GEOTECHNICAL INVESTIGATION

AIRWAY ROAD
INDUSTRIAL BUILDING
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

BADIEE DEVELOPMENT
LA JOLLA, CALIFORNIA

MAY 18, 2020
PROJECT NO. G2467-42-01

Project No. G2467-42-01
May 18, 2020

Badiee Development
Post Office Box 3111
La Jolla, California 92038

Attention: Mr. Scott Merry

Subject: GEOTECHNICAL INVESTIGATION
AIRWAY ROAD INDUSTRIAL BUILDING
AIRWAY ROAD
SAN DIEGO, CALIFORNIA

Dear Mr. Merry:

In accordance with your request, we have prepared this geotechnical investigation report for a proposed industrial building at the subject site. The site is underlain by Pleistocene Terrace Deposits mantled by topsoil. Some undocumented fill stockpiles are present on the property.

This report is based on our observations made during our field investigation performed on November 15, 2019, and laboratory testing. Based on the results of this study, we opine that the subject site is suitable for construction of the proposed industrial building. The accompanying report presents the results of our study and conclusions and recommendations regarding geotechnical aspects of site development.

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

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for a proposed industrial building located south of the intersection of Airway Road and Centurion Street, in San Diego, California (see Vicinity Map, Figure 1). The purpose of our investigation was to evaluate subsurface soil and geologic conditions at the site, and provide conclusions and recommendations pertaining to the geotechnical aspects of developing the property as proposed.

The scope of our investigation included a site reconnaissance, excavation and logging of 17 test pits, performing 3 infiltration tests in areas of proposed storm water basins or other storm water management devices, and reviewing published and unpublished geologic literature and reports (see List of References). Appendix A presents a discussion of our field investigation. We performed laboratory tests on soil samples obtained from the exploratory test pits to evaluate pertinent physical properties for engineering analyses. The results of laboratory testing are presented in Appendix B.

Site geologic conditions are depicted on Figure 2 (Geologic Map). A CAD file of the preliminary grading plan prepared by K&S Engineering was utilized as a base map to plot geologic contacts and trench locations.

The conclusions and recommendations presented herein are based on our analysis of the data obtained during the investigation, and our experience with similar soil and geologic conditions on this and adjacent properties.

2. SITE AND PROJECT DESCRIPTION

The property consists of an undeveloped rectangular parcel located south of the intersection of Airway Road and Centurion Street, in the Otay Mesa district of San Diego, California (see Figure 1). The subject parcel consists of approximately 13 acres of undeveloped land and is approximately 440 feet wide and 1,250 feet long. Site topography is relatively flat with elevations ranging from approximately 477 feet to 489 feet above mean sea level (MSL).

The proposed improvements will consist of a single-story approximately 247,000 square-foot industrial warehouse building with associated improvements such as utilities, paving, storm water management devices, and landscape improvements. Retaining walls are expected for loading docks. Proposed cuts and fills are estimated to be less than seven feet. New slopes are not anticipated.

The locations and descriptions of the site and proposed development are based on our site reconnaissance and recent field investigations, and our understanding of site development as shown on the preliminary grading plan by K&S Engineering. If project details vary significantly from those described, Geocon Incorporated should be contacted to review the changes and provide additional analyses and/or revisions to this report, if warranted.

3. SOIL AND GEOLOGIC CONDITIONS

Based on the results of the field investigation, the site is underlain by Pleistocene Terrace Deposits mantled by topsoil, which are described below in order of increasing age. Portions of the site are covered in undocumented fill and end-dumped piles of trash and construction debris. Mapped geologic conditions are depicted on the *Geologic Map* (Figure 2), and on the *Geologic Cross Section* (Figure 3). Exploratory test pit logs are presented in Appendix A, Figures A-1 through A-17.

3.1 Undocumented Fill (Qudf)

The north and east portions of the site have numerous end-dumped soil and trash piles, consisting of construction and landscaping debris, plastic, metal, and other debris. We also observed several old wooden power poles and fiberglass boat hulls. These soils and debris are unsuitable for support of structural fill or other improvements in their present condition. Trash should be hauled offsite prior to grading. Miscellaneous soil piles can be incorporated during grading, provided they are free of trash and/or hazardous substances.

3.2 Topsoil (Unmapped)

Topsoil mantles the site and typically consists of loose, dry to damp, silty and clayey fine sand. Topsoil ranges from one to two feet thick across the site. Remedial grading in the form of removal and recompaction will be required in areas receiving improvements. In addition, topsoil will likely exhibit “medium” expansion characteristics.

3.3 Terrace Deposits (Qtc and Qtg)

Pleistocene-age Terrace Deposits underlie the surficial deposits across the site. This unit typically consists of two relatively distinct layers or “members”. An upper clay layer (Qtc) overlies a lower, coarse-grained, granular layer (Qtg). The upper layer consists of stiff to hard, damp to moist, brown, olive-brown to reddish-brown clay with varying amounts of sand. The clay layer ranges in thickness from approximately four to seven feet thick in the areas explored.

The lower sand-gravel member (Qtg) of consists of dense to very dense, dry to damp, silty and clayey sand with gravel and sandy gravel. Some cobble up to 12-inches in maximum dimension was observed. Thin, concretionary lenses were observed in some test pits.

Both the upper clay member and lower sand-gravel member of the Terrace Deposits possess adequate strength characteristics for support of structures and/or structural fills. However, based on our experience and laboratory testing, the clay member possesses a "medium to very high" expansion potential (EI greater than 51), and poor pavement support characteristics. Expansion indexes of over 200 have been recorded in the clay member on some nearby projects; the highest expansion index obtained from laboratory testing of soils from this site was 129.

4. GROUNDWATER

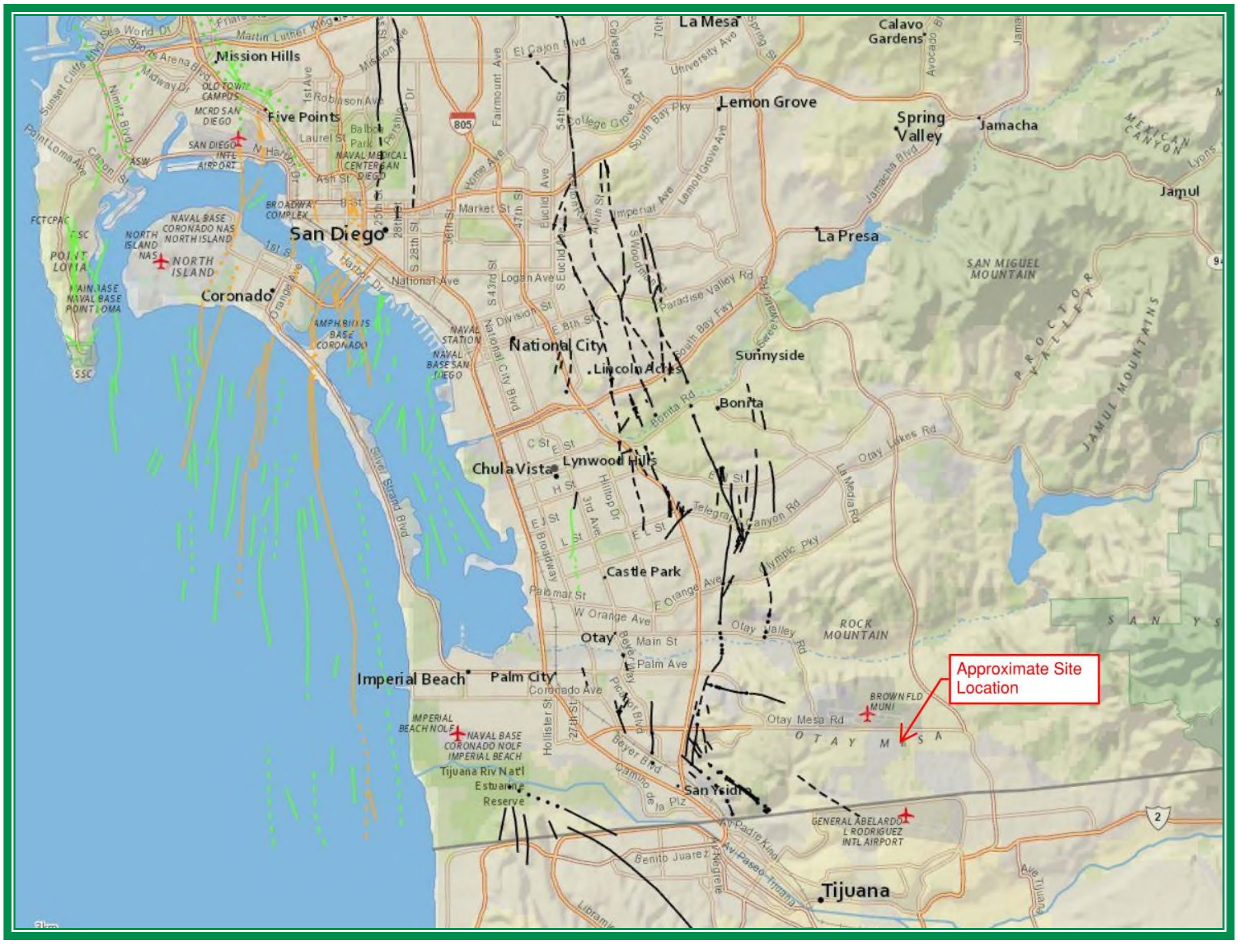
We did not encounter groundwater or seepage during our site investigation. However, it is not uncommon for shallow seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

