GEOTECHNICAL INVESTIGATION

PARKVIEW TOWNHOMES
550 WEST EL NORTE PARKWAY
ESCONDIDO, CALIFORNIA

PREPARED FOR:

PARKWAY TOWNHOME, LLC
SAN DIEGO, CALIFORNIA

PREPARED BY:

GEOCON INCORPORATED

GEOTECHNICAL ENVIRONMENTAL MATERIALS

OCTOBER 3, 2024
PROJECT NO. G3180-32-03

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G3180-32-03
October 3, 2024

Parkway Townhome, LLC
9815 Mira Mesa Boulevard
San Diego, California 92131

Attention: Mr. Kerry Garza

Subject: GEOTECHNICAL INVESTIGATION
PARKVIEW TOWNHOMES
550 WEST EL NORTE PARKWAY
ESCONDIDO, CALIFORNIA

Dear Mr. Garza:

In accordance with your request, and our Proposal No. SD-24-1630-P-GT dated August 26, 2024, we have performed a geotechnical investigation on the subject property located in Escondido, California. The accompanying report presents our findings and conclusions and recommendations relative to the geotechnical aspects of developing the project as presently proposed.

Based on the results of this study, it is our opinion that the project can be developed as planned, provided the recommendations in this report are followed.

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

Very truly yours,

GEOCON INCORPORATED

Dylan Thomas
CEG 2784

Trevor E. Myers
RCE 63773

David B. Evans
CEG 1860

PROFESSIONAL GEOLOGIST • DYLAN THOMAS No. 2784 CERTIFIED ENGINEERING GEOLOGIST • STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER ★ TREVOR E. MYERS No. RCE63773 CIVIL ★ STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST ★ DAVID B. EVANS NO. 1860 CERTIFIED ENGINEERING GEOLOGIST ★ STATE OF CALIFORNIA

DT:TEM:DBE:am

(e-mail) Addressee

6960 Flanders Drive ■ San Diego, California 92121-2974 ■ Telephone 858.558.6900 ■ Fax 858.558.6159

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for a proposed residential development located at 550 West El Norte Parkway in the City of Escondido, California (see *Vicinity Map*, Figure 1). The purpose of this study was to evaluate the soil and geologic conditions at the site and provide specific geotechnical recommendations pertaining to the development of the property as proposed based on the conditions encountered.

The scope of our study consisted of the following:

- Reviewing satellite imagery and readily available published and unpublished geologic literature.
- Reviewing the tentative map prepared by Touchstone Development dated April 2024, which was utilized as our base map to present the subsurface information.
- Performing eleven exploratory trenches using a rubber tire backhoe to evaluate the general extent and condition of the surficial deposits (see Appendix A).
- Advancing ten hydraulic rotary percussion (air-track) borings in the primary cut areas to aid in determining the rippability characteristics of the rock (see Appendix A).
- Performing laboratory tests on selected soil samples collected to evaluate their physical and characteristics for engineering analysis (see Appendix B).
- Performing two infiltration tests within the proposed biofiltration basin area and providing storm water BMP design information (See Appendix C).
- Preparing this report presenting our exploratory information and our conclusions and recommendations regarding the geotechnical aspects of developing the site as presently proposed. The approximate locations of the subsurface excavations are shown on the *Geologic Map*, Figure 2.

2. SITE AND PROJECT DESCRIPTION

The proposed Parkview Townhome development is located at 550 West El Norte Parkway in the City of Escondido, California. The site is located on the north side of West El Norte Parkway and is bounded by Rod Mcleod Park to the north, open space to the west, and residential units to the east. The site is currently occupied by a single-family residential structure located on the northern portion of the site with barns, retaining walls, asphalt driveway, and landscaping. Granitic rock outcrops are located on the western and northern portion of the site. Topographically, the site consists of gentle to moderately sloping terrain that drains towards the south. Native grasses and a few mature trees are present throughout the site.

It is our understanding that the proposed project consists of demolishing the existing structures and improvements and developing the site to support 70, three story condominium units with associated pool and spa, roadways, parking, retaining walls, utilities and landscaping. A biofiltration basin will be located on the southern portion of the site. Based on the grading plan, we understand an approximate 28-foot-high, 1.5:1 (Horizontal: Vertical) cut slope is proposed for the northern portion of the development and an approximate 33-foot-high, 2:1 (H:V) fill slope is proposed for the southern portion of the site. The maximum cut and fill depths, excluding remedial grading, are approximately 22 feet and 20 feet, respectively.

The site location, descriptions, and proposed development discussed above are based on information provided to us and our understanding of the proposed project. If project details vary significantly from those described, Geocon Incorporated should be consulted to provide additional recommendations and/or analysis.

3. SOIL AND GEOLOGIC CONDITIONS

The geologic units encountered on the property include two surficial soils including undocumented fill and topsoil, and one formation consisting of Cretaceous-age granitic rock. Each of the units is described below in order of increasing age.

3.1 Undocumented Fill (Unmapped)

We encountered undocumented fill in Trenches T-4 through T-7 and Trenches T-9 through T-11. The fill ranged in thickness from approximately 1.5 to 7 feet thick and consists of loose to medium dense, silty sand with some gravel and cobble-size granitic rock clasts and construction debris (asphalt, trash, concrete, wood debris). This material is not considered suitable in its current condition for the support of foundations or structural fill and remedial grading will be required. The undocumented fill can be reused provided it is generally free of roots and debris.

3.2 Topsoil (Unmapped)

We encountered topsoil in our trenches at the ground surface and below the undocumented fill. The topsoil varies in thickness from approximately 1.5 to 4 feet thick. In general, this material consists of loose to medium dense, dry to moist, silty to clayey sand with some cobble and gravel. The topsoil is not considered suitable in its current condition for the support of foundations or structural fill and remedial grading will be required. The topsoil can be reused provided it is generally free of roots and debris.

3.3 Granitic Rock (Kgr)

Granitic rock associated with the Southern California Batholith is present beneath the surficial deposits and is exposed at the ground surface on the western and northern portions of the site. The granitic rock generally exhibits variable weathering and consists of completely weathered to fresh rock and will require blasting, rock breaking, or very heavy ripping to excavate. The granitic rock observed in the exploratory trenches generally consisted of completely to moderately weathered, weak to very strong rock that excavated as a fine to coarse silty sand with angular rock fragments. Outcrops of fresh, extremely strong granitic rock, along with corestones, were observed at the surface (see image below), primarily in the western and northwestern portion of the site. It is likely that these areas will require blasting to excavate.

Corestones Exposed at Grade within the Northwestern Portion of the Site.

Air-track borings were performed to further evaluate rippability. The excavations planned within the rock areas will generate oversize materials (rocks greater than 12 inches in dimension). The rippability characteristics of the granitic rock are discussed in the *Rippability and Rock Considerations* section below. Granitic units generally exhibit adequate bearing and slope stability characteristics and cut slopes should be stable to the proposed heights if free of adversely oriented joints or fractures.

4. RIPPABILITY AND ROCK CONSIDERATIONS

We performed a rock rippability evaluation consisting of drilling 10 air-track borings in the proposed cut areas of the site. We performed the study with an Ingersoll-Rand ECM 590 equipped with a 4-inch-diameter bit. Drill penetration rates were measured 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 equate rock rippability to drill penetration rate is that a penetration rate of approximately 0 to 20 seconds per foot (spf) generally indicates rippable material, 20 to 30 spf indicates marginally to non-rippable material, and greater than 30 spf indicates non-rippable rock. These general guidelines are typically based on drill rates using a rotary percussion drill rig similar to an Ingersoll Rand ECM 360 with a 3½-inch drill bit. The penetration rates (recorded in seconds per foot) for the air track boring are presented in Appendix A, Figures A-12 through A-21.

The rippability designations discussed above are based on the use of a D9 or equivalent bulldozer equipped with a single shank ripper. Rippable materials can be excavated with moderate to heavy effort. Marginally rippable includes very heavy ripping and isolated zones of probable blasting. Non-rippable materials will require blasting to excavate the rock.

The estimated thickness of rippable material based on a literal interpretation of penetration rate for each air track boring is presented on Figure 2. 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. We used a threshold of 20 spf to indicate the thickness of rippable material next to each boring on the geologic map.

Based on an air track penetration rate of 20 spf, it is expected that the rippability characteristics will vary. The air-track borings indicate that, where fresh rock is not exposed near the surface (e.g., boulders), the granitic rock is characterized by a rippable weathered mantle varying from approximately 4 to 32-feet-thick. Excavations greater than these depths will encounter difficult ripping conditions and may require blasting techniques and can be expected to generate oversized rock (rocks ≥12 inches in dimension), which will necessitate typical hard rock handling, sizing, and placement procedures during grading operations. Proposed cuts in the weathered mantle may also generate oversized fragments.

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. Roadway/utility corridors and lot undercutting criteria should also be considered when calculating the volume of hard rock. In addition, a volumetric evaluation should be performed to determine if there are available fill placement areas considering the rock hold down criteria.

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.

5. GROUNDWATER

A static, near-surface groundwater table was not observed during the field investigation. Groundwater is not expected to significantly affect the design and construction of the subject site.

Although not encountered during the field investigation, groundwater seepage along the granitic rock contact is relatively common. The seepage is typically transmitted along fractures within the granitic rock and is associated with the runoff of irrigation and/or rainwater. The seepage conditions can typically be mitigated by the construction of shallow surface and subsurface drainage systems.

6. GEOLOGIC HAZARDS

6.1 Ground Rupture

United States Geological Survey maps (2016) indicate that there are no mapped Quaternary faults crossing or trending toward the property. In addition, the site is not located within a currently established Alquist-Priolo Earthquake Fault Zone.

The nearest known active-fault zones are the Rose Canyon and Newport Inglewood Faults, located approximately 16 miles west of the subject site. The risk associated with ground rupture hazard is low.

6.2 Seismicity

The San Diego County and Southern California region is seismically active. Considerations important in seismic design include the frequency and duration of motion and the soil conditions underlying the site. Seismic design of structures should be performed in accordance with the California Building Code (CBC) guidelines currently adopted by the local agency. The risk associated with strong ground shaking due to earthquake at the site is no greater than that for the region.

6.3 Landslides

No landslides were encountered during previous site investigations or grading activities, and none are known to exist on the property or at a location that would impact the proposed development.

6.4 Soil Liquefaction Potential

Soil liquefaction occurs within relatively loose, cohesionless sands located below the water table that are subjected to ground accelerations from earthquakes. Due to the relatively dense nature of the granitic rock beneath the project area, and lack of near surface groundwater, the potential for liquefaction is considered very low.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 The site is underlain by surficial deposits (fill and topsoil) that are unsuitable in their present condition and will require remedial grading where improvements are planned. The estimated thickness of surficial soils requiring remedial grading is shown on the *Geologic Map (Figure 2)*. The actual extent of unsuitable soil removal will be determined in the field by the geotechnical engineer and/or engineering geologist.

7.1.3 The proposed buildings can be supported on conventional continuous and isolated spread shallow foundations founded entirely on compacted fill or granitic rock. Consideration should be given to undercutting the granitic rock in areas of proposed improvements for ease of future excavations.

7.1.4 The presence of hard rock within proposed cut areas will require special consideration during site development. It is anticipated that the majority of the proposed excavations within the granitic rock will encounter moderate to heavy ripping with conventional heavy-duty equipment and/or blasting. In addition, heavy ripping and blasting will generate oversize materials that will require crushing, special handling and fill placement procedures. Oversize materials should be placed in accordance with Figure 3 and Appendix D of this report.

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

7.1.6 With the exception of possible strong seismic shaking, no geologic hazards were observed or are known to exist on the site that would adversely affect the proposed project. No special seismic design considerations, other than those recommended herein, are required.

7.2 Excavation and Soil Characteristics

7.2.1 The soils encountered during field investigation is considered to be “non-expansive” (expansion index [EI] less than or equal to 20) and “expansive (expansion index [EI] greater than 20) as defined by 2022 California Building Code (CBC) Section 1803.5.3. Table 7.2

presents soil classifications based on the expansion index. The laboratory expansion index test results are presented in Appendix B.

TABLE 7.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM 4829 Expansion Classification	2022 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

7.2.2 The on-site soil is considered to have a "very low" to "low" expansion potential (expansion index [EI] of 50 or less). Recommendations presented herein assume that the site will be graded such that soil with an EI of 50 or less will be present to a minimum depth of three feet below finish grade. If soil with an EI greater than 50 is exposed near finish grade, modifications to the foundation and slab-on-grade recommendations presented herein may be required.

7.2.3 Excavations within the granitic rock materials will be difficult; consequently, blasting, rock breaking, very heavy ripping, etc., may be necessary.

7.2.4 Temporary slopes should be made in conformance with OSHA requirements.

7.2.5 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 Corrosion

7.3.1 The laboratory test results indicate that the near-surface on-site materials at the locations tested possess *Not Applicable* sulfate severity and *S0* exposure to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19 Chapter 19. Table 7.3 presents a summary of concrete requirements set forth by 2022 CBC Section 1904 and ACI 318. ACI guidelines should be followed when determining the type of concrete to be used. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 7.3
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class		Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
S0		$SO_4<0.10$	No Type Restriction	n/a	2,500
S1		$0.10\leq SO_4<0.20$	II	0.50	4,000
S2		$0.20\leq SO_4\leq 2.00$	V	0.45	4,500
S3	Option 1	$SO_4>2.00$	V+Pozzolan or Slag	0.45	4,500
	Option 2		V	0.40	5,000

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

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

7.4 Seismic Design Criteria

7.4.1 Table 7.4.1 summarizes site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *U.S. Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client.

TABLE 7.4.1
2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.914g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.334g	Figure 1613.2.1(3)
Site Coefficient, F_A	1.2	Table 1613.2.3(1)
Site Coefficient, F_V	1.5	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.097g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.501g	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.731g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.334g	Section 1613.2.4 (Eqn 16-23)

7.4.2 Table 7.4.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 7.4.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.396g	Figure 22-9
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.457g	Section 11.8.3 (Eqn 11.8-1)

7.4.3 Conformance to the criteria in Tables 7.4.1 and 7.4.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.4.4 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of II and resulting in a Seismic Design Category D. Table 7.4.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 7.4.3
ASCE 7-16 RISK CATEGORIES

Risk Category	Building Use	Examples
I	Low risk to Human Life at Failure	Barn, Storage Shelter
II	Nominal Risk to Human Life at Failure (Buildings Not Designated as I, III or IV)	Residential, Commercial and Industrial Buildings
III	Substantial Risk to Human Life at Failure	Theaters, Lecture Halls, Dining Halls, Schools, Prisons, Small Healthcare Facilities, Infrastructure Plants, Storage for Explosives/Toxins
IV	Essential Facilities	Hazardous Material Facilities, Hospitals, Fire and Rescue, Emergency Shelters, Police Stations, Power Stations, Aviation Control Facilities, National Defense, Water Storage

7.5 Grading

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

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

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

7.5.4 All potentially compressible surficial soils within areas of planned grading should be removed to firm natural ground and properly moisture conditioned and compacted prior to placing additional fill and/or structural loads. The actual extent of unsuitable soil removals should be determined in the field by the soil engineer and/or engineering geologist. Overly wet, surficial materials will require drying and/or mixing with drier soils to facilitate proper compaction.

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

7.5.6 To reduce the potential for differential settlement, and to facilitate the excavation of footings and utility trenches it is recommended that building pads possessing a cut-fill transition be undercut at least 3 feet and replaced with properly compacted "very low" to "low" expansive fill soils. Where the thickness of the fill below the building pad exceeds 15 feet, the depth of the undercut should be increased to one-fifth of the maximum fill thickness. The base of the undercuts should be sloped towards the fill side of the transition. Consideration should be given to undercutting the bedrock portion of cut-fill transitions on any sheet graded pads 5 feet to accommodate future fine grading.

