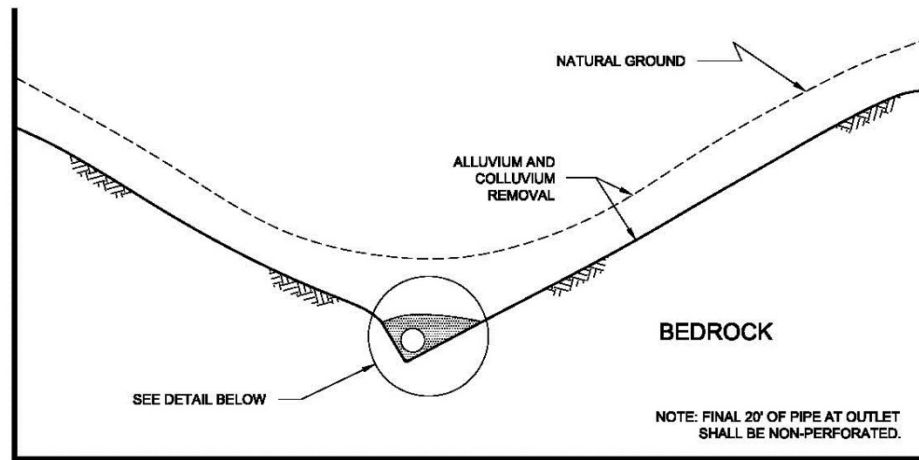
variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

- 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



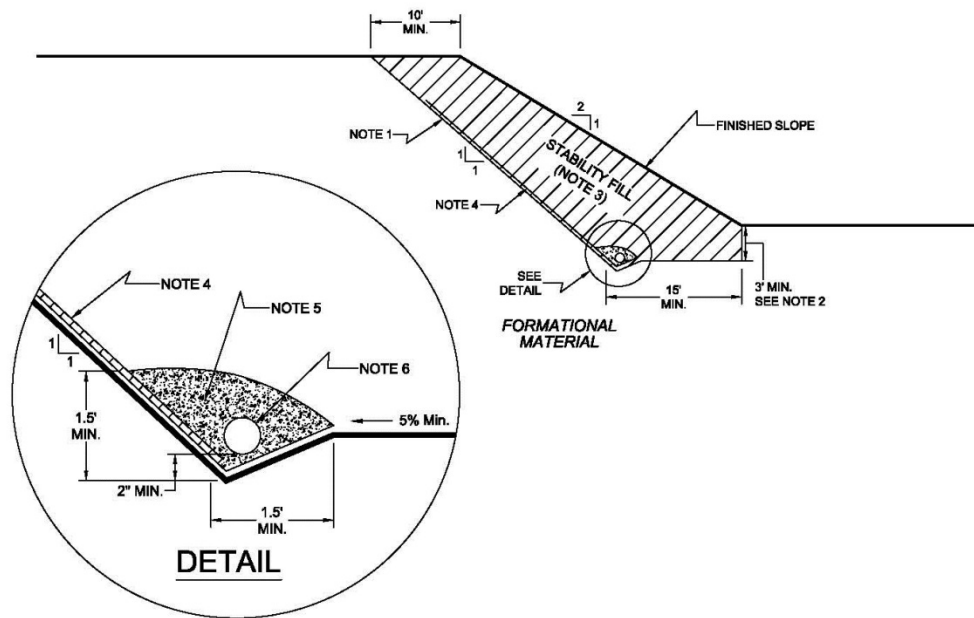
NOTES:

- 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 larger) 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.

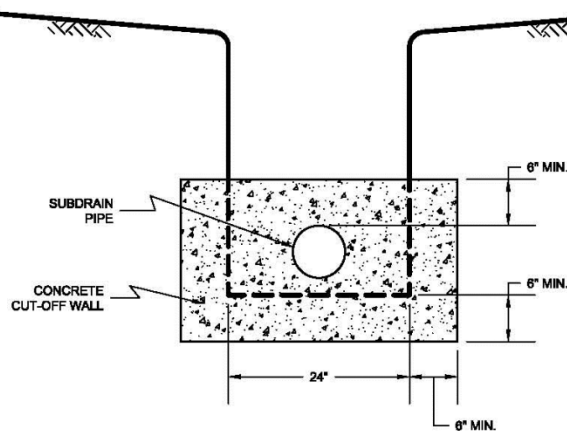
NO SCALE

- 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.

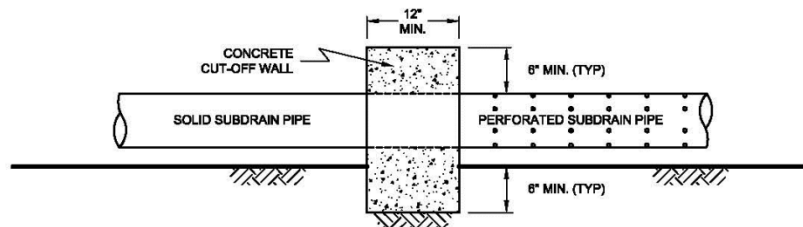
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW

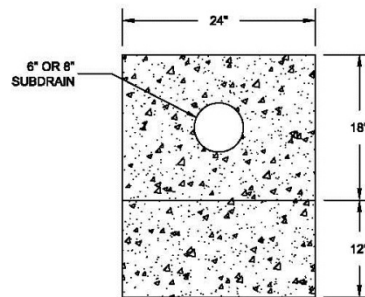


NO SCALE

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

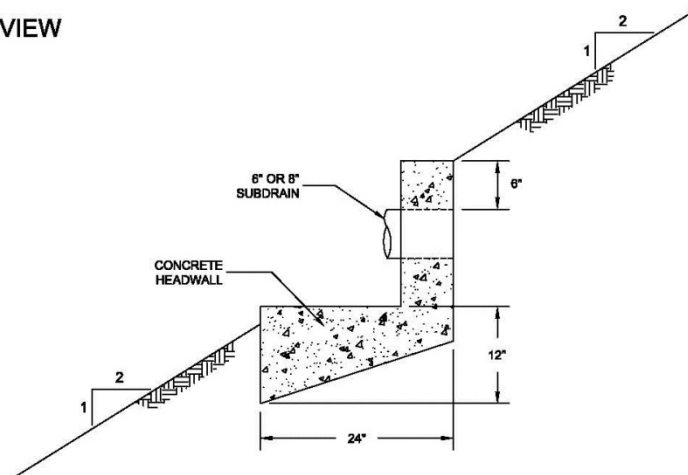
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



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

NO SCALE

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

8. OBSERVATION AND TESTING

- 8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.
- 8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.
- 8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.
- 8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.
- 8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.
- 8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

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

- 8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth)*.
- 8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.
- 8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

- 9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.
- 9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

- 10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.
- 10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

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- Tan, S.S. and Kennedy, M.P. (1999), Geologic Map of the Escondido 7.5' Quadrangle, San Diego County, California;
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- USGS (2016), *Quaternary Fault and Fold Database of the United States*: U.S. Geological Survey website, <http://earthquakes.usgs.gov/hazards/qfaults>, accessed January 7, 2019.