5. GEOLOGIC HAZARDS

5.1 Faulting and Seismicity

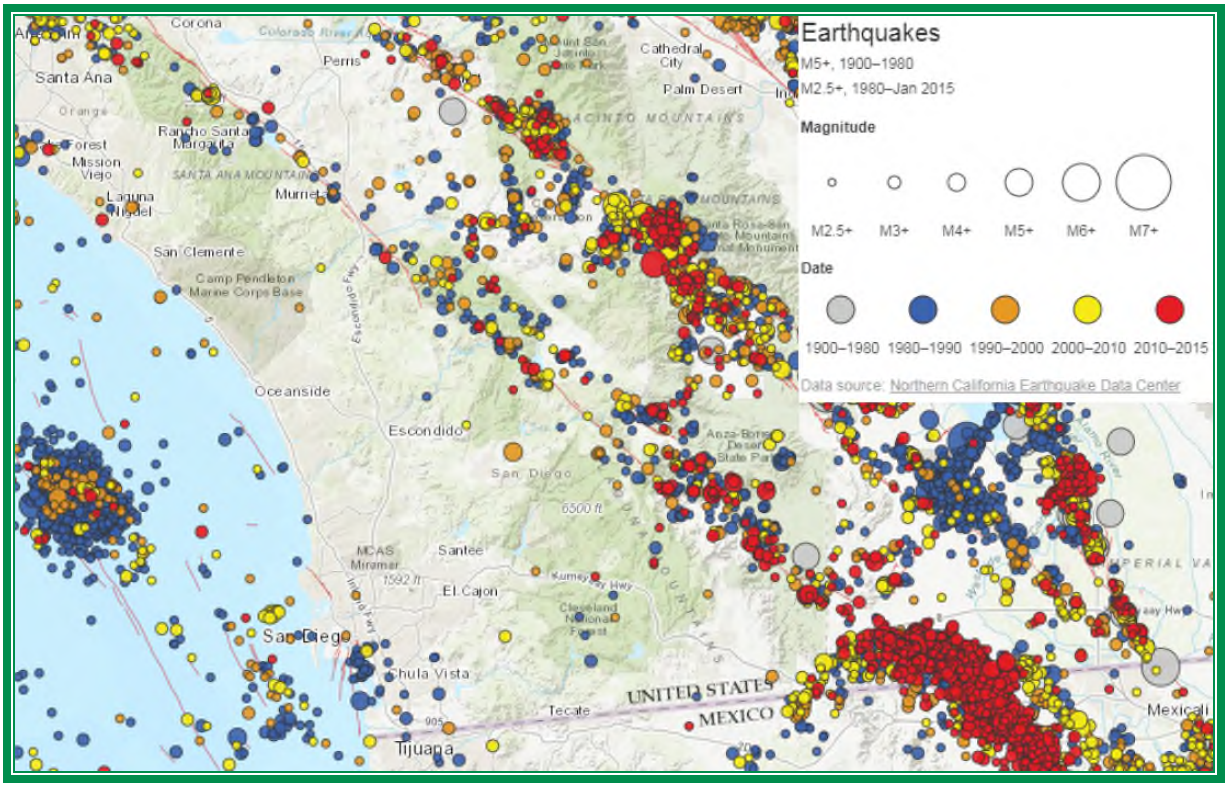
A review of the referenced geologic materials and our knowledge of the general area indicates that the site is not underlain by active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The closest active fault is Newport Inglewood-Rose Canyon Fault zone, located approximately 11 miles west of the site. The site is not located within a State of California Earthquake Fault Zone.

The United States Geological Survey (USGS) has developed a program to evaluate the approximate location of faulting in the area of properties. The following figure shows the location of the existing faulting in the San Diego County and Southern California region. The faults are shown as solid, dashed and dotted traces representing well constrained, moderately constrained and inferred faults, respectively. The fault line colors represent faults with ages less than 150 years (red), 15,000 years (orange), 130,000 years (green), 750,000 years (blue) and 1.6 million years (black).



Faults in the San Diego Area

The San Diego County and Southern California region is seismically active. The following figure presents the occurrence of earthquakes with a magnitude greater than 2.5 from the period of 1900 through 2015 according to the Bay Area Earthquake Alliance website.



Earthquakes in Southern California

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 evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the local agency.

5.2 Ground Rupture

The risk associated with ground rupture hazard is very low due to the absence of active faults at the subject site.

5.3 Storm Surge, Tsunamis, and Seiches

The site is not located near the ocean, therefore the potential of storm surges affecting the site is considered low.

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first-order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). Historically, tsunami wave heights have ranged up to 3.7 feet in the San Diego area. According to the County of San Diego Hazard Mitigation Plan (2010), the largest tsunami effect recorded in San Diego since 1950 was May 22, 1960 which had maximum run-up amplitudes of 2.1 feet (0.7 meters). Wave heights and run-up elevations from tsunamis along the San Diego Coast have historically fallen within the normal range of the tides. The County of San Diego Hazard Mitigation Plan (2010) maps zones of possible tsunami inundation for coastal areas throughout the county. The site is not included within one of these high-risk hazard areas. Therefore, we consider the risk of a tsunami hazard at the site to be low.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located near a lake or embayment, therefore we consider the potential for seiches to impact the site low.

5.4 Flooding

According to maps produced by the Federal Emergency Management Agency (FEMA), the site is zoned as “Zone X – Minimal Flood Hazard.” Based on our review of FEMA flood maps, the risk of site flooding is considered low.

5.5 Liquefaction and Seismically Induced Settlement

Soil liquefaction occurs within relatively loose, cohesionless sand located below the water table that is subjected to ground accelerations from earthquakes. Due to the dense nature of the soils underlying the site, proposed grading, and the lack of permanent, shallow groundwater, there is a very low risk of liquefaction occurring at the site.

5.6 Landslides

The site is relatively flat and lacks sloped topography necessary for landslides to form. Additionally, the published geologic maps do not show landslides on or adjacent to the site. Therefore, we consider the potential for a landsliding on or adjacent to the site is very low.

5.7 Geologic Hazard Category

Review of the 2008 City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Sheet 3, indicates the site is mapped as Geologic Hazard Category 53. Category 53 is described as “Level or sloping terrain, unfavorable geologic structure, low to moderate risk.”

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were observed that would preclude the development of the property as presently proposed provided that the recommendations of this report are followed.

6.1.2 The site is underlain by compressible surficial deposits consisting of topsoil, overlying Pleistocene-age Terrace Deposits. Topsoil ranges from one to two feet thick but may be thicker in unexplored areas of the site. Additionally, significant quantities of soil, trash, and construction debris have been dumped at the site.

6.1.3 Undocumented fill and topsoil are unsuitable in their present condition to receive additional fill soil or settlement-sensitive structures and will require removal and recompaction. The underlying terrace deposits are suitable for support of structural improvements, however, the upper clay portion of the terrace deposits is highly expansive. To reduce the potential for soil heave impacting foundations and site improvements, we recommend selective grading consisting of mining the sand-gravel member of the terrace deposits for use as a pad capping material, in combination with burial of the expansive clay member in mined areas.

6.1.4 We did not encounter groundwater during our subsurface exploration, and groundwater should not be a constraint to project development. However, seepage within surficial soils and formational materials may be encountered during the grading operations, especially during the rainy seasons.

6.1.5 Except for possible strong seismic shaking, no significant geologic hazards were observed or are known to exist on the site that would adversely affect the site. No special seismic design considerations, other than those recommended herein, are required.

6.1.6 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas. Recommendations for site drainage are provided herein.

6.1.7 Based on the results of our field infiltration testing and laboratory testing, we opine full or partial infiltration on the property is infeasible as discussed in Appendix C.

6.1.8 Provided the recommendations of this report are followed, it is our opinion that the proposed development will not destabilize or result in settlement of adjacent properties and City right-of-way.

6.1.9 Subsurface conditions observed may be extrapolated to reflect general soil/geologic conditions; however, some variations in subsurface conditions between trench locations should be anticipated.

6.2 Soil and Excavation Characteristics

6.2.1 In general, special shoring requirements may not be necessary if temporary excavations will be less than 4 feet in height. It is the responsibility of the contractor and their competent person to ensure all excavations, temporary slopes and trenches are properly constructed and maintained in accordance with applicable OSHA guidelines, in order to maintain safety and the stability of the excavations and adjacent improvements. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

6.2.2 Excavation of existing undocumented fill and surficial deposits should be possible with moderate to heavy effort using conventional heavy-duty equipment. We expect excavation of the terrace deposits will require moderate to heavy effort. We expect that gravel, cobbles and cemented zones may be encountered within the terrace deposits requiring very heavy effort to excavate.

6.2.3 The soil encountered in the field investigation is considered to be "expansive" (expansion index [EI] of 20 or greater) as defined by 2019 California Building Code (CBC) Section 1803.5.3. Table 6.2 presents soil classifications based on the expansion index. We expect a majority of the soil encountered in the upper six to ten feet below existing site grades to possess a "medium" to "very high" expansion potential (EI of 51 or greater).

TABLE 6.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2019 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

6.2.4 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Chapter 19. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

6.2.5 We tested samples for potential of hydrogen (pH) and resistivity and chloride to aid in evaluating the corrosion potential. Appendix B presents the laboratory test results. Based on the test results the soils appear to be corrosive.

6.2.6 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements susceptible to corrosion are planned.

6.3 Grading Recommendations

6.3.1 Grading should be performed in accordance with the recommendations provided in this report, the Recommended Grading Specifications contained in Appendix D and the City of San Diego's Grading Ordinance. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during the fill placement.

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

6.3.3 Site preparation should begin with the removal of deleterious material, construction debris, and vegetation. The depth of vegetation removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site. Asphalt and concrete should not be mixed with the fill soil unless approved by the Geotechnical Engineer.

6.3.4 Abandoned foundations and buried utilities (if encountered) should be removed and the resultant depressions and/or trenches should be backfilled with properly compacted material as part of the remedial grading.

6.3.5 We recommend undocumented fill and topsoil be removed and replaced as compacted fill throughout the site. Trash and debris may be encountered in the undocumented fill. Trash and debris, if encountered, should be removed from the fill and exported.

6.3.6 Based on the existing site conditions, we expect grading will result in cuts and fills from existing grade of approximately 7 feet or less to construct the proposed sheet grades. Because of the limited depth of fills to reach pad grade, we expect grading will result in expansive clay soils near finish grade elevations. Therefore, we recommend select grading occur to provide a 5-foot-thick cap of *low*- to *medium*-expansive soil. To obtain select capping material, we recommend mining the underlying *low*- to *medium*-expansive, granular member of the Terrance Deposit (Qtg), which is suitable for site capping, in combination with burial of the expansive clay member in mined areas, as described below.