7.5.7 Cut pads exposing granitic rock should be undercut at least 3 feet below finish grade and replaced with properly compacted "very low" to "low" expansive soil. The base of the undercuts should be sloped towards the front of the lots.

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

7.5.9 Where practical, the upper 3 feet of the building pads should be comprised of soil with a "very low" to "low" expansion potential. The more highly expansive fill soils should be placed in the deeper fill areas and properly compacted, if encountered.

7.5.10 Consideration should be given to using the surficial deposits as "capping material" for building pads and roadways, as discussed hereafter.

7.5.11 "Capping material" refers to select material placed within the upper portion of building pads and streets. The capping material should consist of "soil" fill with a maximum particle dimension of 6 inches, a minimum of 40 percent soil passing the ¾-inch sieve.

7.5.12 Soil generated during remedial grading (i.e., removal of surficial deposits) should be stockpiled and used to cap the site. Oversize rock (greater than 6 or 12 inches depending on the zone) will need to be removed with skeleton buckets or other means to meet the rock hold down criteria shown on the *Rock Placement Detail*, Figure 3.

7.5.13 Structural fills placed and compacted at the site should consist of material that can be classified into four zones as discussed below. A rock placement detail for the fill zones is presented on Figure 3:

Zone A: Material placed within 3 feet of finish grade, 10 feet in streets and 5 feet for sheet graded pads should consist of "soil" fill with a maximum particle size of 6 inches and a minimum of 40 percent of the soil passing the ¾-inch sieve.

Zone B: Fill material should consist of "soil" fill with a maximum particle dimension of 1-foot and a minimum of 40 percent of the soil passing the ¾-inch sieve.

Zone C: Material placed at least 10 feet below finish grade (below *Zone A* and *B*) may consist of "rock" fill or "soil/rock" fill (as defined in Appendix C). Rock should generally consist of 2-foot minus material with occasional rock up to 4-foot in maximum dimension. Alternatively, "soil" fill may be placed in Zone C containing rock with a maximum dimension of 2 feet. Rocks up to 4 feet in maximum dimension can be individually placed in a properly compacted soil matrix. A graded blanket or filter fabric may be required where "rock fills" transition to "soil" fills.

Zone D: Within the outer 15 feet of slopes the fill should consist of "soil" fill with rock up to 12-inches in maximum dimension and a minimum of 40 percent of the soil passing the ¾-inch sieve.

7.5.14 *Rock* fills, if any, may require subdrains during construction. The need for subdrains will be evaluated during grading. The subdrains, if required, shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water. Additionally, the subdrain may extend laterally along portions of the perimeter of the fill.

7.5.15 Placement of fill materials classified as *soil-rock* embankments should consist of spreading and compacting the materials with a bulldozer in 2-foot-thick lifts, or less. Where the matrix consists of finer grained material (greater than 30 percent passing the 200 sieve), lift thicknesses should be 12-inches, or less. During placement of each lift, the fill should be uniformly wheel-rolled with loaded rock trucks. Prior to compacting, the soil should be properly moisture conditioned during spreading. Selective in-place density tests should be performed within the soil matrix to evaluate whether or not the minimum relative compaction requirements are being achieved.

7.5.16 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.

7.5.17 Import fill (if necessary) should consist of granular materials with a "very low" to "low" expansion potential (EI of 50 or less) free of deleterious material or stones larger than 3 inches and should be compacted as recommended above. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

7.6 Slope Stability

7.6.1 Slope stability analyses were performed considering a 35-foot high, 2:1 (horizontal:vertical) fill slope utilizing average drained direct shear strength parameters obtained from our laboratory testing and our experience with similar soil conditions. These analyses indicate that any proposed fill slopes constructed of on-site materials should have calculated factors of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions. Slope stability calculations for deep-seated and surficial fill slope stability are presented on Figure 4.

7.6.2 Fill slopes should be overbuilt at least 3 feet horizontally, and cut back to the design finish grade. As an alternative, slopes should be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked with a D-8 dozer or similar equipment, as the slope is constructed, such that the fill soils are uniformly compacted to at least 90 percent relative compaction to the face of the finished slope.

7.6.3. Cut slopes in rock materials, if any, do not lend themselves to conventional slope stability analyses. Based on experience with similar rock conditions, cut slopes up to approximately 30 feet should possess a factor of safety of at least 1.5 with respect to slope instability, if free of adversely oriented joints or fractures. The cut slope excavations should be observed during grading by an engineering geologist to verify that soil and geologic conditions do not differ significantly from those anticipated. In the event that adverse conditions are observed, stabilization recommendations can be provided. Generalized slope stability calculations for deep-seated and surficial cut slope stability are presented on Figure 5.

7.6.4 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular "soil" fill to reduce the potential for surficial sloughing. In general, soils with an Expansion Index of less than 90 or at least 35 percent sand size particles should be acceptable as "granular" fill. Soils of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength.

7.6.5 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.

7.7 Foundation and Concrete Slabs-On-Grade Recommendations

7.7.1 Proposed structures supported on compacted fill over granitic rock may be designed using conventional shallow foundations.

7.7.2 The foundation recommendations herein are for proposed one- to three-story residential structures. The foundation recommendations have been separated into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 7.7.1.

TABLE 7.7.1
FOUNDATION CATEGORY CRITERIA

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

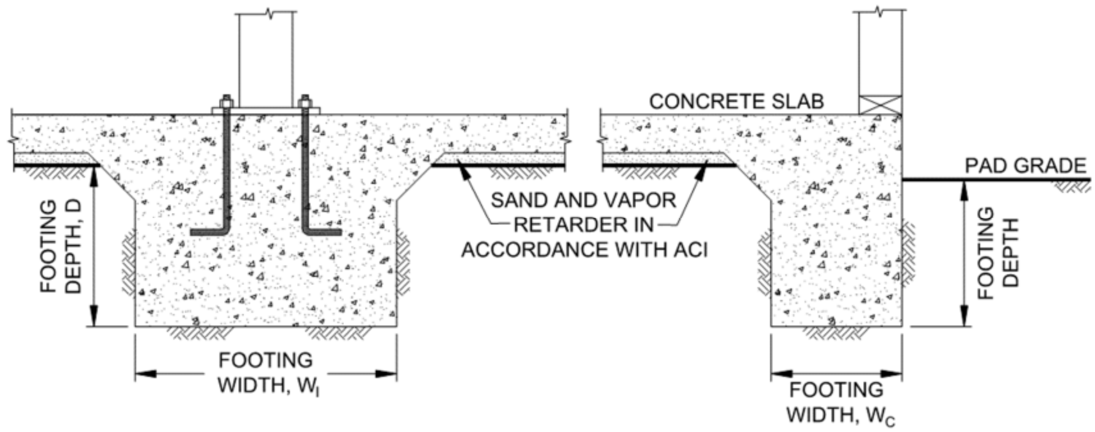
7.7.3 We will provide final foundation categories for each building or lot after finish pad grades have been achieved, the underlying fill-bedrock geometry is evaluated and we perform laboratory testing of the subgrade soil.

7.7.4 Table 7.7.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 7.7.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

<table>
<tr><th>Foundation Category</th><th>Minimum Footing Embedment Depth, D (inches)</th><th>Minimum Continuous Footing Reinforcement</th><th>Minimum Footing Width (Inches)</th></tr>
<tr><td>I</td><td>12</td><td>Two No. 4 bars, one top and one bottom</td><td rowspan="3">12 – Continuous, W_C
24 – Isolated, W_I</td></tr>
<tr><td>II</td><td>18</td><td>Four No. 4 bars, two top and two bottom</td></tr>
<tr><td>III</td><td>24</td><td>Four No. 5 bars, two top and two bottom</td></tr>
</table>

7.7.5 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. 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 (unless designed with a post-tensioned foundation system as discussed herein).



Wall/Column Footing Dimension Detail

7.7.6 The proposed structures can be supported on a shallow foundation system founded in the compacted fill/formational materials. Table 7.7.3 provides a summary of the foundation design recommendations.

TABLE 7.7.3
SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Allowable Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
Estimated Total Settlement*	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

7.7.7 The bearing capacity values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

7.7.8 The concrete slab-on-grades should be a designed in accordance with Table 7.7.4.

TABLE 7.7.4
CONVENTIONAL SLAB-ON-GRADE RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Concrete Slab Thickness (inches)	Interior Slab Reinforcement	Typical Slab Underlayment
I	4	6 x 6 - 10/10 welded wire mesh at slab mid-point	3 to 4 Inches of Sand/Gravel/Base
II	4	No. 3 bars at 24 inches on center, both directions	
III	5	No. 3 bars at 18 inches on center, both directions	

7.7.9 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). The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

7.7.10 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. It is common to see 3 inches and 4 inches of sand below the concrete slab-on-grade for 5-inch and 4-inch thick slabs, respectively, in the southern California area. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

7.7.11 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The post-tensioned systems (foundation dimensions and embedment depths, slab thickness and steel placement) should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5 as required by the 2022 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented in Table 7.7.5 for the particular

Foundation Category designated. The parameters presented in Table 7.7.5 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 7.7.5
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) DC10.5 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

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

7.7.13 If the structural engineer proposes a post-tensioned foundation design method other than the 2022 CBC (PTI, DC 10.5):

- The deflection criteria presented in Table 7.7.5 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.

7.7.14 Foundation systems for the lots that possess a foundation Category I and a “very low” expansion potential (expansion index of 20 or less) can be designed using the method described in Section 1808 of the 2022 CBC. If post-tensioned foundations are planned, an alternative, commonly accepted design method (other than PTI) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of 1 inch. Geocon Incorporated should be contacted to review the plans and provide additional information, if necessary.

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

7.7.16 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift from tensioning, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

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

7.7.18 Isolated footings outside of the slab area, if present, should have the minimum embedment depth and width recommended for conventional foundations for a particular Foundation Category. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams in both directions. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

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

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

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

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

- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. A post-tensioned slab and foundation system or mat foundation system can be used to reduce the potential for distress in the structures associated with strain softening and lateral fill extension. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

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

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

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

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

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

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

7.7.25 We should observe the foundation excavations prior to the placement of reinforcing steel to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

7.8 Concrete Flatwork

7.8.1 The following recommendations apply to exterior flatwork where near surface soils are low to medium expansive (EI less than 90). Exterior slabs not subjected to vehicular traffic should be a minimum of 4 inches thick and reinforced with 6 x 6-6/6 welded wire mesh or No. 3 steel reinforcing bars at 18 inches on center both directions. The reinforcement should be placed in the middle of the slab. Proper positioning is critical to future performance of the slab. The contractor should take extra measures to provide proper 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.

7.8.2 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. A 4-inch-thick slab should have a maximum joint spacing of 10 feet. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented above prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be checked prior to placing concrete.

7.8.3 Even with the incorporation of the recommendations within this report, the exterior concrete flatwork has a likelihood of experiencing some settlement due to potentially compressible and liquefiable soil beneath grade; therefore, the welded wire mesh should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

7.8.4 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. Periotic maintenance such as slab replacement and/or grinding of elevated slab margins may be necessary due to the highly expansive soils. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.9 Retaining Walls

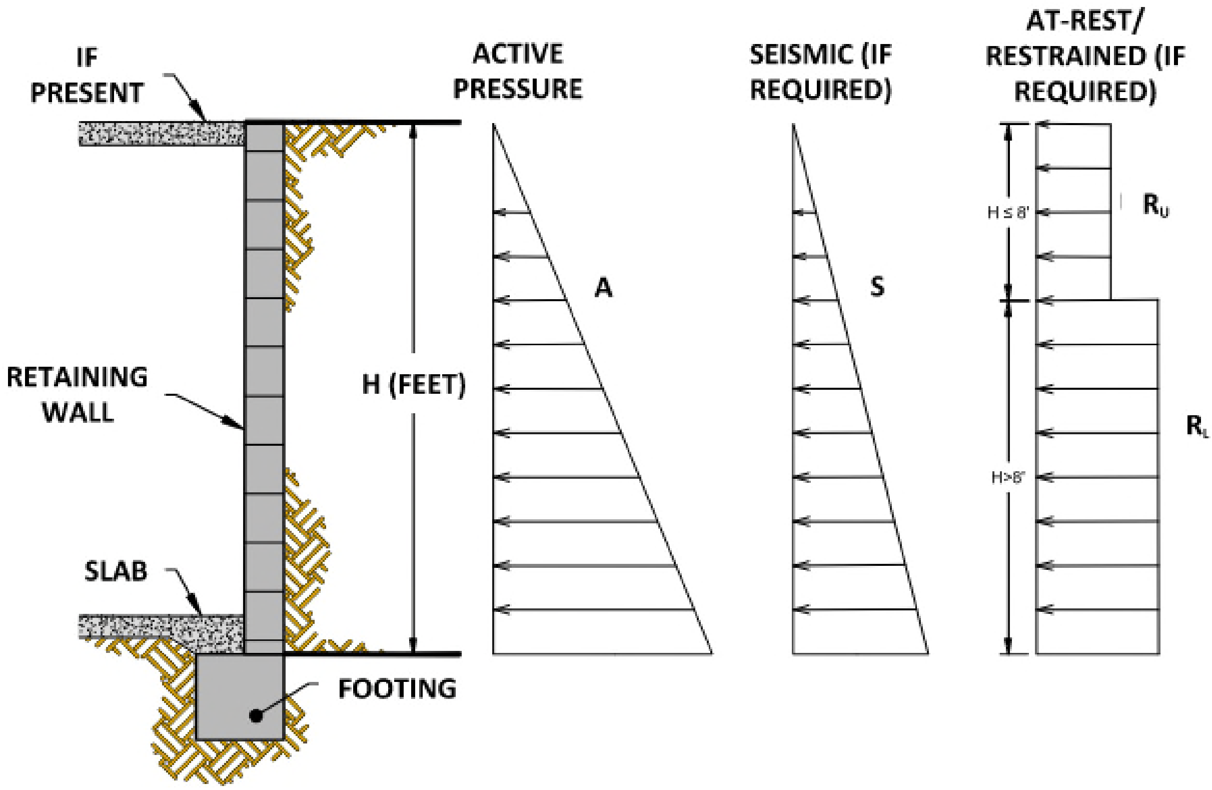
7.9.1 Retaining walls should be supported on compacted fill or granitic rock and should be designed using the values presented in Table 7.9.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 7.9.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	50 pcf
Seismic Pressure, S	16H psf
At-Rest/Restrained Walls Additional Uniform Pressure R_U (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure R_L (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI≤50

H equals the height of the retaining portion of the wall

7.9.2 The project retaining walls should be designed as shown in the Retaining Wall Loading Diagram.



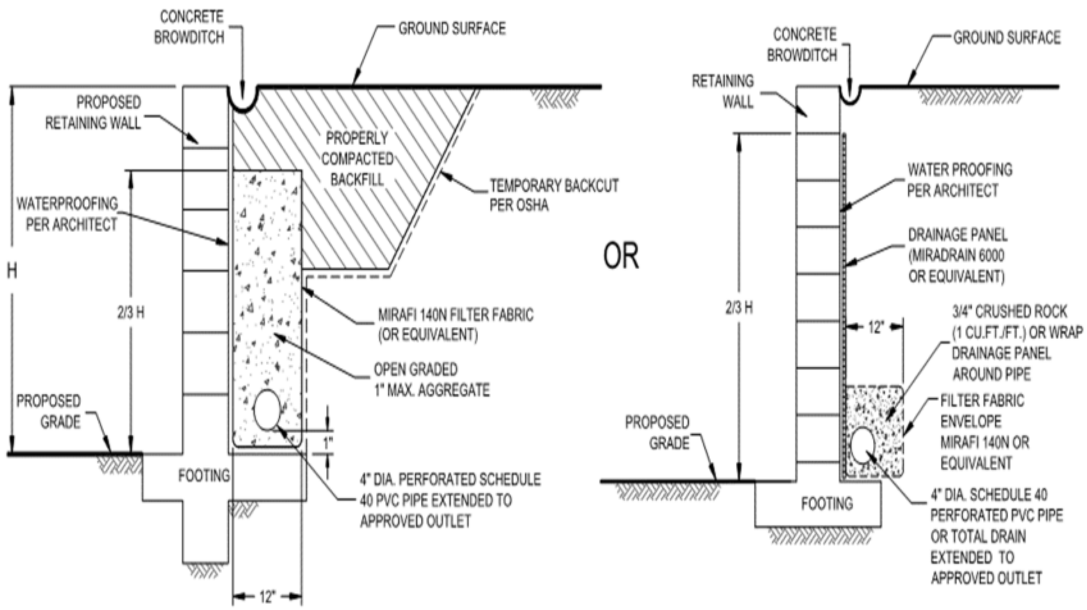
Retaining Wall Loading Diagram

7.9.3 Unrestrained walls are those 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. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be applied to the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

7.9.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2022 CBC or Section 11.6 of ASCE 7-16. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2022 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.