6.3.7 Within structural improvement areas (building pads, parking lots, etc.) we recommend grading to provide a select pad cap that extends at least 5-feet below pad grade and to a minimum of at least 3-feet below bottom of footing elevation, whichever is deeper. Pad-cap elevation should be adjusted for loading dock ramps and wall footings, which are typically lower than the building pad grade. Based on our experience with nearby sites, the sand-gravel can be mined to depths up to 30-feet below existing site grades. The approximate depth to the sand-gravel terrace deposits soil is shown on Figure 2 next to each trench location.

6.3.8 Mined areas should be selected so as to not create a fill differential greater than 15 feet within the building pad, if possible.

6.3.9 Within the building pad, the remedial excavation should extend to a horizontal distance beyond the building pad limits of 5 feet or to a distance equal to the depth of the excavation, whichever is greater. Excavations outside of the building pad should extend to at least 5 feet beyond the structural improvement limits.

6.3.10 The bottom of the excavations (including mining excavations) should be sloped one percent to the adjacent street or deepest fill. Prior to fill soil being placed, the existing ground surface should be scarified, moisture conditioned as necessary, and compacted to a depth of at least 12 inches. A representative of Geocon should be on-site during removals to evaluate the limits of the remedial grading.

6.3.11 The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use from a geotechnical engineering standpoint as fill if relatively free from vegetation, debris and other deleterious material. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM Test Procedure D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

6.3.12 Import fill (if necessary) should consist of the characteristics presented in Table 6.3. 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.

TABLE 6.3
SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" to "Low" (Expansion Index of 50 or less)
Particle Size	Maximum Dimension Less Than 3 Inches
	Generally Free of Debris

6.4 Seismic Design Criteria

6.4.1 Table 6.4.1 summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 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 of California (SEAOC) 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 2019 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 6.4.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.717g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.267g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.213	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}	0.87g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.401g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.58g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.267g*	Section 1613.2.4 (Eqn 16-39)

***Note:** Using the code-based values presented in this table, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed by the project structural engineer. Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis should be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed.

6.4.2 Table 6.4.2 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 6.4.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.312g	Figure 22-7
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.374g	Section 11.8.3 (Eqn 11.8-1)

6.4.3 Conformance to the criteria in Tables 6.4.1 and 6.4.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4.4 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 6.4.3 presents a summary of the risk categories.

TABLE 6.4.3
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

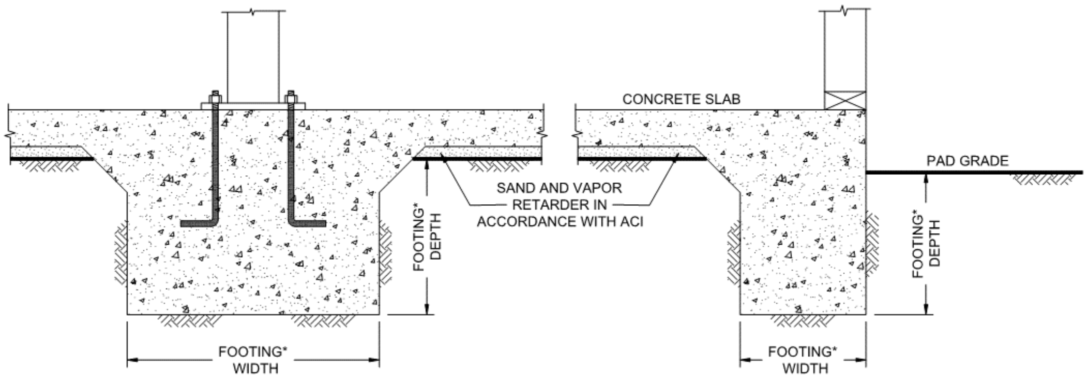
6.5 Shallow Foundations

6.5.1 The proposed structure can be supported on a shallow foundation system founded in compacted fill provided the grading recommendations provided in Section 6.4 are followed. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Table 6.5 provides a summary of the foundation design recommendations.

TABLE 6.5
SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width	12 inches
Minimum Isolated Foundation Width	24 inches
Minimum Foundation Depth	24 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
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	3,500 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	9-Foot Square
Design Expansion Index	50 or less

6.5.2 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.



Wall/Column Footing Dimension Detail

6.5.3 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.

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

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.
- 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 that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

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

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

6.6 Conventional Retaining Wall Recommendations

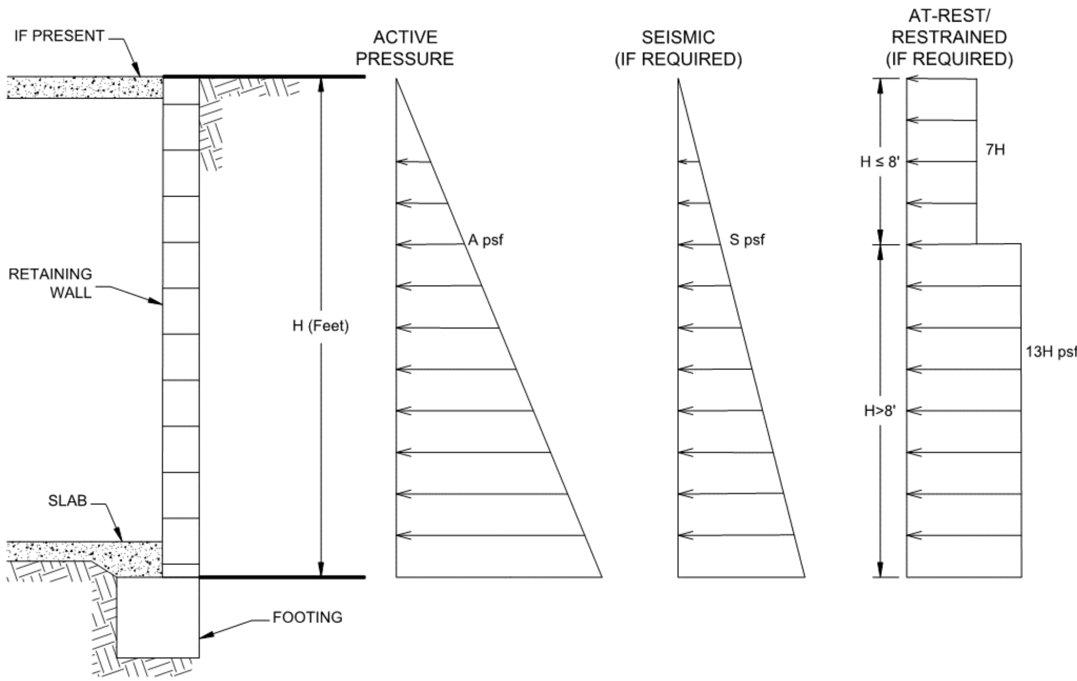
6.6.1 Retaining walls should be designed using the values presented in Table 6.6.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 6.6.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value	
	EI≤50	EI≤90
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf	45 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	50 psf	60 pcf
Seismic Pressure, S	15H psf	
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf	
At-Rest/Restrained Walls Additional Uniform Pressure (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

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



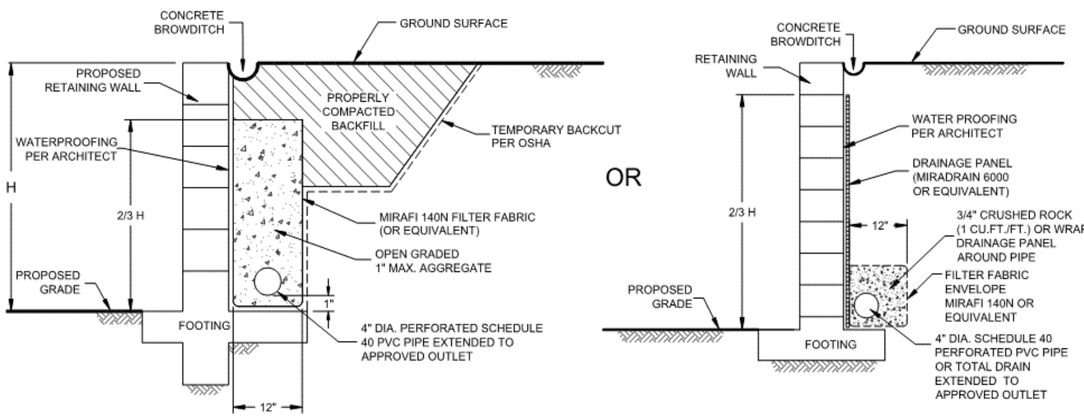
Retaining Wall Loading Diagram

6.6.3 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 of 7H psf should be added to the active soil pressure for walls 8 feet or less. For walls greater than 8 feet tall, an additional uniform pressure of 13H psf should be applied to the wall starting at 8 feet from the top of the wall to the base of 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.

6.6.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.2.5 of the 2019 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 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 18H psf should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.374g calculated from ASCE 7-16 Section 11.8.3 and applied a pseudo-static coefficient of 0.3.

6.6.5 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.

6.6.6 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

6.6.7 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.

6.6.8 In general, wall foundations having should be designed in accordance with Table 6.6.2. 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 6.6.2
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
Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per additional foot of footing depth
	300 psf per additional foot of footing width
Maximum Bearing Capacity	4,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

6.6.9 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, soil nail walls, or soldier pile walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

6.6.10 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.

6.6.11 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.

6.7 Lateral Loading

6.7.1 Table 6.7 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 design for passive resistance. Where walls are planned adjacent to and/or on descending slopes, a passive pressure of 150 pcf should be used in design.

TABLE 6.8
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	350 pcf
Passive Pressure Fluid Density Adjacent to and/or on Descending Slopes	150 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

6.7.2 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.

6.8 Preliminary Pavement Recommendations

6.8.1 Preliminary pavement recommendations for the streets and parking areas are provided below. The final pavement sections should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. For pavement design we used an R-Value of 16 based on laboratory testing of a sample of soil taken during our field investigation. Preliminary flexible pavement sections for varying traffic indices are presented in Table 6.8.1. The project civil engineer or traffic engineer should determine the appropriate Traffic Index (TI) or traffic loading expected on the project for the various pavement areas that will be constructed.