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

7.9.6 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

7.9.7 The retaining walls may be designed using either the active and restrained (at-rest) loading condition or the active and seismic loading condition as suggested by the structural engineer. Typically, it appears the design of the restrained condition for retaining wall loading may be adequate for the seismic design of the retaining walls. However, the active earth pressure combined with the seismic design load should be reviewed and also considered in the design of the retaining walls.

7.9.8 In general, wall foundations should be designed in accordance with Table 7.9.2 for foundations bearing entirely on compacted fill. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 7.9.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,500 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	5,000 psf
Estimated Total Static Settlement	1 Inch
Estimated Differential Static Settlement	½ Inch in 40 Feet

7.9.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls (such as mechanically stabilized earth [MSE] walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

7.9.10 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 retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

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

7.10 Lateral Loading

7.10.1 Table 7.10 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in the design for passive resistance.

TABLE 7.10
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	300 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

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

7.11 Preliminary Pavement Recommendations

7.11.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using estimated Traffic Indices (TI's) of 4.5, 5.0, 6.0 and 7.0 for the parking stalls and interior roadways. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. We have assumed an R-Value of 20 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. The final pavement sections should be based on the R-Value of the subgrade soil encountered at final subgrade elevation once site grading and utility trench backfill is completed. Table 7.11.1 presents the preliminary flexible pavement sections.

TABLE 7.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking Stalls	4.5	20	3	6
Interior Roadways (light-duty)	5.0	20	3	8
Interior Roadways (medium duty)	6.0	20	3.5	10
Interior Roadways (heavy duty)	7.0	20	4	12

7.11.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

7.11.3 Base materials should conform to Section 26-1.02B of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*.

7.11.4 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway entrance aprons, cross-gutters and trash bin loading/storage areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 7.11.2.

TABLE 7.11.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	B and C
Average daily truck traffic, ADTT	25 and 100

7.11.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.11.3.

TABLE 7.11.3
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Medium Duty Areas (TC=B)	6.0
Heavy Duty Areas (TC=C)	7.0

7.11.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch). Base materials will not be required beneath concrete improvements including cross-gutters, curb and gutters, and sidewalks.

7.11.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., a 7.5-inch-thick slab would have a 9.5-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

7.11.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 15 feet for slabs 6 inches and thicker and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report. The depth of the crack-control joints should be at least ¼ of the slab thickness when using a conventional saw, or at least 1 inch when using early-entry saws on slabs 9 inches or less in thickness, as determined by the referenced ACI report discussed in the pavement section herein. Cuts at least ¼ inch wide are required for sealed joints, and a ⅜ inch wide cut is commonly recommended. A narrow joint width of $^{1}/_{10}$ to $^{1}/_{8}$-inch wide is common for unsealed joints.

7.11.9 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, cross-gutters, or sidewalk so water is not able to migrate from the adjacent parkways to the pavement sections.

7.11.10 The performance of pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement and subgrade will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

7.12 Slope Maintenance

7.12.1 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.

7.13 Storm Water Management

7.13.1 Based on the City of Escondido storm water standards manual, full or partial infiltration is considered infeasible and the site exhibits a “no infiltration” condition. Details of our storm water evaluation and recommendations are provided in Appendix C.

7.13.2 As plans progress and details for the storm water design are available for our review, we can provide additional recommendations.

7.14 Site Drainage and Moisture Protection

7.14.1 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 2022 CBC 1804.3 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.

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

7.15 Grading and Foundation Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

3. This report is issued with the understanding that it is the responsibility of the owner, or 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 that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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

THE GEOGRAPHICAL INFORMATION MADE AVAILABLE FOR DISPLAY WAS PROVIDED BY GOOGLE EARTH, SUBJECT TO A LICENSING AGREEMENT. THE INFORMATION IS FOR ILLUSTRATIVE PURPOSES ONLY; IT IS NOT INTENDED FOR CLIENT'S USE OR RELIANCE AND SHALL NOT BE REPRODUCED BY CLIENT. CLIENT SHALL INDEMNIFY, DEFEND AND HOLD HARMLESS GEOCON FROM ANY LIABILITY INCURRED AS A RESULT OF SUCH USE OR RELIANCE BY CLIENT.

N

NO SCALE

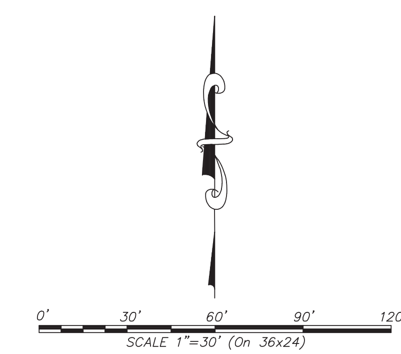
VICINITY MAP

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

PARKVIEW TOWNHOMES
550 WEST EL NORTE PARKWAY
ESCONDIDO, CALIFORNIA

DT / RA		DSK/GTYPD	DATE 10 - 03 - 2024	PROJECT NO. G3180 - 32 - 03	FIG. 1

Plotted:10/03/2024 9:14AM \| By:RUBEN AGUILAR \| File Location:Y:\PROJECTS\G3180-32-03 Parkview Townhomes\DETAILS\G3180-32-03 VicinityMap.dwg



GEOCON LEGEND

- **Kgr** ....GRANITIC ROCK
- AT-10 ....APPROX. LOCATION OF AIR-TRACK BORING
- T-11 ....APPROX. LOCATION OF EXPLORATORY TRENCH
- I-2 ....APPROX. LOCATION OF INFILTRATION TEST
- 32' ....APPROX. THICKNESS OF RIPPABLE ROCK BASED ON A LITERAL INTERPRETATION OF 20 SECONDS PER FOOT AS THE BOUNDARY BETWEEN RIPPABLE AND MARGINALLY TO NON-RIPPABLE ROCK (In Feet)
- (7.0') ....APPROX. THICKNESS OF SURFICIAL DEPOSITS REQUIRING REMEDIAL GRADING (In Feet)
- ....APPROX. AREA WITH HARD ROCK/CORESTONES AT THE GROUND SURFACE THAT WILL LIKELY REQUIRE BLASTING TO EXCAVATE

GEOLOGIC MAP

PARKVIEW TOWNHOMES
550 WEST EL NORTE PARKWAY
ESCONDIDO, CALIFORNIA

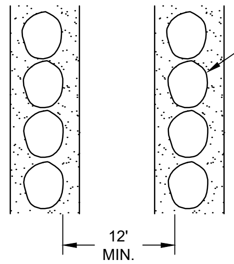
GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

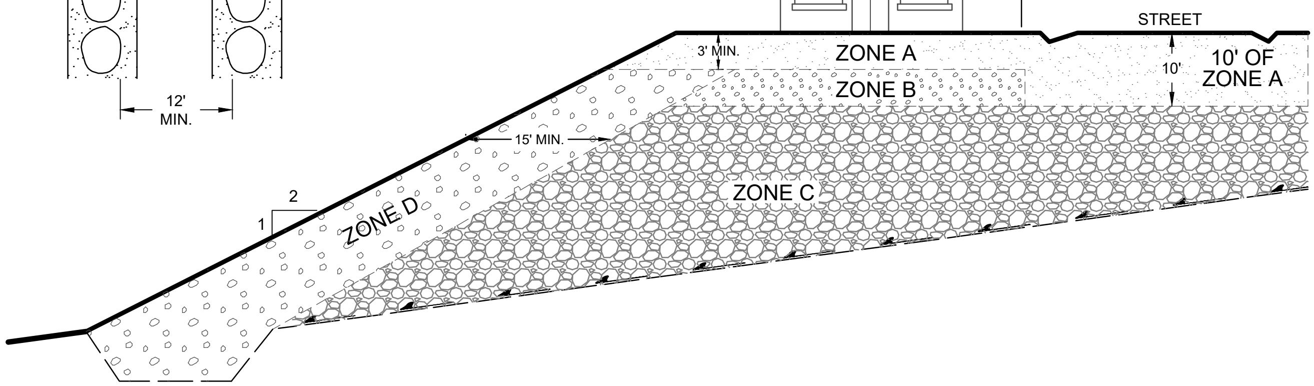
SCALE 1" = 30'	DATE 10 - 03 - 2024		FIGURE 2
PROJECT NO. G3180 - 32 - 03			
SHEET 1 OF 1			

Plotted:10/03/2024 9:12AM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\G3180-32-03 Parkview Townhomes\SHEETS\G3180-32-03 GeoMap.dwg

ZONE B

WINDROWS DETAIL
(PLAN VIEW)





NO SCALE

LEGEND

ZONE A: COMPACTED SOIL FILL. NO ROCK FRAGMENTS OVER 6 INCHES IN DIMENSION. 3 FEET THICK FOR RESIDENTIAL PADS AND 5 FEET THICK FOR SHEET GRADED PADS.

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

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

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

NOTES

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

2. CONTINUOUS OBSERVATION REQUIRED BY GEOCON DURING ROCK PLACEMENT.

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

4. COMBINED DEPTH OF ZONE A AND D SHOULD BE AT LEAST 7 FEET AND SHOULD BE EXTENDED TO AT LEAST 2 FEET BELOW DEEPEST UTILITY WITHIN ROADWAYS.

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

ROCK PLACEMENT DETAIL

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

PARKVIEW TOWNHOMES
550 WEST EL NORTE PARKWAY
ESCONDIDO, CALIFORNIA

DT / RA		DSK/GTYPD	DATE 10 - 03 - 2024	PROJECT NO. G3180 - 32 - 03	FIG. 3

Surficial Slope Stability Evaluation

Slope Height, H (feet)	∞	
Vertical Depth of Stauration, Z (feet)	3	
Slope Inclination	2.00	:1
Slope Inclination, I (degrees)	26.6	
Unit Weight of Water, γW (pcf)	62.4	
Total Unit Weight of Soil, γ_T (pcf)	125	
Friction Angle, ϕ (degrees)	29	
Cohesion, C (psf)	200	
Factor of Safety = $(C+(\gamma_T-\gamma_W)Z \cos^2 i \tan\phi)/(\gamma_T Z \sin i \cos i)$	**1.89**	

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.

Slope Stability Evaluation

Slope Height, H (feet)	35	
Slope Inclination	2.0	:1
Total Unit Weight of Soil, γ_T (pcf)	125	
Friction Angle, ϕ (degrees)	29	
Cohesion, C (psf)	200	
$\gamma_{C\phi} = (\gamma H \tan\phi)/C$	12.1	
N_{Cf} (from Chart)	35	
Factor of Safety = $(N_{Cf}C)/(\gamma H)$	**1.60**	

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, DimensionlessParameters for Homogeneous Earth Slopes,* Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

GEOCON INCORPORATED

GEOTECHNICAL CONSULTANTS

6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121-2974

PHONE 858 558-6900 - FAX 858 558-6159

FILL SLOPE STABILITY ANALYSIS

PARKVIEW TOWNHOMES
ESCONDIDO, CALIFORNIA

SW / SW			DATE 10/3/2024	PROJECT NO. G3180-32-03	FIG. 4

Surficial Slope Stability Evaluation

Slope Height, H (feet)	∞	
Vertical Depth of Stauration, Z (feet)	3	
Slope Inclination	2.00	:1
Slope Inclination, I (degrees)	26.6	
Unit Weight of Water, γW (pcf)	62.4	
Total Unit Weight of Soil, γ_T (pcf)	130	
Friction Angle, ϕ (degrees)	33	
Cohesion, C (psf)	250	

Factor of Safety = $(C+(\gamma_T-\gamma_W)Z \cos^2 i \tan\phi)/(\gamma_T Z \sin i \cos i)$ **2.28**

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.

Slope Stability Evaluation

Slope Height, H (feet)	30	
Slope Inclination	1.5	:1
Total Unit Weight of Soil, γ_T (pcf)	130	
Friction Angle, ϕ (degrees)	33	
Cohesion, C (psf)	500	
$\gamma_{C\phi} = (\gamma H \tan\phi)/C$	5.1	
N_{Cf} (from Chart)	17	

Factor of Safety = $(N_{Cf}C)/(\gamma H)$ **2.18**

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, DimensionlessParameters for Homogeneous Earth Slopes,* Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

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

SW / SW

CUT SLOPE STABILITY ANALYSIS

PARKVIEW TOWNHOMES
ESCONDIDO, CALIFORNIA

DATE 10/3/2024 | PROJECT NO. G3180-32-03 | FIG. 5

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

A field investigation was performed on August 26, 2023 and consisted of visual site reconnaissance and advancing 10 air-track borings using an Ingersoll Rand ECM-590 with a 4-inch bit. A supplemental field investigation was performed on August 16, 2024 and consisted of excavation of 11 exploratory trenches. The trenches (Trench Nos. T-1 through T-11) were advanced by Dave's Drilling using a CAT 420 rubber tire backhoe equipped with a 24-inch-wide bucket. The logs of the trenches depicting the soil and geologic conditions encountered and the depth at which samples were obtained are presented on Figures A-1 through A-11 and the logs of the air track borings are presented on Figures A-12 through A-21. The approximate locations of the exploratory trenches and air track borings are shown on the Geologic Map, Figure 2.

The soil conditions encountered in the excavations were visually classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual Manual Procedure D 2844) and, where applicable, in general conformance with current Caltrans Soil and Rock Logging, Classification and Presentation Manual.