TABLE 6.8.1
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete (inches)	Class 2 Base (inches)
4.5	3	6
5	3	8
5.5	3	10
6	3.5	10.5
6.5	3.5	12
7	4	13
7.5	4.5	13.5
8	5	14.5

6.8.2 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.

6.8.3 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. 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 6.8.2.

TABLE 6.8.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 and 300

6.8.4 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 6.8.3.

TABLE 6.8.3
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A)	5.5
Driveways (TC=C)	7.5

6.8.5 The PCC vehicular 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.

6.8.6 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 6.8.4.

TABLE 6.8.4
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Spacing	30 Times Slab Thickness
	Max. Spacing of 12 feet for 5.5-Inch-Thick
	Max. Spacing of 15 Feet for Slabs 6 Inches and Thicker
Crack Control Joint Depth	Per ACI 330R-08
	1 Inch Using Early-Entry Saws on Slabs Less Than 9 Inches Thick
Crack Control Joint Width	¼-Inch for Sealed Joints
	⅜-Inch is Common for Sealed Joints
	1/10- to 1/8-Inch is Common for Unsealed Joints

6.8.7 Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

6.8.8 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 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.

6.8.9 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed as recommended in

Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

6.8.10 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 that receives vehicular 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, or cross-gutters so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

6.9 Exterior Concrete Flatwork

6.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 6.9. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 6.9
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI $\leq$ 90	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	
EI $\leq$ 130	4x4-W4.0/W4.0 (4x4-4/4) welded wire mesh	
	No. 4 Bars 12 inches on center, Both Directions	

*In excess of 8 feet square.

6.9.2 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement 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.

6.9.3 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. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted, and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

6.9.4 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

6.9.5 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

6.10 Slope Maintenance

6.10.1 Slopes that are steeper than 3:1 (horizontal:vertical) may, under conditions which are both difficult to prevent and predict, be susceptible to near surface (surficial) slope instability. The instability is typically limited to the outer three 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.

6.11 Storm Water Management

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

6.11.2 We performed an infiltration study on the property. A summary of our study and storm water management recommendations are provided in Appendix C. Based on the results of our study, full and partial infiltration is considered infeasible due to slow infiltration characteristics of the on-site soil. Basins should utilize a liner to prevent infiltration from causing adverse settlement and heave, and migrating to utilities, and foundations.

6.12 Site Drainage and Moisture Protection

6.12.1 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 2019 CBC 1803.3 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.

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

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

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

6.13 Grading and Foundation Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

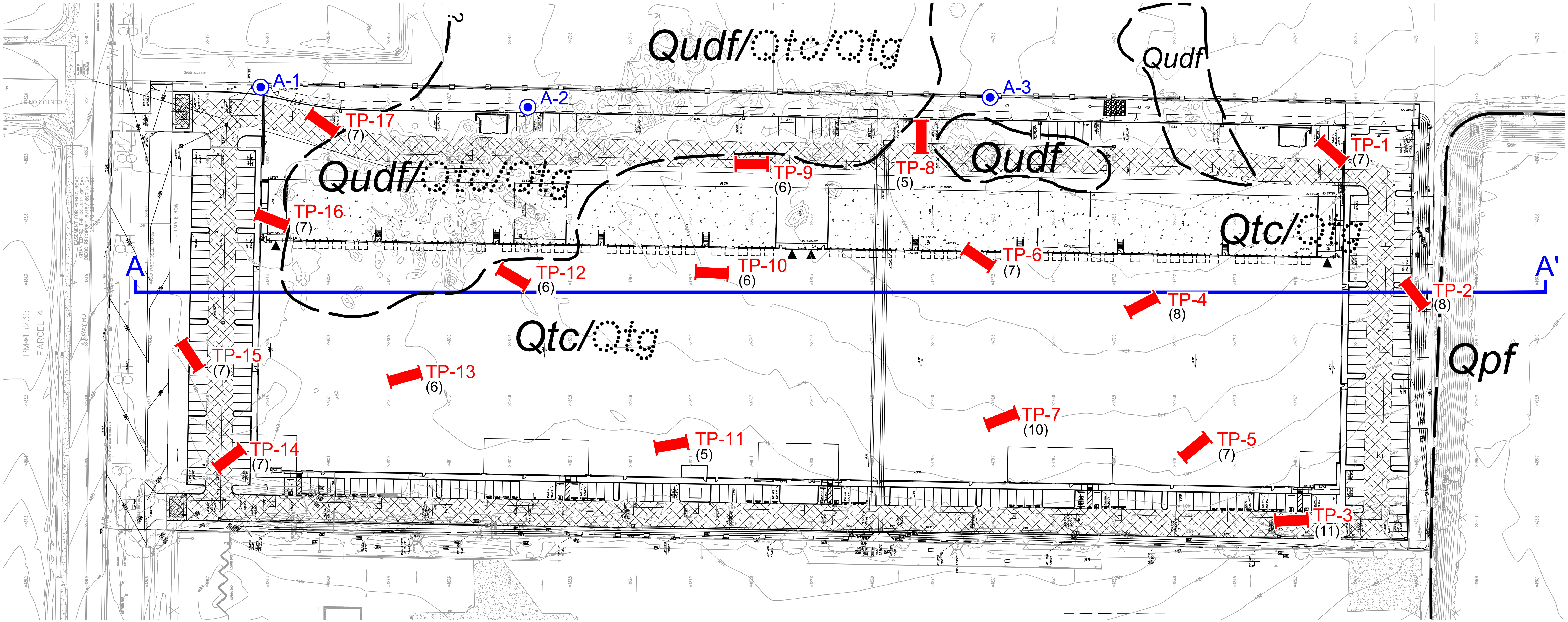
4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards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

VICINITY MAP

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

OTAY MESA
AIRWAY ROAD
SAN DIEGO, CALIFORNIA

RA / CW		DSK/GTYPD	DATE 05 - 18 - 2020	PROJECT NO. G2467 - 42 - 01	FIG. 1



GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL (Soil, Trash and Construction Debris)

Qpf ....PREVIOUSLY PLACED FILL

Qtc ....TERRACE DEPOSITS, CLAY MEMBER (Dotted Where Buried)

Qtg ....TERRACE DEPOSITS, SAND-GRAVEL MEMBER (Dotted Where Buried)

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

T-17 ....APPROX. LOCATION OF TRENCH

A-3 ....APPROX. LOCATION OF INFILTRATION TEST

(7) ....APPROX. DEPTH TO TOP OF TERRACE DEPOSIT (SANDS-GRAVEL MEMBER)

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

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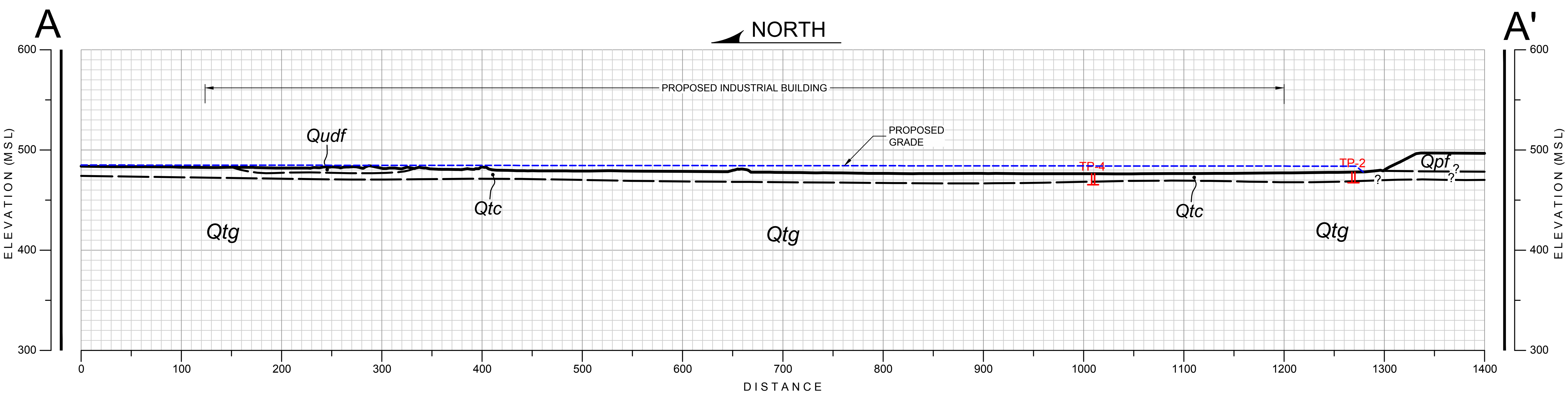
PROJECT NO. G2467 - 42 - 01

FIGURE 2

GEOLOGIC MAP

DATE 05 - 18 - 2020

OTAY MESA
AIRWAY ROAD
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 100' (Vert. = Horiz.)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL
(Soil, Trash and Construction Debris)

Qpf ....PREVIOUSLY PLACED FILL

Qtc ....TERRACE DEPOSITS, CLAY MEMBER

Qtg ....TERRACE DEPOSITS, SAND-GRAVEL MEMBER

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

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

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PROJECT NO. G2467 - 42 - 01
FIGURE 3
DATE 05 - 18 - 2020

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on November 15, 2019. Our investigation consisted of a site reconnaissance, and logging of 17 exploratory test pits. The exploratory test pits were excavated to depths between 8 and 13 feet. We also performed three infiltration borings on March 6, 2020, which were excavated with a CME 75 truck-mounted drill rig. The approximate locations of the exploratory test pits and infiltration tests are shown on Figure 2.

The soil conditions encountered in the trenches were visually examined, classified, and logged in general conformance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). Exploratory test pit logs are presented on Figures A-1 through A-17. The logs depict the various soil types encountered and indicate the depths at which samples were obtained.