GEOCON

TRENCH NUMBER: T - 1

PROJECT NAME Parkview Townhomes

PROJECT NUMBER G3180-32-03

DATE STARTED 09/16/2024 **COMPLETED** 09/16/2024

CONTRACTOR Dave's Drilling

LOGGED BY D. Thomas

LOCATION South west side of property

LATITUDE / LONGITUDE 33.14548, -117.10041

DEPTH 4' **SURFACE ELEVATION** ~706'

METHOD -

EQUIPMENT CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	706							
	705			SM	**TOPSOIL** Loose to medium dense, dry to damp, reddish brown, **Silty**, fine **SAND**; some granitic clast up to 8"	X		T1-1
2					**GRANITIC ROCK (Kgr)** Strong to very Strong, moderately weathered, gray to reddish brown, **Granitic Rock**; upper 6" completely weathered; excavates to rock clast (3-6")			
4								

PRACTICAL REFUSAL AT 4 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

TRENCH NUMBER: T - 2

PROJECT NAME Parkview Townhomes
LOCATION West side of property
PROJECT NUMBER G3180-32-03
LATITUDE / LONGITUDE 33.14571, -117.1004
DATE STARTED 09/16/2024 **COMPLETED** 09/16/2024
DEPTH 10' **SURFACE ELEVATION** ~718'
CONTRACTOR Dave's Drilling
METHOD -
LOGGED BY D. Thomas
EQUIPMENT CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Sample Interval	Sample Number
	718						
0–3	718–715			SM	**TOPSOIL** Loose to medium dense, dry to damp, reddish brown, **Silty**, fine **SAND**; some granitic clast up to 8"		
3–10	715–708				**GRANITIC ROCK (Kgr)** Weak, completely to highly weathered, reddish brown, **Granitic Rock**; excavates to DG SAND -From 9 feet, becomes moderately weathered, strong to very strong, gray; excavates to gravel/cobble clast		

Depth scale: 2, 4, 6, 8, 10; Elevation marks: 718, 715, 710

PRACTICAL REFUSAL AT 10 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

TRENCH NUMBER: T - 3

PROJECT NAME	Parkview Townhomes	**LOCATION**	North west side of property
PROJECT NUMBER	G3180-32-03	**LATITUDE / LONGITUDE**	33.14604, -117.10037
DATE STARTED	09/16/2024 **COMPLETED** 09/16/2024	**DEPTH**	4' **SURFACE ELEVATION** ~740'
CONTRACTOR	Dave's Drilling	**METHOD**	-
LOGGED BY	D. Thomas	**EQUIPMENT**	CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	740							
0–3				SM	**TOPSOIL** Loose to medium dense, dry to damp, reddish brown, **Silty**, fine to medium **SAND**	X (1–3)		T3-1
2.5–4					**GRANITIC ROCK** (**Kgr**) Strong, moderately weathered, gray, **Granitic Rock**; excavates to gravel/cobble clast			

PRACTICAL REFUSAL AT 4 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

TRENCH NUMBER: T - 4

PROJECT NAME	Parkview Townhomes	**LOCATION**	Central portion of lot
PROJECT NUMBER	G3180-32-03	**LATITUDE / LONGITUDE**	33.14563, -117.09986
DATE STARTED	09/16/2024 **COMPLETED** 09/16/2024	**DEPTH**	8' **SURFACE ELEVATION** ~718'
CONTRACTOR	Dave's Drilling	**METHOD**	-
LOGGED BY	D. Thomas	**EQUIPMENT**	CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	718							
0–3.5	718–714.5			SM	**UNDOCUMENTED FILL** (**Qudf**) Medium dense, damp, medium to reddish brown, **Silty**, fine to medium **SAND**; some gravel and cobble sized granitic clast -Asphalt concrete clast at contact at 3.5 feet	X (0–4)		T4-1
3.5–5.5	715			SM	**TOPSOIL** Loose, moist, reddish brown, **Silty**, fine to medium **SAND**; some organics			
5.5–8	710				**GRANITIC ROCK** (**Kgr**) Strong, moderately weathered, reddish brown to gray, **Granitic Rock**; upper 6" completely to highly weathered; excavates to DG			

Depth scale: 2, 4, 6, 8

PRACTICAL REFUSAL AT 8 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

TRENCH NUMBER: T - 5

PROJECT NAME Parkview Townhomes
LOCATION South side of property
PROJECT NUMBER G3180-32-03
LATITUDE / LONGITUDE 33.14531, -117.09996
DATE STARTED 09/16/2024 **COMPLETED** 09/16/2024
DEPTH 7' **SURFACE ELEVATION** ~705'
CONTRACTOR Dave's Drilling
METHOD -
LOGGED BY D. Thomas
EQUIPMENT CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Sample Interval	Sample Number
	705						
				SM	**UNDOCUMENTED FILL (Qudf)** Loose to medium dense, damp, reddish brown, **Silty**, fine **SAND**, with approx. 3-6" layer of asphalt concrete (2-6' below surface)		
2				SM	**TOPSOIL** Dense, damp, reddish brown, **Silty**, fine **SAND**		
4					**GRANITIC ROCK (Kgr)** Strong to very Strong, moderately weathered, reddish brown to gray, **Granitic Rock**; upper 6" highly weathered; excavates to DG SAND		
	700						
6							

PRACTICAL REFUSAL AT 7 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

TRENCH NUMBER: T - 6

PROJECT NAME Parkview Townhomes	**LOCATION** Center of lot
PROJECT NUMBER G3180-32-03	**LATITUDE / LONGITUDE** 33.14552, -117.09949
DATE STARTED 09/16/2024 **COMPLETED** 09/16/2024	**DEPTH** 9' **SURFACE ELEVATION** ~718'
CONTRACTOR Dave's Drilling	**METHOD** -
LOGGED BY D. Thomas	**EQUIPMENT** CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	718							
0–2				SM	**UNDOCUMENTED FILL** (**Qudf**) Loose, dry, reddish brown, **Silty**, fine to medium **SAND**; some asphalt concrete clast, trace of gravel			
2–4	715			SM	**TOPSOIL** Medium dense, moist to wet, reddish brown, **Silty**, fine **SAND**; trace gravel and cobble sized granitic clast			
4–6						X		T6-1
6				SC	Medium dense, moist, reddish brown, **Clayey**, fine to coarse **SAND** to **CLAY**	X		
8	710				**GRANITIC ROCK** Strong to very Strong, moderately weathered, light brown, **Granitic Rock**; upper 6" highly weathered			

PRACTICAL REFUSAL AT 9 FEET

DRAFT

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

TRENCH NUMBER: T - 7

PROJECT NAME Parkview Townhomes
LOCATION Central esatern portion of site
PROJECT NUMBER G3180-32-03
LATITUDE / LONGITUDE 33.14543, -117.09909
DATE STARTED 09/16/2024 **COMPLETED** 09/16/2024
DEPTH 11' **SURFACE ELEVATION** ~714'
CONTRACTOR Dave's Drilling
METHOD -
LOGGED BY D. Thomas
EQUIPMENT CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	714							
0 – 3.5				SM	**UNDOCUMENTED FILL** (**Qudf**) Loose, damp, reddish brown, **Silty**, fine **SAND**; some 3/4" gravel; some asphalt concrete clast	X (0 – 5)		T7-1
3.5 – 6	710 (at 4)			SM	**TOPSOIL** Medium dense, moist, reddish brown, **Silty**, fine to medium **SAND**			
6 – 7				SC	Medium dense, moist, reddish brown, **Clayey**, fine to coarse **SAND**			
7 – 11	705 (at 9)				**GRANITIC ROCK** (**Kgr**) Strong, highly weathered, light reddish brown, **Granitic Rock**; excavates to DG, upper 8" completely weathered			

Depth scale: 2, 4, 6, 8, 10

PRACTICAL REFUSAL AT 11 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

TRENCH NUMBER: T - 8

PROJECT NAME	Parkview Townhomes	**LOCATION**	-
PROJECT NUMBER	G3180-32-03	**LATITUDE / LONGITUDE**	33.14562, -117.09886
DATE STARTED	09/16/2024 **COMPLETED** 09/16/2024	**DEPTH**	13' **SURFACE ELEVATION** ~720'
CONTRACTOR	Dave's Drilling	**METHOD**	-
LOGGED BY	D. Thomas	**EQUIPMENT**	CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
	720							
0–4				SM	**TOPSOIL** Loose, dry, reddish brown, **Silty**, fine **SAND**; some gravel and cobble sized granitic clast	X (1–5)		T8-1
4–13	715 (5), 710 (10)				**GRANITIC ROCK** (**Kgr**) Weak, completely to highly weathered, reddish brown, **Granitic Rock**; some localized corestones; excavates to DG -From 6 feet, becomes moderately strong to strong, highly weathered; excavates to DG/rock clast up to 8"			

PRACTICAL REFUSAL AT 13 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

TRENCH NUMBER: T - 9

GEOCON

PROJECT NAME	Parkview Townhomes	**LOCATION**	South east portion of site
PROJECT NUMBER	G3180-32-03	**LATITUDE / LONGITUDE**	33.1451, -117.09911
DATE STARTED	09/16/2024	**COMPLETED**	09/16/2024
DEPTH	10'	**SURFACE ELEVATION**	~700'
CONTRACTOR	Dave's Drilling	**METHOD**	-
LOGGED BY	D. Thomas	**EQUIPMENT**	CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Sample Interval	Sample Number
0	700			SM	**UNDOCUMENTED FILL** (**Qudf**) Loose, dry, reddish brown, **Silty**, fine to coarse **SAND**; some 3/4" gravel, trace of rock up to 18", some trash/wood debris		
2							
4							
	695						
6					-From 6 feet, 24-36" boulders above contact		
7					**GRANITIC ROCK** (**Kgr**) Strong, moderately weathered, gray, **Granitic Rock**; excavates to gravel/cobble		
8							
10	690						

PRACTICAL REFUSAL AT 10 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

TRENCH NUMBER: T - 1 0

PROJECT NAME Parkview Townhomes
LOCATION Southern portion of site
PROJECT NUMBER G3180-32-03
LATITUDE / LONGITUDE 33.14502, -117.0993
DATE STARTED 09/16/2024 **COMPLETED** 09/16/2024
DEPTH 5.5' **SURFACE ELEVATION** ~696'
CONTRACTOR Dave's Drilling
METHOD -
LOGGED BY D. Thomas
EQUIPMENT CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Sample Interval	Sample Number
	696						
	695			SM	**UNDOCUMENTED FILL** (**Qudf**) Medium dense, dry, light brown, **Silty**, fine **SAND**; some gravel and cobble sized granitic clast		
2				SM	**TOPSOIL** Medium dense, moist, reddish brown, **Silty**, fine **SAND**		
4					**GRANITIC ROCK** (**Kgr**) Strong, moderately weathered, grayish brown, **Granitic Rock**		

TRENCH TERMINATED AT 5.5 FEET

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

TRENCH NUMBER: T - 1 1

PROJECT NAME Parkview Townhomes
LOCATION Southern portion of site
PROJECT NUMBER G3180-32-03
LATITUDE / LONGITUDE 33.14516, -117.09961
DATE STARTED 09/16/2024 **COMPLETED** 09/16/2024
DEPTH 6' **SURFACE ELEVATION** ~700'
CONTRACTOR Dave's Drilling
METHOD -
LOGGED BY D. Thomas
EQUIPMENT CAT 430F Backhoe

Depth (ft)	Elevation (ft)	Water Levels	Graphic Log	USCS	Material Description	Sample Interval	Sample Number
	700						
				SM	**UNDOCUMENTED FILL** (**Qudf**) Loose, dry, light reddish brown, **Silty**, fine **SAND**; trace of asphalt concrete, trace of 3/4" rock		
2				SM	**TOPSOIL** Medium dense, damp, reddish brown, **Silty**, fine to medium **SAND**		
				SC	Dense, moist, brown, **Clayey**, fine to coarse **SAND**		
4	695				**GRANITIC ROCK** Strong, moderately weathered, gray, **Granitic Rock**; upper 6" highly weathered		
6							

TRENCH TERMINATED AT 6 FEET

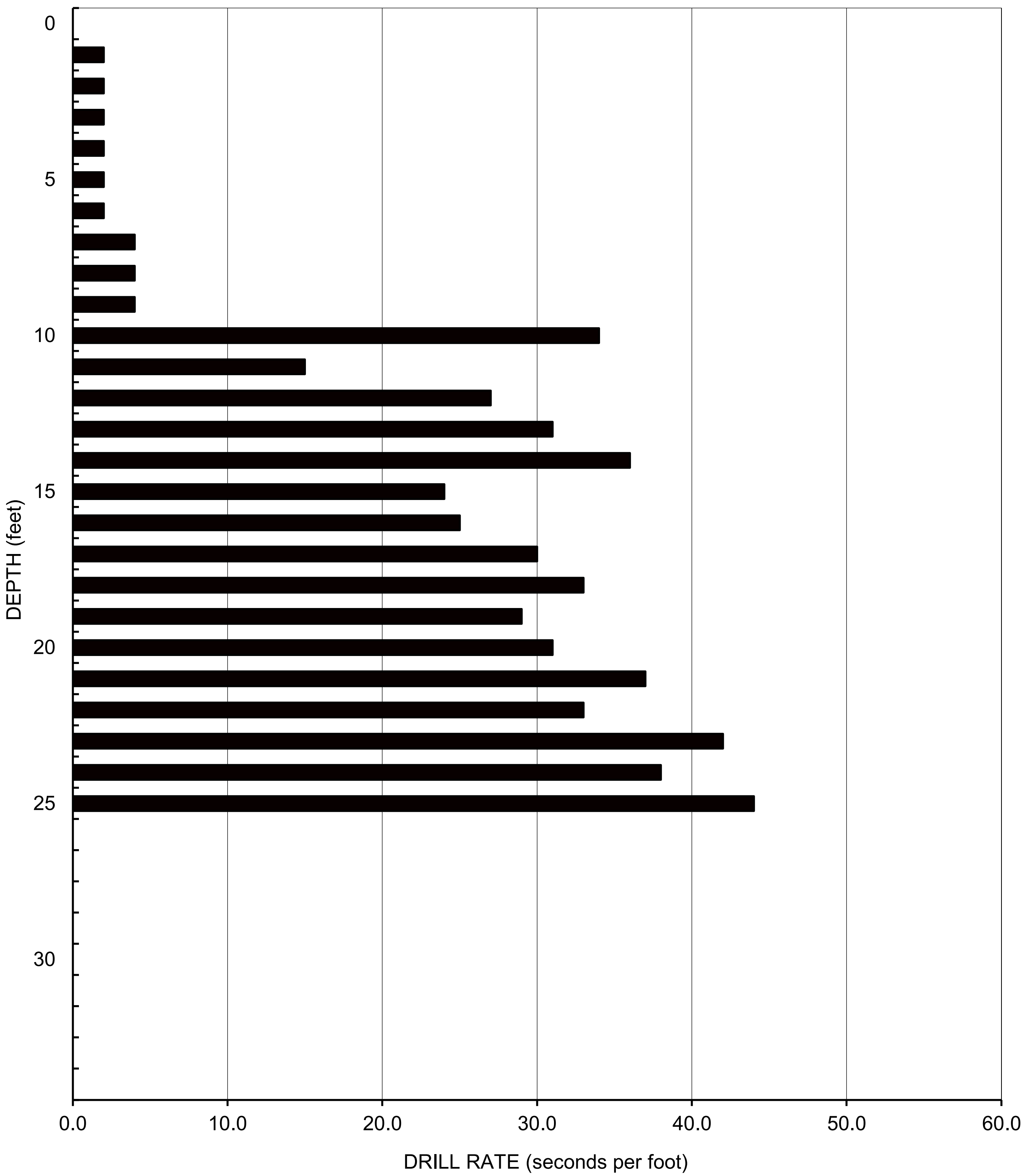
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

AIR TRACK BORING AT-1

Elevation - 747 Feet (MSL)