TRENCH TP 1

ELEV. (MSL.) +/-478' DATE COMPLETED 11-15-2019

EQUIPMENT JOHN DEERE 310L BY: R. ADAMS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**TOPSOIL** Loose, damp, dark brown, Clayey, medium to coarse SAND; trace cobble fragments			
2								
				CL	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Firm to stiff, damp to moist, dark brown to reddish-brown, Sandy CLAY			
4								
	TP1-1			SC	Medium dense, dry to damp, orange-brown to reddish-brown, Clayey, medium coarse SAND; blocky texture, trace caliche			
6					-At 6 feet: becomes dense, some subrounded cobble up to 6-inch in width			
				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense, damp, orange-brown to pale yellowish brown, fine to medium Sandy GRAVEL; gravel/cobble subrounded up to 10-inches in width. Difficult excavation			
8								
10								
					PRACTICAL REFUSAL 11 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

TRENCH TP 2

ELEV. (MSL.) +/-481' DATE COMPLETED 11-15-2019

EQUIPMENT JOHN DEERE 310L BY: R. ADAMS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**TOPSOIL** Loose, dry, grayish-brown, Clayey, fine to medium SAND; trace silt			
2	TP2-1			CL/CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Firm to stiff, damp, dark brown to dark reddish brown, fine to medium Sandy CLAY			
4	TP2-2							
6				SC	Medium dense to dense, dry to damp, reddish-brown, Clayey, medium to coarse SAND; blocky texture, manganese staining, trace caliche -At 7.5 feet: becomes very dense; difficult excavation			
8				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Very dense, dry to damp, reddish brown to yellowish brown, Silty GRAVEL; some fine to coarse sand, sub-rounded cobble up to 8-inch in width			

TRENCH DEPTH 9 FEET PRACTICAL REFUSAL
No groundwater
Backfilled 11-15-2019

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

G2467-42-01.GPJ

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

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

GEOCON

PROJECT NO. G2467-42-01

TRENCH TP 3

ELEV. (MSL.) **+/-485'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	TP3-1			SM	**TOPSOIL** Loose, dry, grayish-brown, Silty, fine SAND; trace gravel up to 1-inch in width			
2				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff to hard, dry to damp, dark brown, fine, Sandy CLAY; few rootlets			
					-At 3 feet: becomes reddish-brown (mottled), blocky, some caliche			
4	TP3-2							
6								
				SC	Dense to very dense, reddish brown, Clayey SAND; some caliche			
8	TP3-3				-At 8 feet: becomes very coarse			
10								
				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Very dense, damp, reddish-brown, Sandy GRAVEL; trace silt, gravel up to 2 inches, few cobble up to 8 inches in width			
12					TRENCH DEPTH 12 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

SAMPLE SYMBOLS

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

GEOCON

PROJECT NO. G2467-42-01

TRENCH TP 4

ELEV. (MSL.) **+/-480'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM/SC	**TOPSOIL** Loose, dry, grayish-brown, Silty, fine SAND; some clay			
	TP4-1			CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Firm to stiff, moist, dark brown to reddish brown CLAY; some caliche			
2								
4					-At 4 feet: abundant caliche			
				CL	Stiff to very stiff, damp, orange brown, medium coarse Sandy CLAY; some caliche			
6								
8	TP4-2			GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense, damp, fine to coarse Sandy GRAVEL; trace clay, gravel up to 4 inches, few cobble up to 9 inches in width			
10					TRENCH DEPTH 10 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

TRENCH TP 5

ELEV. (MSL.) **+/-482'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, grayish-brown, Silty, fine to medium SAND			
				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Firm, damp, dark brown, CLAY; trace sand, trace caliche			
2								
					-At 3 feet: becomes reddish-brown			
4					-At 4 feet: becomes moist to wet			
				SM	Dense, moist, orange-brown, Silty, fine to medium SAND; some gravel up to 4 inches, few cobble up to 10 inches; trace silt			
6								
				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense, moist, orange-brown, medium coarse, Sandy GRAVEL			
8					TRENCH DEPTH 8 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

GEOCON

TRENCH TP 6

ELEV. (MSL.) +/-479' DATE COMPLETED 11-15-2019

EQUIPMENT JOHN DEERE 310L BY: R. ADAMS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	Loose, dry, grayish-brown, Silty, fine to medium SAND; trace caliche			
2				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Firm to stiff, damp to moist, dark brown, medium coarse, Sandy CLAY; weak blocky texture -At 3 feet: becomes dark reddish-brown			
4	TP6-1			CL	Firm, damp to moist, reddish-brown to brown, medium to coarse Sandy CLAY			
6								
				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense, damp to moist, medium coarse Sandy GRAVEL; some cobble up to 10 inch in width, some clay			
8								
10								
					TRENCH DEPTH 10.5 FEET No groundwater encountered Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

GEOCON

PROJECT NO. G2467-42-01

TRENCH TP 7

ELEV. (MSL.) **+/-482'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, grayish-brown to reddish-brown, Silty, fine to medium SAND; few rootlets			
2				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Firm to stiff, damp, dark brown, CLAY; trace medium sand			
4					-At 3.5 feet: becomes reddish-brown, some caliche			
6				SC	Dense, damp to moist, reddish-brown to pale yellowish-brown Clayey, fine to coarse SAND; some gravel up to 8 inch in width			
8								
10				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Very dense, damp, yellowish-brown, medium coarse Sandy GRAVEL; few cobble up to 8 inches			
					-At 11 feet: becomes weakly moderately cemented-concretion lens			
12								
					TRENCH DEPTH 13 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

GEOCON

PROJECT NO. G2467-42-01

TRENCH TP 8

ELEV. (MSL.) +/-479' DATE COMPLETED 11-15-2019

EQUIPMENT JOHN DEERE 310L BY: R. ADAMS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				GM	**TOPSOIL** Loose, dry, grayish-brown, SIlty, fine to medium SAND; few rootlets			
2				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff, damp, reddish-brown to brown CLAY; trace medium to coarse sand; trace subrounded cobble up to 6 inches in width			
4								
6				SC	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense, damp, orange-brown Clayey, medium to coarse SAND; little gravel up to 3 inches; few cobble up to 10 inches in width			
					TRENCH DEPTH 6.5 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

GEOCON

TRENCH TP 9

ELEV. (MSL.) **+/-481'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, grayish brown to brown, Silty, fine to medium SAND; trace rootlets			
				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff, damp to moist, brown to reddish-brown CLAY			
2	TP9-1							
4								
6				SP	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Medium dense, damp, orange-brown to grayish brown, fine to medium SAND			
8				SP	Medium dense to dense, damp to moist, whitish-gray to orange-brown, fine to coarse SAND; some gravel up to 4 inches, trace cobble up to 8 inches			
				GM	Dense, damp, orange-brown, medium coarse, Sandy GRAVEL			
10					TRENCH DEPTH 10 FEET, PRACTICAL REFUSAL No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2467-42-01

TRENCH TP 10

ELEV. (MSL.) **+/-482'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, grayish-brown, Silty, fine to medium SAND			
				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff, damp, dark brown to reddish-brown, CLAY; trace, medium coarse sand			
2								
4								
				SC	Dense, damp, reddish-brown, Clayey medium coarse SAND; some gravel up to 3 inch in width			
6				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Very dense, dry to damp, pale yellowish-brown, fine to coarse Sandy GRAVEL; trace cobble up to 8 inch in width			
8					TRENCH DEPTH 8 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

GEOCON

PROJECT NO. G2467-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH TP 11** ELEV. (MSL.) **+/-484'** DATE COMPLETED **11-15-2019** EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Loose, dry, gray to grayish-brown, Silty, fine SAND; trace clay			
				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff to very stiff, damp, dark reddish-brown CLAY; trace coarse sand			
2								
4					-At 4 feet: becomes moist, trace cobble up to 8 inch in width; trace caliche stringers			
				GC	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Very stiff to hard, damp, brown to orange brown, Clayey GRAVEL; some medium to coarse sand			
6								
				GM	Dense to very dense, damp, orange-brown, medium to coarse Sandy GRAVEL; trace clay, few cobble up to 12 inch in width			
8					PRACTICAL REFUSAL 8 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

GEOCON

TRENCH TP 12

ELEV. (MSL.) **+/-483'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, gray to grayish-brown, Silty, fine SAND; trace clay			
2				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff, damp, dark reddish brown CLAY; trace sand, some caliche			
4	TP12-1			CL	Stiff to hard, dry to damp, brown to orange brown, medium coarse Sandy CLAY			
6				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense to very dense, damp, medium coarse, Sandy GRAVEL; trace clay, few subrounded cobble up to 10 inch in width			
8								
10					-At 10 feet: becomes medium dense, moist			
					TRENCH DEPTH 11.5 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

GEOCON

PROJECT NO. G2467-42-01

TRENCH TP 13

ELEV. (MSL.) **+/-484'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, gray to grayish-brown, Silty, fine SAND; trace clay			
1 – 2				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Firm to stiff, damp, dark brown to dark reddish-brown, CLAY; trace medium coarse sand, few caliche stringers			
3 – 4				SC	Medium dense to dense, dry to damp, orange-brown, Clayey, medium to coarse SAND; blocky texture			
6				SP	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Medium dense to dense, damp, orange brown, medium to coarse SAND; trace clay			
8 – 10				GM	Dense, damp, orange-brown, medium to coarse, Sandy GRAVEL; trace subrounded cobble up to 8 inch in width			
					TRENCH DEPTH 11.5 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

GEOCON

PROJECT NO. G2467-42-01

TRENCH TP 14

ELEV. (MSL.) **+/-485'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, gray to grayish-brown, Silty, fine SAND; trace clay			
				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff to hard, damp, brown to reddish-brown, medium to coarse Sandy CLAY; caliche stringers, manganese staining on parting surfaces			
2								
4								
				SC	Dense to very dense, damp, orange-brown Clayey, medium coarse SAND			
6								
				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Very dense, damp, orange-brown to reddish brown, medium coarse, Sandy GRAVEL			
8								
				GC	Very dense, damp, orange-brown, Clayey GRAVEL; some medium to coarse sand			
10								

TRENCH DEPTH 11 FEET
No groundwater
Backfilled 11-15-2019

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

G2467-42-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

TRENCH TP 15

ELEV. (MSL.) +/-486' DATE COMPLETED 11-15-2019

EQUIPMENT JOHN DEERE 310L BY: R. ADAMS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, gray to grayish-brown, Silty, fine SAND; trace clay			
2				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff, damp, brown to reddish-brown CLAY; trace coarse sand, some caliche stringers			
				CH	Stiff to very stiff, damp, reddish-brown to orange brown, medium to coarse Sandy CLAY; blocky texture, clay films on parting surfaces			
4								
6								
8				SC	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense, damp, orange-brown, Clayey, medium coarse SAND			
10				GM	Very dense, damp, orange-brown to reddish brown, medium coarse Sandy GRAVEL; trace cobble up to 12 inch in width			

PRACTICAL REFUSAL 11 FEET
No groundwater
Backfilled 11-15-2019

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

G2467-42-01.GPJ

SAMPLE SYMBOLS

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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH TP 16 ELEV. (MSL.) +/-486' DATE COMPLETED 11-15-2019 EQUIPMENT JOHN DEERE 310L BY: R. ADAMS	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Loose, dry to damp, grayish-brown, Silty, fine to medium SAND			
2	TP16-1			CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Stiff, damp, brown to reddish-brown CLAY; manganese staining on parting surfaces, abundant caliche; trace subrounded gravel up to 4 inch in width			
4	TP16-2			CH	Stiff, damp, orange to reddish-brown, medium to coarse Sandy CLAY			
6								
8				GM	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense to very dense, damp, orange-brown, medium coarse Sandy GRAVEL; trace clay, trace cobble			
					TRENCH DEPTH 9 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

GEOCON

TRENCH TP 17

ELEV. (MSL.) **+/-484'** DATE COMPLETED **11-15-2019**

EQUIPMENT **JOHN DEERE 310L** BY: **R. ADAMS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, grayish-brown, Silty fine SAND; trace clay			
2				CH	**TERRACE DEPOSITS-CLAY MEMBER (Qtc)** Firm to stiff, damp, grayish-brown to brown CLAY; trace coarse sand, abundant caliche			
4					-At 4 feet: trace gravel			
6				CH	Stiff, damp to moist, orange-brown, medium coarse Sandy CLAY; trace caliche			
8				SW	**TERRACE DEPOSITS-SAND-GRAVEL MEMBER (Qtg)** Dense, damp, whitish orange, medium coarse SAND; trace clay, few gravel, trace cobble			
				GP	Dense to very dense, damp, orange-brown, medium coarse, Sandy GRAVEL; few cobble up to 12 inch in width			
10					TRENCH DEPTH 10 FEET No groundwater Backfilled 11-15-2019			

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

G2467-42-01.GPJ

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

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

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APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for *in-situ* dry density and moisture content, maximum dry density and optimum moisture content, expansion potential, soluble sulfate content, chloride content, p.H. and resistivity, and resistance value (R-Value). The results of these tests are summarized on Tables B-I through B-VI.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-02

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T4-2	Reddish brown Silty Gravel (GM); some (f-c) sand, trace clay	133.7	7.9
T12-1	Brown Clayey Silt (ML); trace (f-c) sand	115.8	13.0

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-03

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T3-1	8.7	16.2	116.2	22
T4-1	12.2	30.9	101.9	129
T4-2	8.2	16.1	116.6	16
T6-1	8.4	16.9	115.9	48
T12-1	12.9	27.6	101.2	76

TABLE B-III
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate (%)	Sulfate Exposure
T4-2	0.039	S0
T12-1	0.069	S0

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE CHLORIDE ION CONTENT TEST RESULTS AASHTO TEST NO. T 291

Sample No.	Chloride Ion Content ppm (%)
T4-2	660 (0.066)

TABLE B-V
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (PH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST METHOD 643

Sample No.	pH	Minimum Resistivity (ohm-centimeters)
T4-2	7.0	450

TABLE B-VI
SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS ASTM D 2844

Sample No.	Depth (Feet)	Description (Geologic Unit)	R-Value
T4-2	8-10	Reddish brown Silty Gravel (GM); some (f-c) sand, trace clay	16

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT

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

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table C-1 presents the descriptions of the hydrologic soil groups. 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 undocumented fill, surficial deposits such as topsoil, and Terrace Deposits. Table C-2 presents the information from the USDA website for the subject property.

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

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group
Huerhuero loam, 2 to 9 percent slopes	HrC	93	D
Stockpen gravelly clay loam, 2 to 5 percent slopes	SuB	7	D

Infiltration Testing

We performed three infiltration tests at the locations shown on Figure 2. The tests were performed in 6-inch-diameter, drilled borings or in a hand-auger boring. Table C-3 presents the results of the testing. The calculation sheets are also attached.

We used the guidelines presented in the Riverside County Low Impact Development BMP Design Handbook. Based on this widely accepted guideline, the saturated hydraulic conductivity (Ksat) is equivalent to the infiltration rate. Therefore, the Ksat value determined from our testing is assumed to be the unfactored infiltration rate.

TABLE C-3
UNFACTORED, FIELD-SATURATED, INFILTRATION TEST RESULTS

Test No.	Depth (inches)	Geologic Unit	Field Infiltration Rate, I (in/hr)	Factored* Field Infiltration Rate, I (in/hr)
A-1	39	Qtc	0.003	0.001
A-1a	46	Qtc	0.005	0.003
A-2	33	Qtc	0.121	0.060
A-2a	25	Topsoil/Qtc	0.054	0.027
A-3	24	Topsoil/Qtc	0.004	0.002
A-3a	36	Qtc	0.023	0.012

* Factor of Safety of 2.0 for feasibility determination.

STORM WATER MANAGEMENT CONCLUSIONS

Soil Types

Undocumented Fill (Qudf) – We encountered undocumented fill dumped at existing grade some portions of the site. The undocumented fill within structural improvement areas will be fully or partially removed and replaced with compacted fill. Water that is allowed to migrate into the undocumented fill will cause settlement. Therefore, full and partial infiltration should be considered infeasible within undocumented fill.

Topsoil (Unmapped) – We encountered topsoil varying between 1 to 2 feet thick across the site. Topsoil within structural improvement areas will be removed and replaced with compacted fill. Water that is allowed to migrate into the topsoils may cause settlement. Therefore, full and partial infiltration should be considered infeasible within topsoil.

Terrace Deposits (Qtc/Qtg) – We encountered approximately 6 to 8 feet of stiff clay and sandy clay overlying dense to very dense clayey sand and sandy gravel. Infiltration into terrace deposits is not feasible due to low infiltration characteristic and high expansion potential.

Groundwater Elevation

Groundwater was not encountered in our test pits to a depth of 13 feet below the existing ground surface. Infiltration should not impact groundwater.

Existing Utilities

No known utilities cross the site. Infiltration due to utility concerns would be feasible.

Soil or Groundwater Contamination

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

Slopes

There are no existing slopes that would be impacted by infiltration.

Infiltration Rates

Our test results indicated very slow infiltration rates. The rates were 0.003 and 0.121 in/hr. The average rate is 0.035 in/hr with a factored rate for feasibility determination of 0.018 in/hr. The infiltration rates are not high enough to support full or partial infiltration.

Storm Water Management Devices

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

Storm Water Standard Worksheets

The SWS 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 Form D.5-1) that helps the project civil engineer estimate the factor of safety based on several factors. Table C-4 describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE C-4
SUITABILITY ASSESSMENT RELATED CONSIDERATIONS FOR INFILTRATION FACILITY SAFETY FACTORS

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

Table C-5 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.

TABLE C-5
FACTOR OF SAFETY WORKSHEET D.5-1 DESIGN VALUES[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	3	0.75
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \Sigma p$			2.0

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

CONCLUSIONS

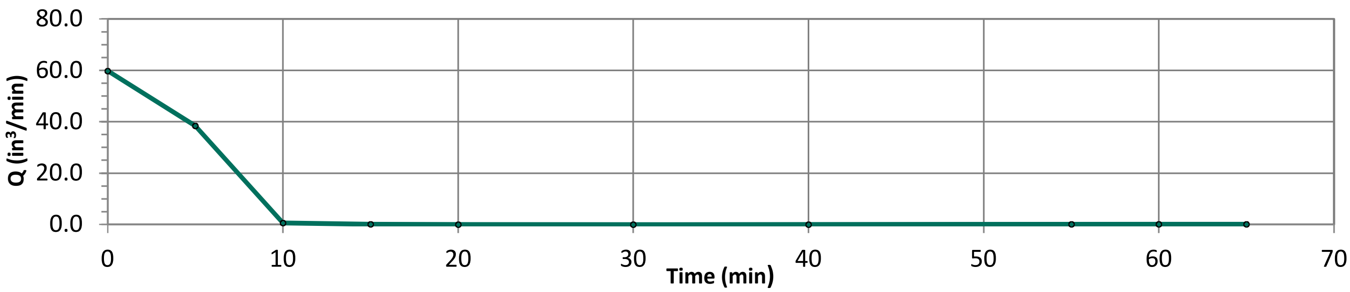
Our results indicate the site has relatively slow infiltration characteristics. Because of the site conditions, it is our opinion that there is a potential for lateral water migration. Undocumented and exists on the property and has a high potential for adverse settlement when wetted. It is our opinion that full or partial infiltration is infeasible on this site. Our evaluation included the soil and geologic conditions, estimated settlement and volume change of the underlying soil, slope stability, utility considerations, groundwater mounding, retaining walls, foundations and existing groundwater elevations.