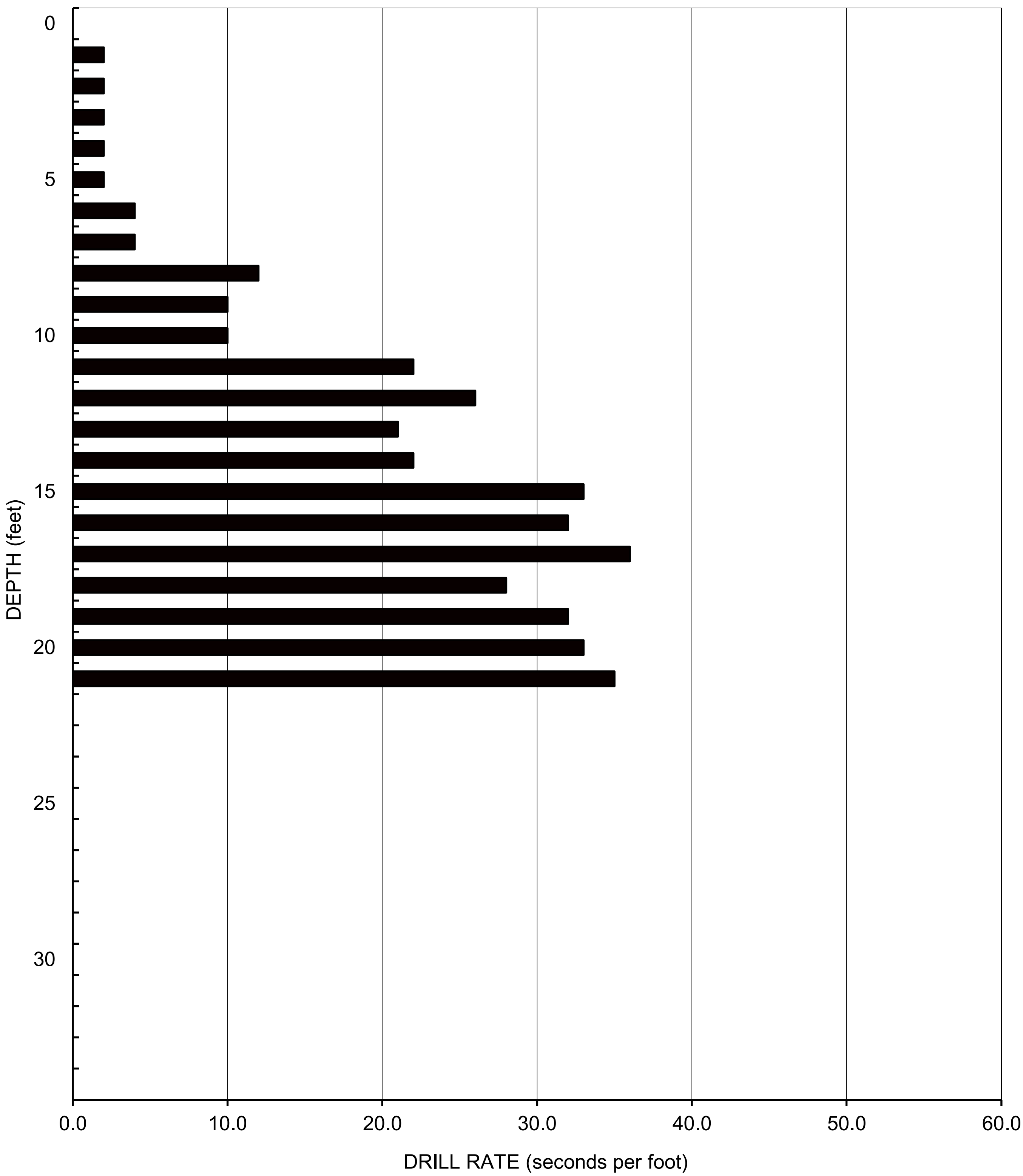
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



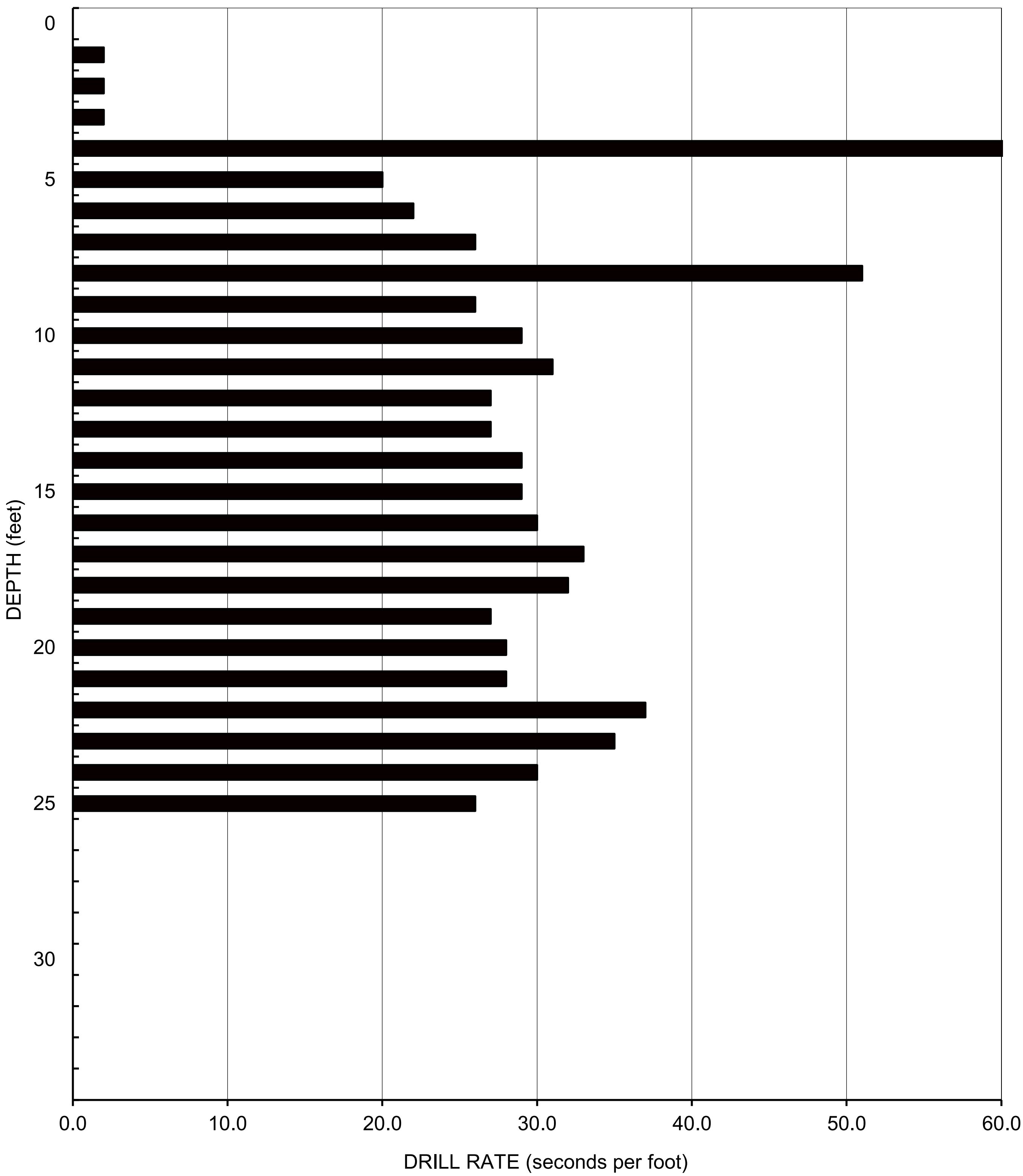
AIR TRACK BORING AT-2
Elevation - 742 Feet (MSL)
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



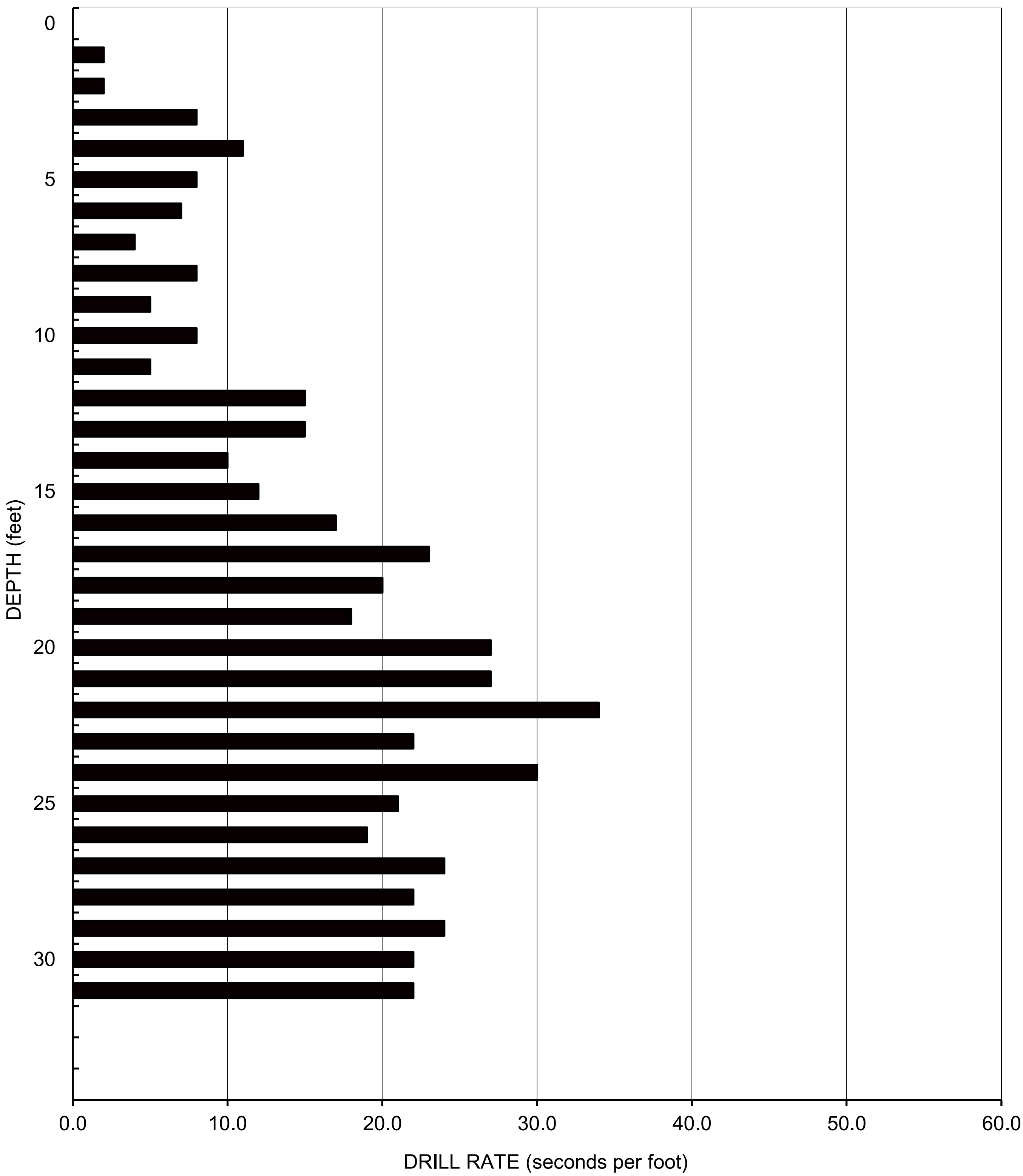
AIR TRACK BORING AT-3
Elevation - 748 Feet (MSL)
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



AIR TRACK BORING AT-4
Elevation - 750 Feet (MSL)
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED

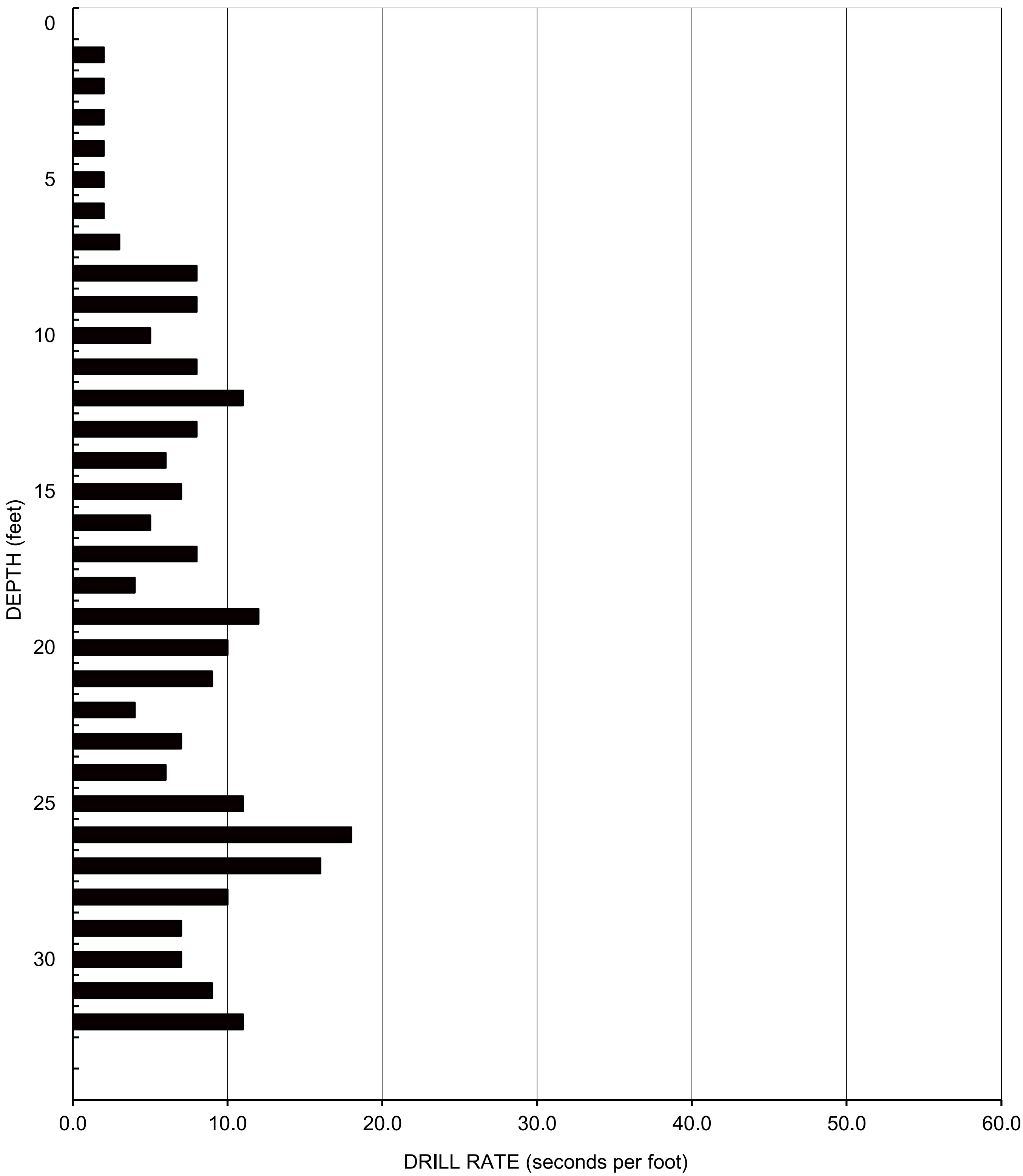


AIR TRACK BORING AT-5

Elevation - 749 Feet (MSL)

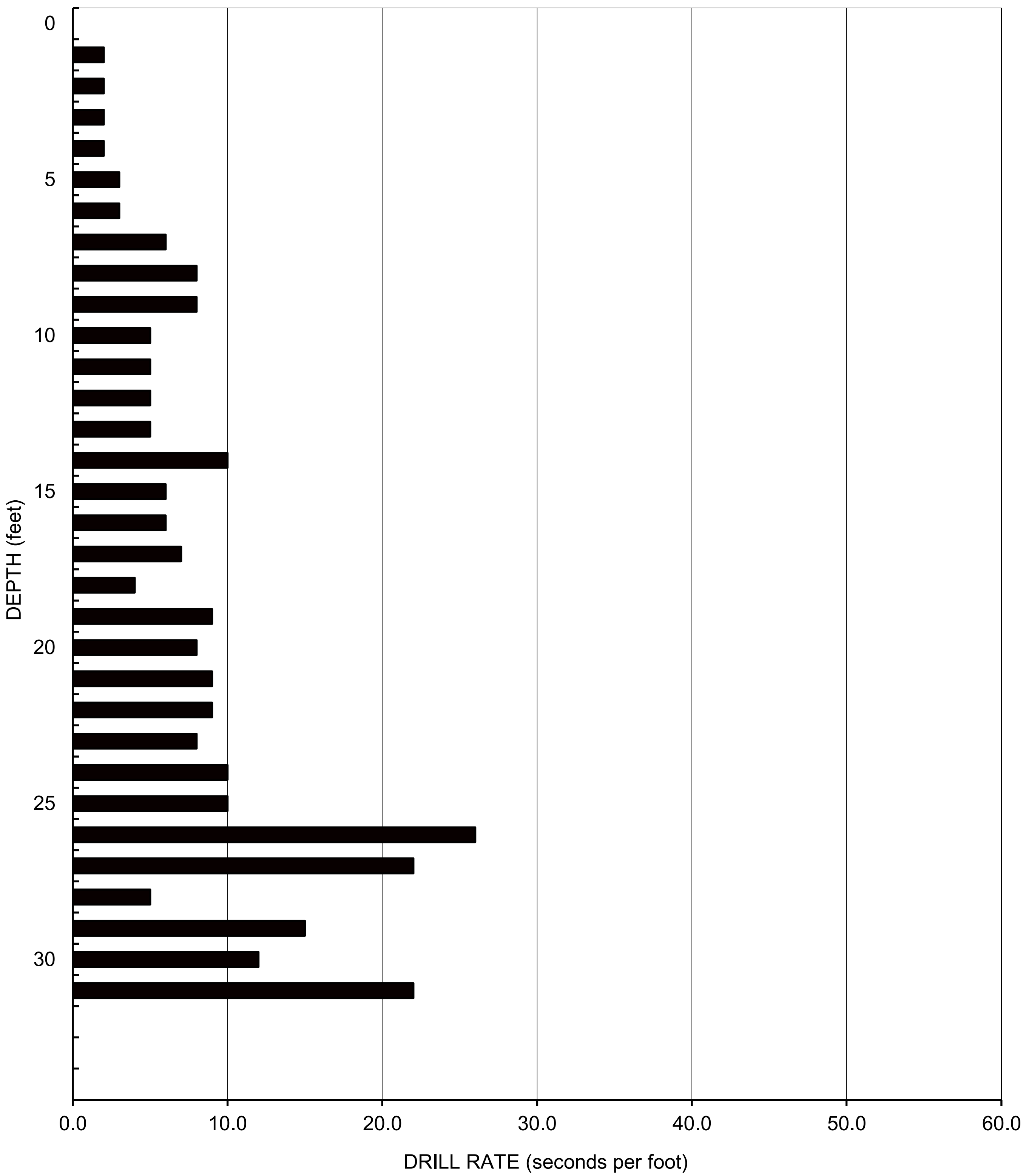
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



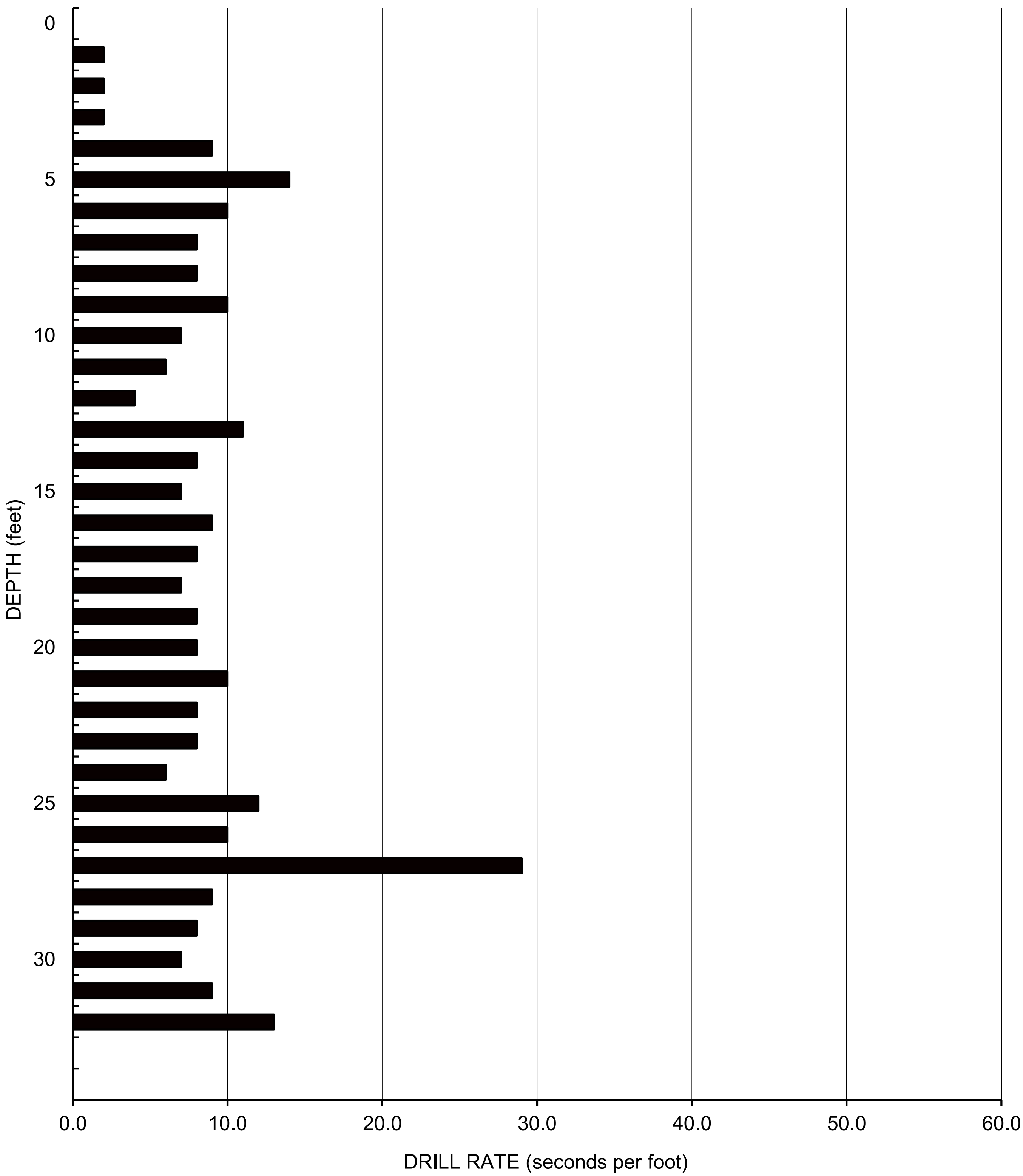
AIR TRACK BORING AT-6
Elevation - 748 Feet (MSL)
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



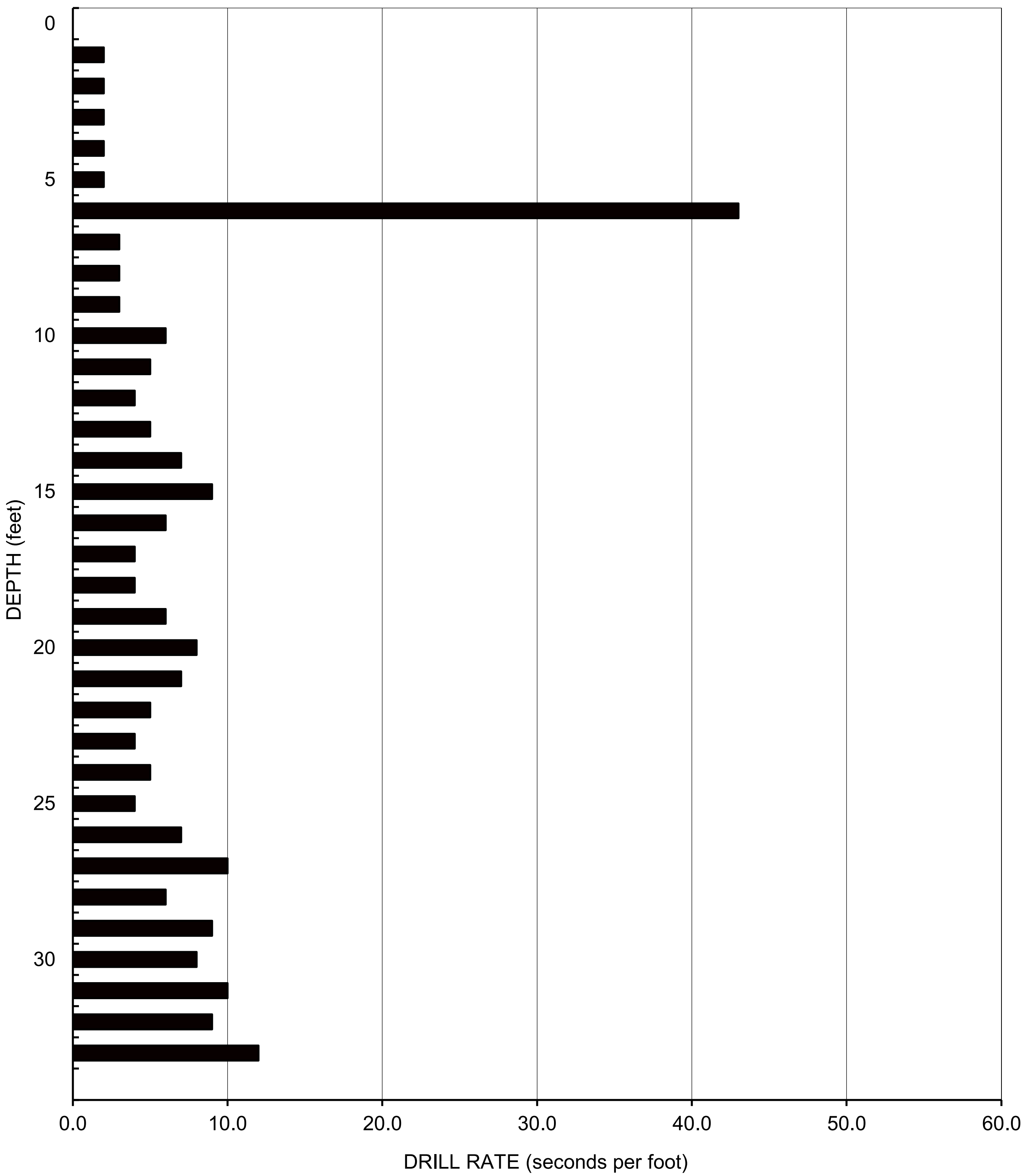
AIR TRACK BORING AT-7
Elevation - 750 Feet (MSL)
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



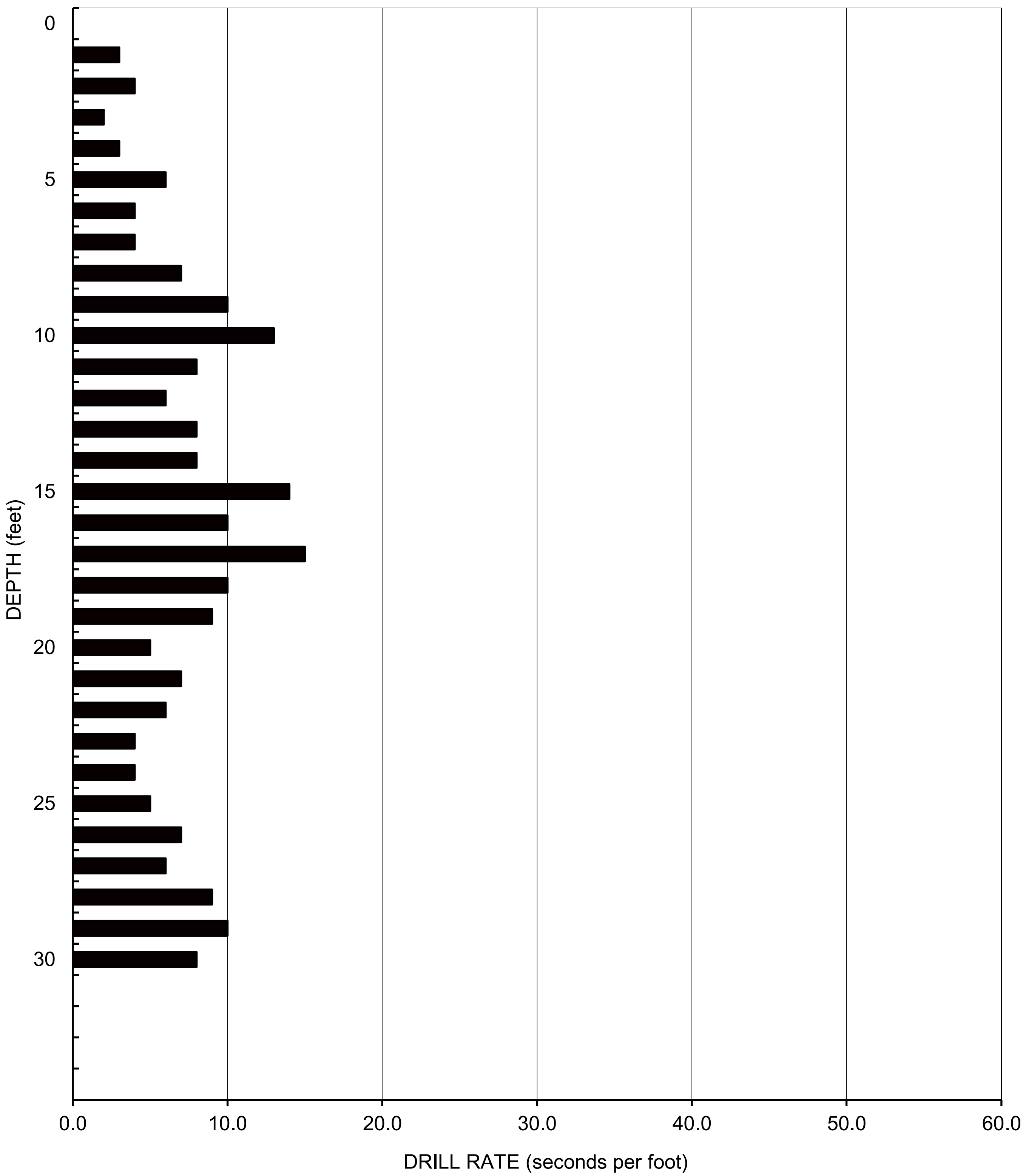
AIR TRACK BORING AT-8
Elevation - 747 Feet (MSL)
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



AIR TRACK BORING AT-9
Elevation - 746 Feet (MSL)
Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