TEST NO.: **A-1** GEOLOGIC UNIT: **Qvop**

EXCAVATION ELEVATION (MSL, FT): **478**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	6
BOREHOLE DEPTH (FT):	3.3
TEST/BOTTOM ELEVATION (MSL, FT):	475
MEASURED HEAD HEIGHT (IN):	5.3
CALCULATED HEAD HEIGHT (IN):	5.5
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.028**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.003**
FACTORED INFILTRATION RATE (IN/HR):	**0.001**



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	10.800	299.08	59.815
3	5.00	6.940	192.18	38.437
4	5.00	0.105	2.91	0.582
5	5.00	0.005	0.14	0.028
6	10.00	0.005	0.14	0.014
7	10.00	0.005	0.14	0.014
8	15.00	0.005	0.14	0.009
9	5.00	0.005	0.14	0.028
10	5.00	0.005	0.14	0.028
11	5.00	0.005	0.14	0.028

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AARDVARK PERMEAMETER TEST RESULTS

OTAY MESA AIRWAY ROAD

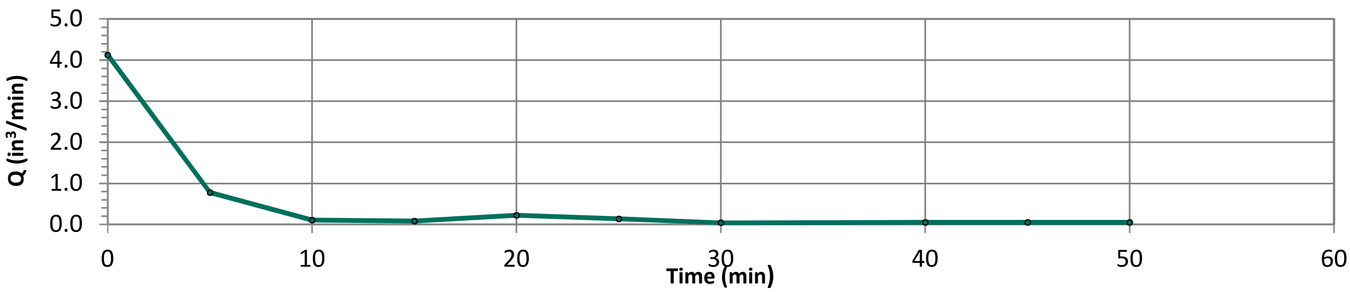
PROJECT NO.: G2467-42-01

TEST NO.: **A-1a** GEOLOGIC UNIT: **Qvop**

EXCAVATION ELEVATION (MSL, FT): **478**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	4
BOREHOLE DEPTH (FT):	3.8
TEST/BOTTOM ELEVATION (MSL, FT):	474
MEASURED HEAD HEIGHT (IN):	3.0
CALCULATED HEAD HEIGHT (IN):	4.1
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.028**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.005**
FACTORED INFILTRATION RATE (IN/HR):	**0.003**



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	0.745	20.63	4.126
3	5.00	0.140	3.88	0.775
4	5.00	0.020	0.55	0.111
5	5.00	0.015	0.42	0.083
6	5.00	0.040	1.11	0.222
7	5.00	0.025	0.69	0.138
8	10.00	0.015	0.42	0.042
9	5.00	0.010	0.28	0.055
10	5.00	0.010	0.28	0.055
11	5.00	0.010	0.28	0.055
12	5.00	0.010	0.28	0.055
13	5.00	0.005	0.14	0.028
14	5.00	0.005	0.14	0.028
15	5.00	0.005	0.14	0.028

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AARDVARK PERMEAMETER TEST RESULTS

OTAY MESA AIRWAY ROAD

PROJECT NO.: **G2467-42-01**

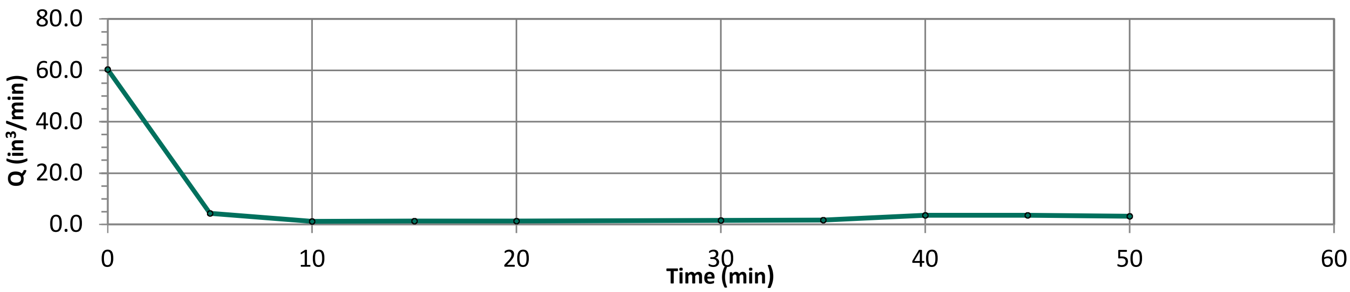
TEST NO.: **A-2**

GEOLOGIC UNIT: **Qvop**

EXCAVATION ELEVATION (MSL, FT): **483**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	6
BOREHOLE DEPTH (FT):	2.8
TEST/BOTTOM ELEVATION (MSL, FT):	480
MEASURED HEAD HEIGHT (IN):	4.3
CALCULATED HEAD HEIGHT (IN):	4.9
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**1.052**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.121**
FACTORED INFILTRATION RATE (IN/HR):	**0.060**



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	10.895	301.71	60.342
3	5.00	0.790	21.88	4.375
4	5.00	0.215	5.95	1.191
5	5.00	0.235	6.51	1.302
6	10.00	0.500	13.85	1.385
7	5.00	0.285	7.89	1.578
8	5.00	0.300	8.31	1.662
9	5.00	0.655	18.14	3.628
10	5.00	0.645	17.86	3.572
11	5.00	0.580	16.06	3.212
12	5.00	0.325	9.00	1.800
13	5.00	0.275	7.62	1.523
14	5.00	0.205	5.68	1.135
15	5.00	0.215	5.95	1.191
16	5.00	0.135	3.74	0.748
17	5.00	0.215	5.95	1.191
18	5.00	0.195	5.40	1.080

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AARDVARK PERMEAMETER TEST RESULTS

OTAY MESA AIRWAY ROAD

PROJECT NO.: G2467-42-01

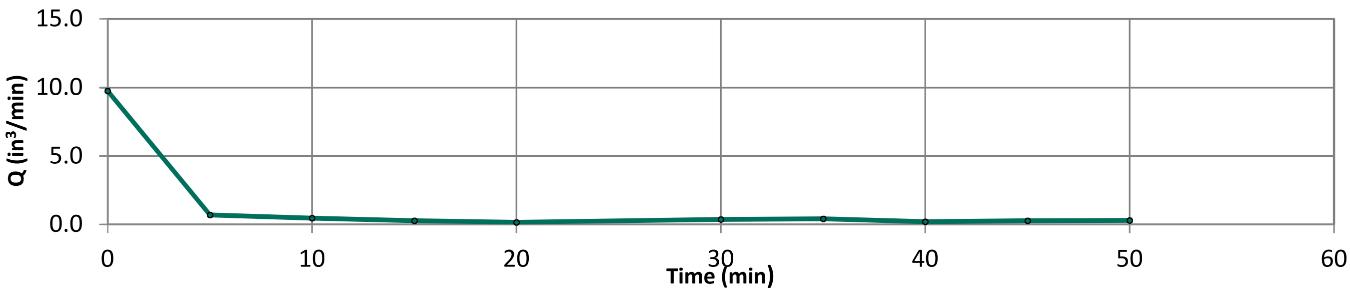
TEST NO.: **A-2a**

GEOLOGIC UNIT: **Qvop**

EXCAVATION ELEVATION (MSL, FT): **483**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	4
BOREHOLE DEPTH (FT):	2.1
TEST/BOTTOM ELEVATION (MSL, FT):	481
MEASURED HEAD HEIGHT (IN):	3.0
CALCULATED HEAD HEIGHT (IN):	3.7
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.286**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.054**
FACTORED INFILTRATION RATE (IN/HR):	**0.027**



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	1.760	48.74	9.748
3	5.00	0.125	3.46	0.692
4	5.00	0.085	2.35	0.471
5	5.00	0.050	1.38	0.277
6	10.00	0.060	1.66	0.166
7	5.00	0.065	1.80	0.360
8	5.00	0.075	2.08	0.415
9	5.00	0.035	0.97	0.194
10	5.00	0.050	1.38	0.277
11	5.00	0.055	1.52	0.305
12	5.00	0.045	1.25	0.249
13	5.00	0.050	1.38	0.277
14	5.00	0.055	1.52	0.305
15	5.00	0.050	1.38	0.277

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AARDVARK PERMEAMETER TEST RESULTS

OTAY MESA AIRWAY ROAD

PROJECT NO.: **G2467-42-01**

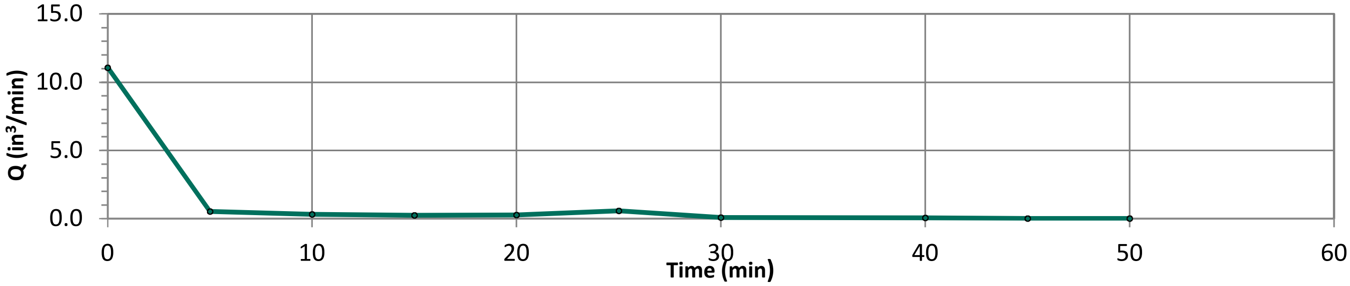
TEST NO.: **A-3**

GEOLOGIC UNIT: **Qvop**

EXCAVATION ELEVATION (MSL, FT): **485**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	5
BOREHOLE DEPTH (FT):	2.0
TEST/BOTTOM ELEVATION (MSL, FT):	483
MEASURED HEAD HEIGHT (IN):	3.3
CALCULATED HEAD HEIGHT (IN):	3.8
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.028**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.004**
FACTORED INFILTRATION RATE (IN/HR):	**0.002**