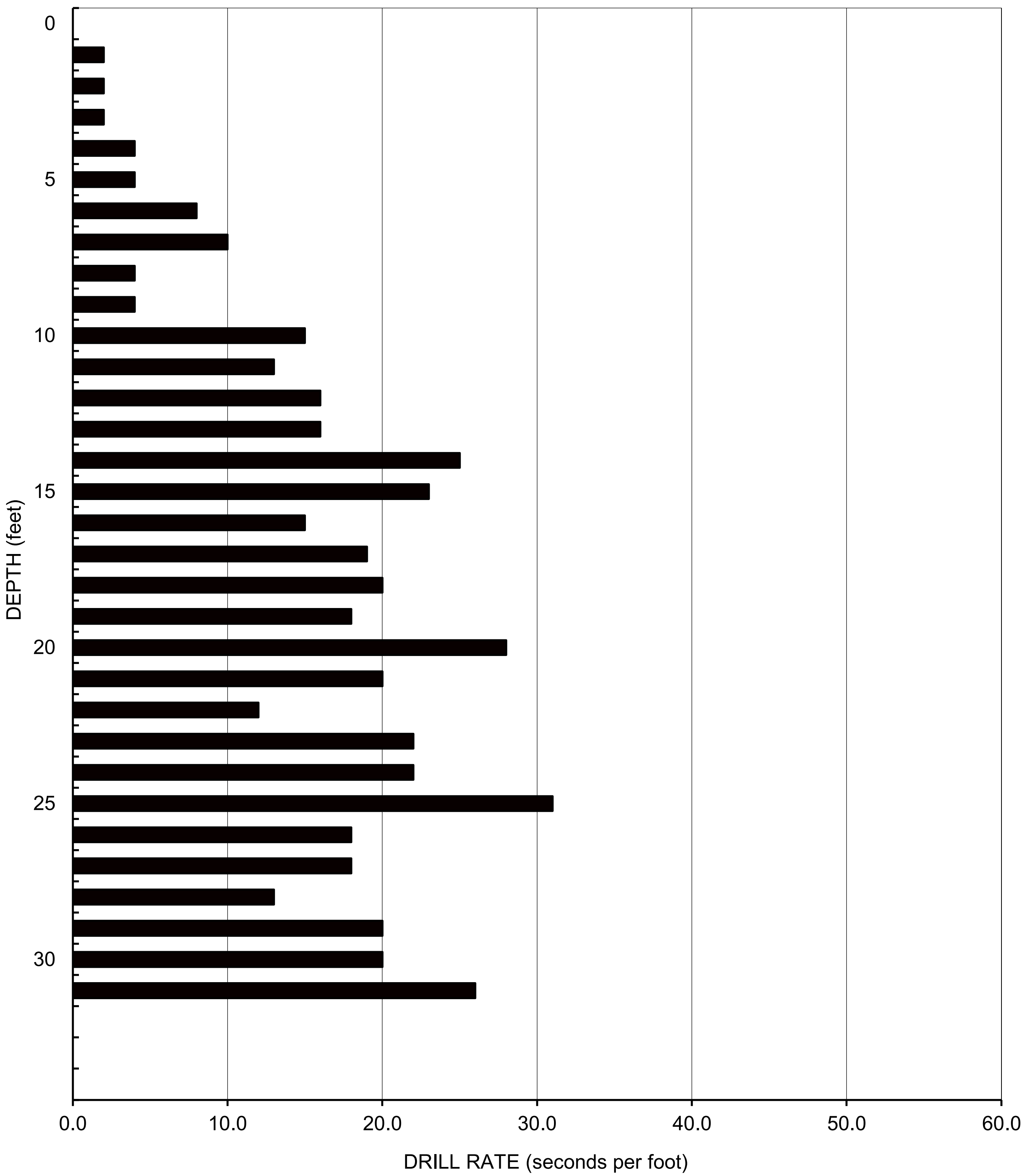
PROJECT NO. G3180-32-03 PARKVIEW TOWNHOMES

AIR TRACK BORING AT-10

Elevation - 747 Feet (MSL)

Date 08-29-2023 - Equipment: 4-Inch Bit IR ECM-590

GEOCON INCORPORATED



AT010.xls

FIGURE A-21

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory testing on select soil samples in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures.

Selected relatively undisturbed samples were tested for dry density and moisture content and shear strength. Selected bulk samples were tested for maximum dry density and optimum moisture content, expansion, and water soluble sulfate content. The results of our laboratory tests are presented in Tables B-1 through B-IV.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T3-1	Reddish brown, Silty, fine to medium SAND	127.6	10.1
T4-1	Reddish brown, Silty, fine to medium SAND; some gravel and cobble sized clasts	128.6	9.4
T8-1	Reddish brown, Silty, fine SAND; some gravel and cobble sized clasts	128.2	9.0

TABLE B-II
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS

Sample No.*	Geologic Unit Symbol (USCS Soil Type)	Dry Density (pcf)	Moisture Content (%)	Peak [Ultimate] Cohesion (psf)	Peak [Ultimate] Angle of Shear Resistance (degrees)
T3-1	Topsoil / Kgr	114.5	10.1	350 [200]	29 [30]

Note (*): Sample remolded to approximately 90 percent relative compaction

TABLE B-III
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No. (Geologic Unit)	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T3-1 (Topsoil/ Kgr)	7.0	12.4	120.5	0
T4-1 (Qudf)	7.6	12.3	120.1	1
T8-1 (Topsoil/ Kgr)	7.4	12.6	119.0	0

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST 417

Sample No.	Geologic Unit (Soil Class)	Water-Soluble Sulfate (%)	Sulfate Severity	Sulfate Class
T3-1	Topsoil/ Kgr (SM)	0.002	Not Applicable	S0
T4-1	Qudf (SM)	0.003	Not Applicable	S0
T8-1	Topsoil/ Kgr (SM)	0.004	Not Applicable	S0

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

FOR

PARKVIEW TOWNHOMES
550 WEST EL NORTE PARKWAY
ESCONDIDO, CALIFORNIA

PROJECT NO. G3180-32-03

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the 2022 City of Escondido Storm Water 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 (NRCS), possesses general information regarding the existing soil conditions for areas within the United States. The USDA NRCS website also provides the Hydrologic Soil Group. Table C-1 presents the descriptions of the hydrologic soil groups. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

TABLE 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 three units identified as Escondido very fine sandy loam, 15 to 30 percent slopes, eroded (EsE2), Vista coarse sandy loam, 5 to 9 percent slopes (VsC), and Vista coarse sandy loam, 15 to 30 percent slopes, MLRA 20 (VsE). The Escondido very fine sandy loam, 15 to 30 percent slopes, eroded (EsE2), Vista coarse sandy loam, 5 to 9 percent slopes (VSC), and Vista coarse sandy loam, 15 to 30 percent slopes, MLRA 20 (VsE) is classified as Soil Group C, B, and B, respectively. Table C-2 presents the information from the USDA NRCS website for the subject property. The Hydrologic Soil Group Map presents output from the USDA website showing the limits of the soil units.

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

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group	k_{SAT} of Most Limiting Layer (inches/hour)
Escondido very fine sandy loam, 15 to 30 percent slopes, eroded	EsE2	37	C	0.57-1.98
Vista coarse sandy loam, 5 to 9 percent slopes	VsC	34	B	1.98 - 5.95
Vista coarse sandy loam, 15 to 30 percent slopes, MLRA 20	VsE	28	B	0.00 – 0.06

Hydrologic Soil Group Map

In-Situ Testing

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

TABLE C-3
SOIL PERMEABILITY DEFINITIONS

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

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

In-Situ Testing

We performed 2 infiltration tests within the general area of the proposed storm water management basin using a downhole permeameter method. The results of the tests provide parameters regarding the saturated hydraulic conductivity and infiltration characteristics of on-site soil and geologic units. The following table represents the results of the estimated field saturated hydraulic conductivity and estimated infiltration rates obtained from the percolation tests. The field sheets are also attached herein. Based on the *City of Escondido BMP Design Manual*, the infiltration rate should be considered equal to the saturated hydraulic conductivity rate. We applied a feasibility factor of safety of 2.0 to our estimated infiltration rates to provide input on Worksheet C.4-1. Soil infiltration rates from in-situ tests can vary significantly from one location to another due to the heterogeneous characteristics inherent to most soil.

TABLE C-4
FIELD PERMEATER INFILTRATION TEST

Test Location	Test Depth Elevation (in feet MSL)	Geologic Unit	Field-Saturated Infiltration Rate, k_{sat} (inch/hour)	C.4-1 Worksheet Infiltration Rate[1], k_{sat} (inch/hour)
I-1	691	Kgr	0.08	0.04
I-2	694	Kgr	0.08	0.04
		Average:	**0.08**	**0.04**

[1] Using a factor of safety of 2.

Infiltration categories include full infiltration, partial infiltration and no infiltration. Table C-5 presents the commonly accepted definitions of the potential infiltration categories based on the infiltration rates.

TABLE C-5
INFILTRATION CATEGORIES

Infiltration Category	Field Infiltration Rate, I (Inches/Hour)	Factored Infiltration Rate*, I (Inches/Hour)
Full Infiltration	I > 1.0	I > 0.5
Partial Infiltration	0.10 < I ≤ 1.0	0.05 < I ≤ 0.5
No Infiltration (Infeasible)	I < 0.10	I < 0.05

*Using a Factor of Safety of 2.

GEOLOGIC HAZARDS AND CONSIDERATIONS

Groundwater Elevations

We did not encounter static groundwater during our field investigation to the maximum depth explored of 13 feet. We expect static groundwater exists at depths greater than 100 feet below existing grades. Therefore, we do not consider groundwater elevations to be a constraint for infiltration at the subject site.

Soil Properties

We encountered moderate to fresh granitic rock during our field investigation. Moderate weathering to fresh granitic rock can significantly reduce the permeability of the materials and restricts the movement of water through soil. In our experience, moderate to fresh granitic rock is impervious and does not allow for infiltration and water tends to perch on the contact, causing lateral water migration. Therefore, based on the geologic characteristics of the granitic rock, infiltration should not be allowed when considering the geologic properties of the formational unit. The proposed storm water BMPs will be underlain by granitic rock. Hazards that occur as a result of infiltration on relatively impervious layers include a potential for lateral water migration, slope instability and daylight water seepage, that may adversely impact onsite and adjacent properties.

Slope Hazards

Topographically, the site consists of gentle to moderately sloping terrain that drains towards the south. The elevations on the site range from 754 feet mean sea level (MSL) to 686 feet MSL. Infiltration BMP's situated in close proximity to cut or fill slopes are not recommended due to the potential for daylight water seepage, lateral water migration, and slope instability.

New or Existing Utilities

Existing utilities are present within the existing streets and public sidewalks and new utilities will be constructed within the site boundaries. We expect that all on-site utilities will be removed prior to site development. Full or partial infiltration near existing or proposed utilities should be avoided to prevent lateral water migration into the permeable trench backfill materials. For areas with existing or proposed utilities, full and partial infiltration should be considered infeasible. Setbacks for infiltration should be incorporated if infiltration will be used for the devices. The setback for infiltration devices should be a minimum of 10 feet and a 1:1 plane of 1 foot below the closest edge of the deepest adjacent utility. Utilities should not be installed below infiltration devices, if planned.

Existing and Planned Structures

Existing structures are located at the northern portion of the property and existing residential structures are situated near the eastern proximity to the property boundaries. Additionally, residential buildings are planned for the central and northern portions of the site. If water is allowed to infiltrate into the soil, the water could migrate laterally and into other properties in the vicinity of the subject site. The water migration may negatively affect other buildings and improvements. Water should not be allowed to infiltrate areas where it could affect the existing adjacent structures or properties. Mitigation for existing structures consists of not allowing water infiltration within a lateral distance of at least 10 feet from the new or existing foundations and property lines.

Soil or Groundwater Contamination

We are unaware of contaminated soil on the property. Therefore, infiltration associated with this risk is not considered a constraint.

Infiltration Rates

Our test results indicated an average unfactored infiltration rate of 0.08 in/hr. The average design infiltration rate would be 0.04 in/hr. assuming a factor of safety of 2. The unfactored and factored infiltration rates of 0.08 and 0.04 in/hr., respectively, are below the minimum thresholds for full or partial infiltration.

CONCLUSIONS AND RECOMMENDATIONS

The Geologic Map, Figure 2, depicts the existing property, proposed development, the approximate lateral limits of the geologic units, the locations of the field excavations and the in-situ infiltration test locations.

Storm Water Evaluation Narrative and Conclusions

The in-place infiltration test locations were selected in areas used for possible basins and storm water management devices that correspond with the natural drainage on the site (located in the low points of the property). We performed 2 infiltration tests within the granitic rock and the results indicate an average rate of 0.04 inches per hour with an applied factor of safety of 2.

At the conclusion of grading, we expect the area of the proposed BMP will be underlain by granitic rock. Granitic rock exhibits very low infiltration rates that do not support infiltration BMP's. Based on the results of our infiltration tests performed within the granitic rock (less than 0.05 inches per hour), we opine full and partial infiltration on the property to be considered restricted on this site.

Storm Water Infiltration Recommendations

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 help prevent vertical and lateral water migration. The subdrains should be perforated within the liner area, installed at the base and above the liner, be at least 4 inches in diameter and consist of Schedule 40 PVC pipe. The subdrains outside of the liner should consist of solid pipe. 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

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

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

SUITABILITY ASSESSMENT RELATED CONSIDERATIONS FOR INFILTRATION FACILITY SAFETY FACTORS

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

Based on our geotechnical investigation and the previous table, the following table presents the estimated factor values for the evaluation of the factor of safety. This table only presents the suitability assessment safety factor (Part A) of the worksheet. The project civil engineer should evaluate the safety factor for design (Part B) and use the combined safety factor for the design infiltration rate.

FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A1

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	2	0.50
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/ Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \sum p$			1.75

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

CONCLUSION

Our infiltration test results indicate the underlying granitic rock at the site possesses very low infiltration rates that are not conducive to infiltration. We expect there is a potential for lateral water migration, daylight water seepage and slope instability if infiltration is allowed. It is our opinion that full or partial infiltration is considered restricted on this site.

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions[9]

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

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

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

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]

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

We performed 2 constant head borehole infiltration tests. The unfactored test results were 0.08 iph, respectively. After applying a feasibility factor of safety of 2, the design infiltration rates would be 0.04 iph, which are below the minimum threshold for full infiltration BMP's. The formational materials are too dense to support infiltration BMP's.

Criteria 2: Geologic/Geotechnical Screening

2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	☐ Yes	☐ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐ Yes	☐ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐ Yes	☐ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐ Yes	☐ No
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐ Yes	☐ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐ Yes	☐ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	☐ Yes	☐ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No
Summarize findings and basis; provide references to related reports or exhibits.			

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]
Part 1 Result – Full Infiltration Geotechnical Screening [12]		**Result**
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.		☐ Full infiltration Condition ☑ Complete Part 2
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		**Project Phase:**
All		Planning
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☐ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☑ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ☐ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ☑ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ☐ Yes; Continue to Criteria 4. ☑ No: Skip to Part 2 Result.		
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). We performed 2 constant head borehole infiltration tests. The unfactored test results were 0.08 iph, respectively. After applying a feasibility factor of safety of 2, the design infiltration rates would be 0.04 iph, which are below the minimum threshold for partial infiltration BMP's. The formational materials are too dense to support infiltration BMP's.			
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	☐ Yes	☐ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐ Yes	☐ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐ Yes	☐ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1 If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐ Yes	☐ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐ Yes	☐ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐ Yes	☐ No
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐ Yes	☐ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐ Yes	☐ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	☐ Yes	☐ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No
Summarize findings and basis; provide references to related reports or exhibits.			

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]
Part 2 – Partial Infiltration Geotechnical Screening Result[13]	**Result**
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	☐ Partial Infiltration Condition ☑ No Infiltration Condition

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

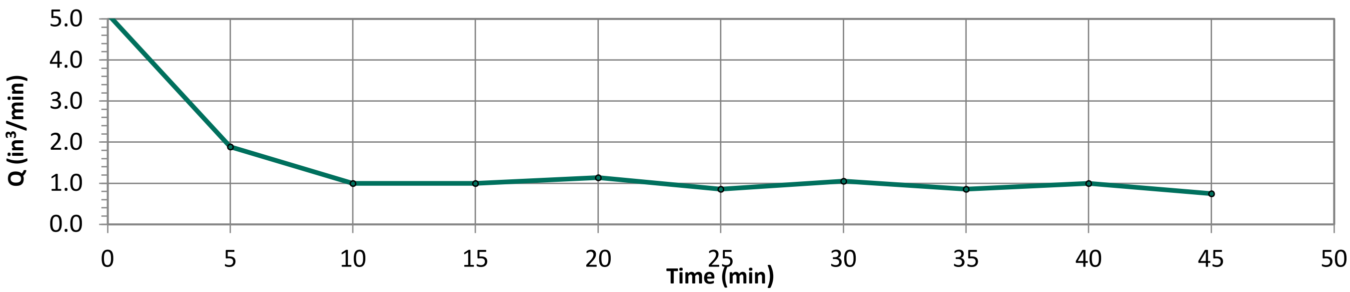
TEST NO.: **I-1**

GEOLOGIC UNIT: **Kgr**

EXCAVATION ELEVATION (MSL, FT): **696**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	8
BOREHOLE DEPTH (FT):	5.5
TEST/BOTTOM ELEVATION (MSL, FT):	691
MEASURED HEAD HEIGHT (IN):	4.3
CALCULATED HEAD HEIGHT (IN):	6.5
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.868**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.081**
FACTORED INFILTRATION RATE (IN/HR):	**0.040**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.925	25.62	5.123
3	5.00	0.340	9.42	1.883
4	5.00	0.180	4.98	0.997
5	5.00	0.180	4.98	0.997
6	5.00	0.205	5.68	1.135
7	5.00	0.155	4.29	0.858
8	5.00	0.190	5.26	1.052
9	5.00	0.155	4.29	0.858
10	5.00	0.180	4.98	0.997
11	5.00	0.135	3.74	0.748

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

DOWNHOLE PERMEAMETER TEST RESULTS

PARKVIEW TOWNHOMES

PROJECT NO.: G3180-32-03

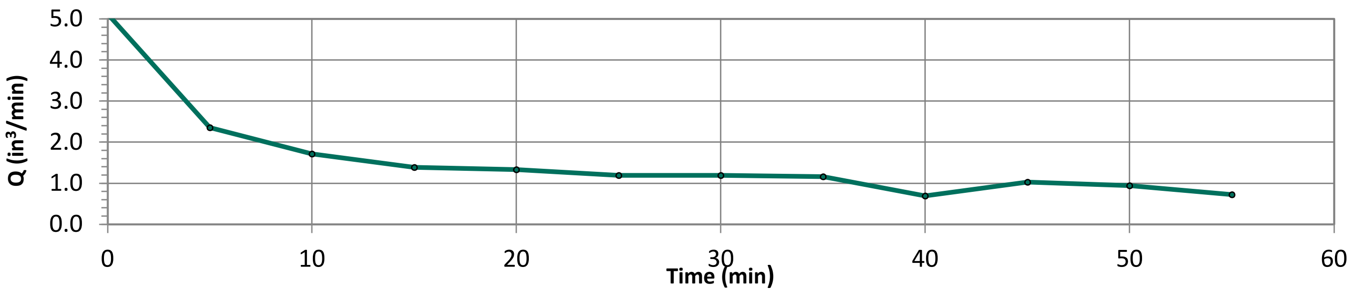
TEST NO.: **I-2**

GEOLOGIC UNIT: **Kgr**

EXCAVATION ELEVATION (MSL, FT): **700**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	8
BOREHOLE DEPTH (FT):	6.0
TEST/BOTTOM ELEVATION (MSL, FT):	694
MEASURED HEAD HEIGHT (IN):	4.5
CALCULATED HEAD HEIGHT (IN):	6.6
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.895**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.080**
FACTORED INFILTRATION RATE (IN/HR):	**0.040**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.925	25.62	5.123
3	5.00	0.425	11.77	2.354
4	5.00	0.310	8.58	1.717
5	5.00	0.250	6.92	1.385
6	5.00	0.240	6.65	1.329
7	5.00	0.215	5.95	1.191
8	5.00	0.215	5.95	1.191
9	5.00	0.210	5.82	1.163
10	5.00	0.125	3.46	0.692
11	5.00	0.185	5.12	1.025
12	5.00	0.170	4.71	0.942
13	5.00	0.130	3.60	0.720

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

DOWNHOLE PERMEAMETER TEST RESULTS

PARKVIEW TOWNHOMES

PROJECT NO.: G3180-32-03

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

PARKVIEW TOWNHOMES
550 WEST EL NORTE PARKWAY
ESCONDIDO, CALIFORNIA

PROJECT NO. G3180-32-03

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

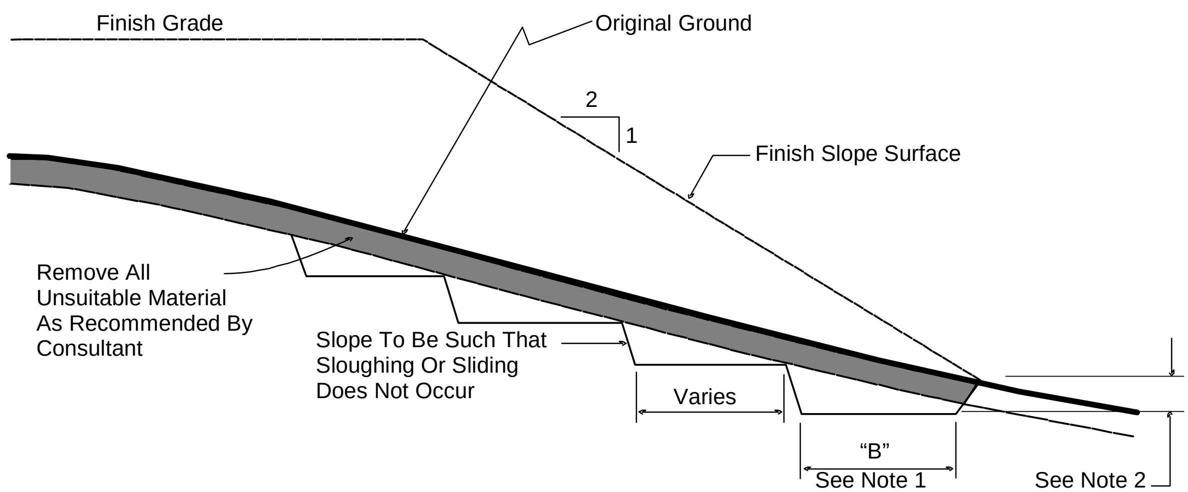
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



No Scale

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

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

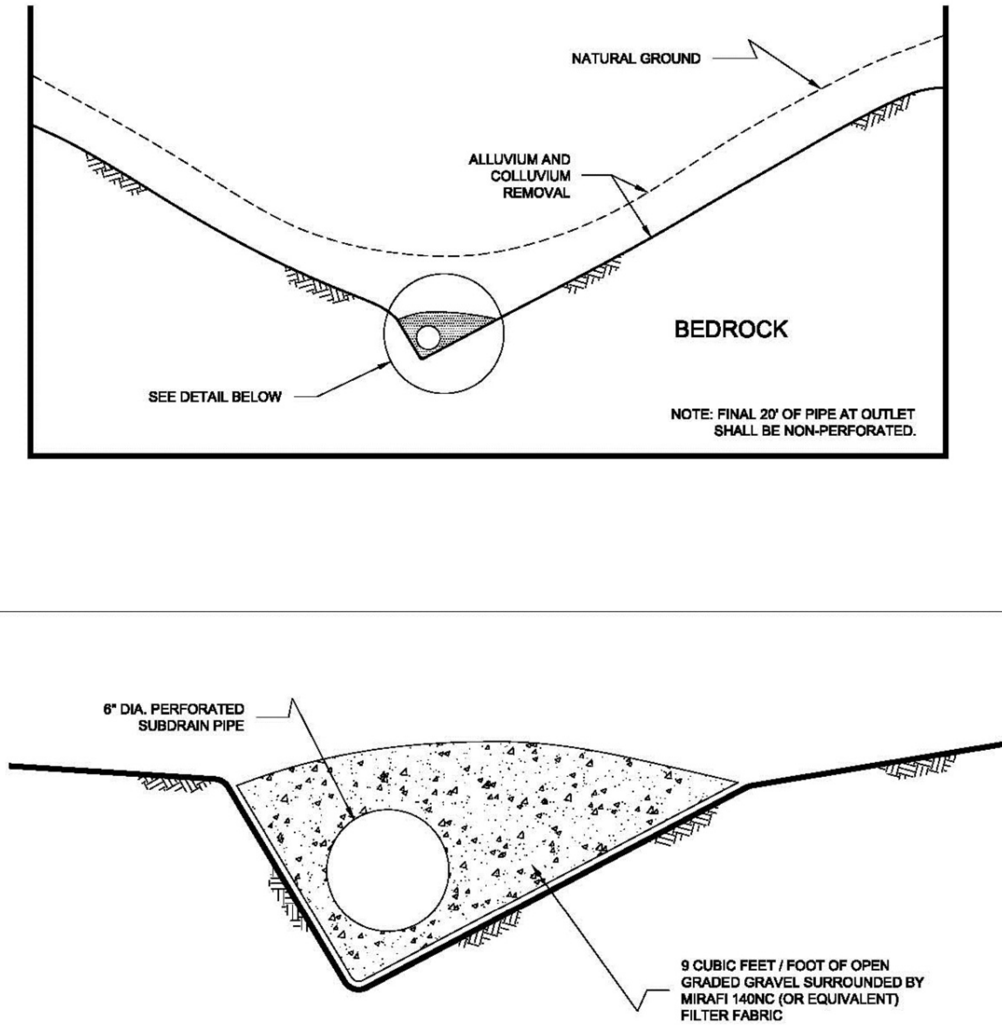
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

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

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL



NOTES:

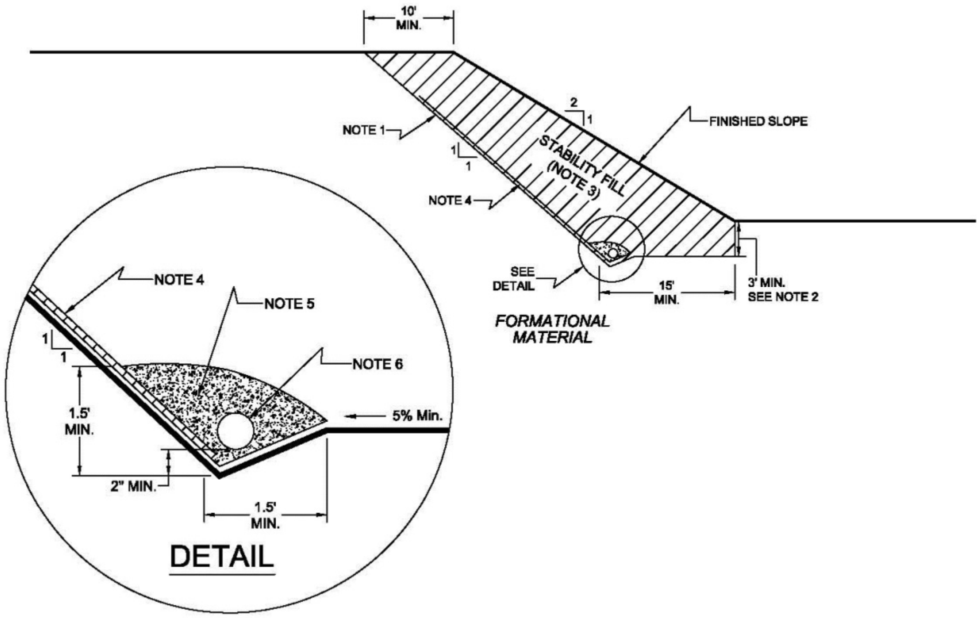
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

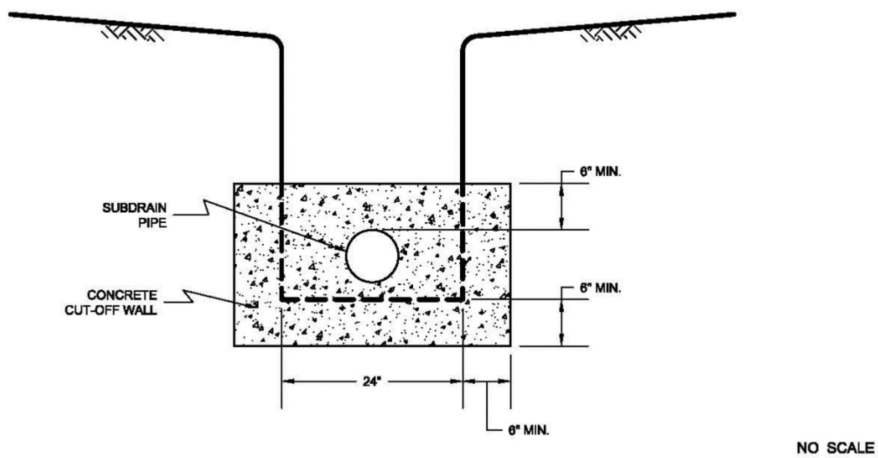
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

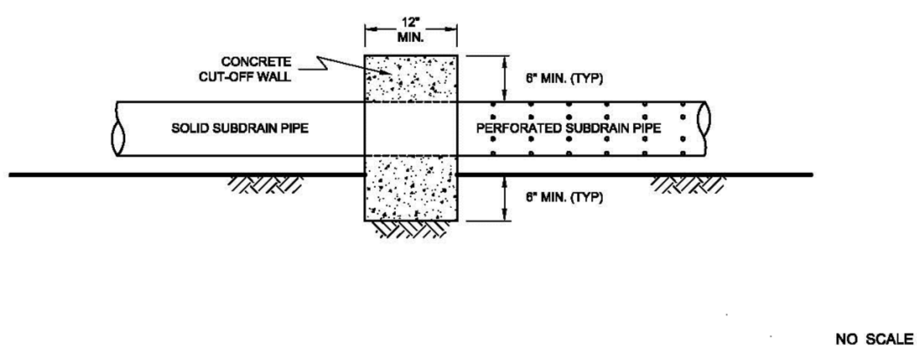
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



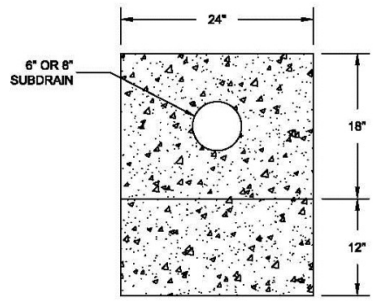
SIDE VIEW



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

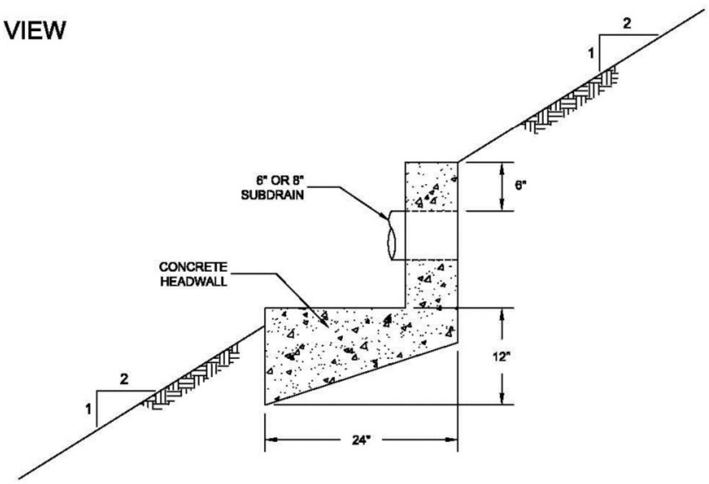
TYPICAL HEADWALL DETAIL

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.

LIST OF REFERENCES

1. 2022 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2021 International Building Code, prepared by California Building Standards Commission, adopted January 2023.

2. ACI 330-08, Guide for the Design and Construction of Concrete Parking Lots, prepared by the American Concrete Institute, dated June, 2008.

3. American Concrete Institute, ACI 318-11, Building Code Requirements for Structural Concrete and Commentary, dated August, 2011.

4. American Society of Civil Engineers (ASCE), ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, 2017.

5. County of San Diego, San Diego County Multi-Jurisdiction Hazard Mitigation Plan, San Diego, California – Final Draft, dated October 2017.

6. Department of Conservation, California Geologic Survey (formerly California Division of Mines and Geology), *Geologic Maps of the Valley Center, 7.5 minute Quadrangle, San Diego County, California*, by Siang Tan and Michael P. Kennedy, 1999.

7. Historical Aerial Photos. http://www.historicaerials.com

8. http://www.water.ca.gov.

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

10. 1953 stereoscopic aerial photographs of the subject site and surrounding areas.