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	2.000	55.38	11.077
3	5.00	0.094	2.60	0.521
4	5.00	0.056	1.55	0.310
5	5.00	0.045	1.25	0.249
6	5.00	0.050	1.38	0.277
7	5.00	0.105	2.91	0.582
8	10.00	0.030	0.83	0.083
9	5.00	0.010	0.28	0.055
10	5.00	0.005	0.14	0.028
11	5.00	0.005	0.14	0.028
12	5.00	0.005	0.14	0.028

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AARDVARK PERMEAMETER TEST RESULTS

OTAY MESA AIRWAY ROAD

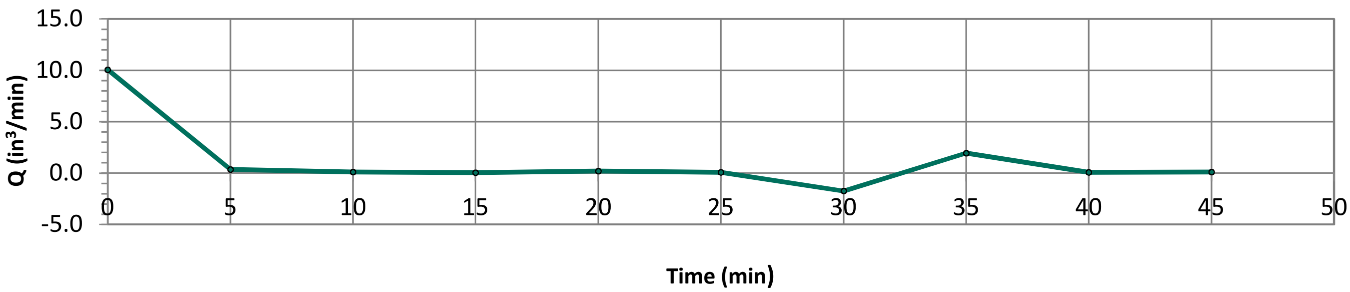
PROJECT NO.: G2467-42-01

TEST NO.: **A-3a** GEOLOGIC UNIT: **Qvop**

EXCAVATION ELEVATION (MSL, FT): **485**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	4
BOREHOLE DEPTH (FT):	3.0
TEST/BOTTOM ELEVATION (MSL, FT):	482
MEASURED HEAD HEIGHT (IN):	3.3
CALCULATED HEAD HEIGHT (IN):	4.0
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.129**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.023**
FACTORED INFILTRATION RATE (IN/HR):	**0.012**



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	1.820	50.40	10.080
3	5.00	0.065	1.80	0.360
4	5.00	0.020	0.55	0.111
5	5.00	0.010	0.28	0.055
6	5.00	0.035	0.97	0.194
7	5.00	0.015	0.42	0.083
8	5.00	-0.315	-8.72	-1.745
9	5.00	0.355	9.83	1.966
10	5.00	0.015	0.42	0.083
11	5.00	0.020	0.55	0.111
12	5.00	0.020	0.55	0.111
13	5.00	0.025	0.69	0.138
14	5.00	0.025	0.69	0.138
15	5.00	0.020	0.55	0.111

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AARDVARK PERMEAMETER TEST RESULTS

OTAY MESA AIRWAY ROAD

PROJECT NO.: G2467-42-01

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:
Entire Site		Preliminary
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ☐ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ☑ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ☐ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ☐ Yes; Continue to Step 1C. ☐ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ☐ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ☐ Yes; continue to Step 1E. ☐ No; select an appropriate infiltration testing method.	

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

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

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

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

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

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions</th><th colspan="2">Worksheet C.4-1: Form I-8A[10]</th></tr>
<tr><td colspan="4">Criteria 2: Geologic/Geotechnical Screening</td></tr>
<tr><td>2A</td><td colspan="3">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.</td></tr>
<tr><td>2A-1</td><td>Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?</td><td>☐Yes</td><td>☐No</td></tr>
<tr><td>2A-2</td><td>Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?</td><td>☐Yes</td><td>☐No</td></tr>
<tr><td>2A-3</td><td>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?</td><td>☐Yes</td><td>☐No</td></tr>
<tr><td>2B</td><td colspan="3">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.</td></tr>
<tr><td>2B-1</td><td>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?</td><td>☐Yes</td><td>☐No</td></tr>
<tr><td>2B-2</td><td>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?</td><td>☐Yes</td><td>☐No</td></tr>
</table>

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

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

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:
Entire Site	Preliminary
Criteria 3 : Infiltration Rate Screening	

3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☐ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☑ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ☐ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ☑ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ☐ Yes; Continue to Criteria 4. ☑ No: Skip to Part 2 Result.

Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).

Six infiltration tests were performed on the property. The factored test results were as follows:

A-1: 0.001 in/hr
A-1a: 0.003 in/hr
A-2: 0.060 in/hr
A-2a: 0.027 in/hr
A-3: 0.002 in/hr
A-3a: 0.012 in/hr

The average rate of the six tests is 0.02 in/hr. Five of the six tests had infiltration rates below 0.05 in/hr.

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

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

<table>
<tr><th colspan="3">Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions</th><th colspan="2">Worksheet C.4-1: Form I-8A[10]</th></tr>
<tr><td>Criteria 4 Result</td><td colspan="2">Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?</td><td>☐ Yes</td><td>☐ No</td></tr>
<tr><td colspan="5">Summarize findings and basis; provide references to related reports or exhibits.</td></tr>
<tr><td colspan="3">Part 2 – Partial Infiltration Geotechnical Screening Result[13]</td><td colspan="2">Result</td></tr>
<tr><td colspan="3">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.</td><td colspan="2">☐ Partial Infiltration Condition

☑ No Infiltration Condition</td></tr>
</table>

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

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

AIRWAY ROAD
INDUSTRIAL BUILDING
SAN DIEGO, CALIFORNIA

PROJECT NO. G2467-42-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

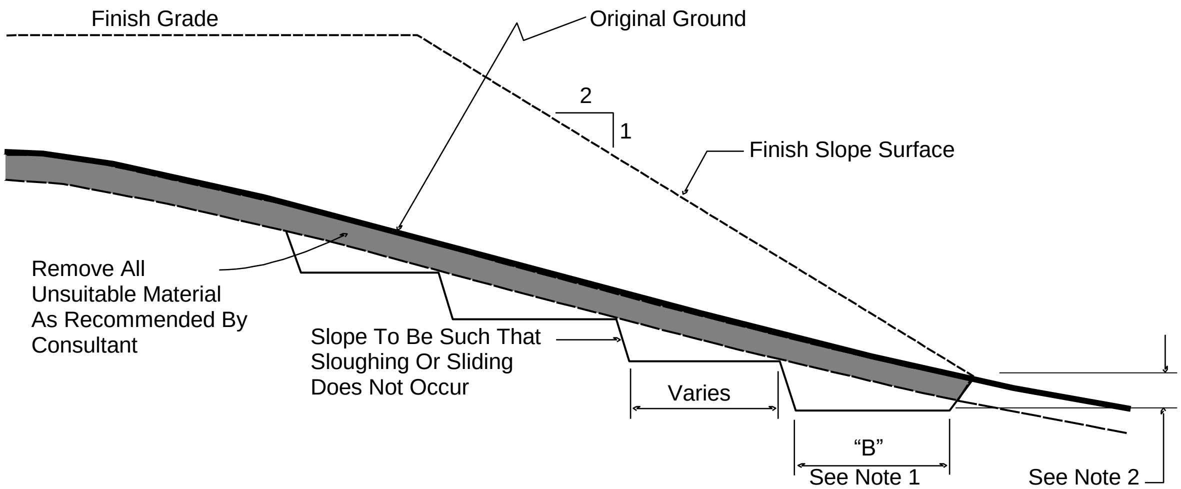
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



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.

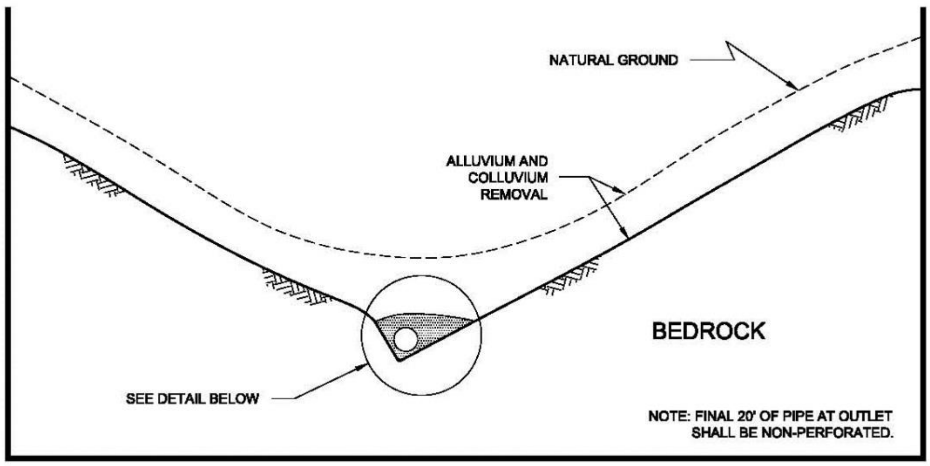
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

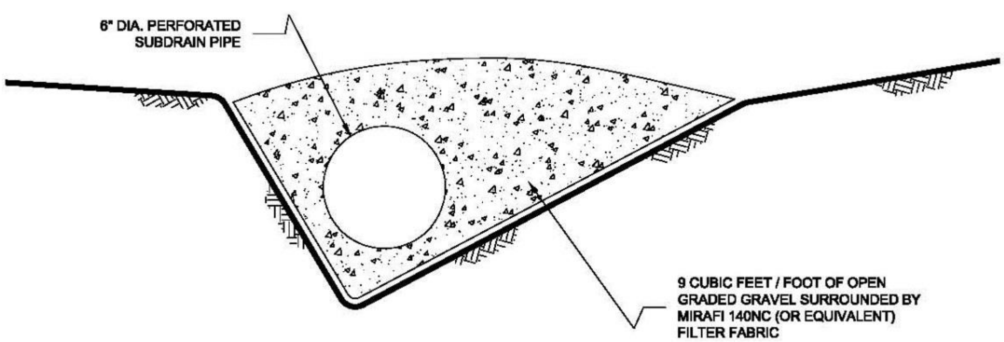
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

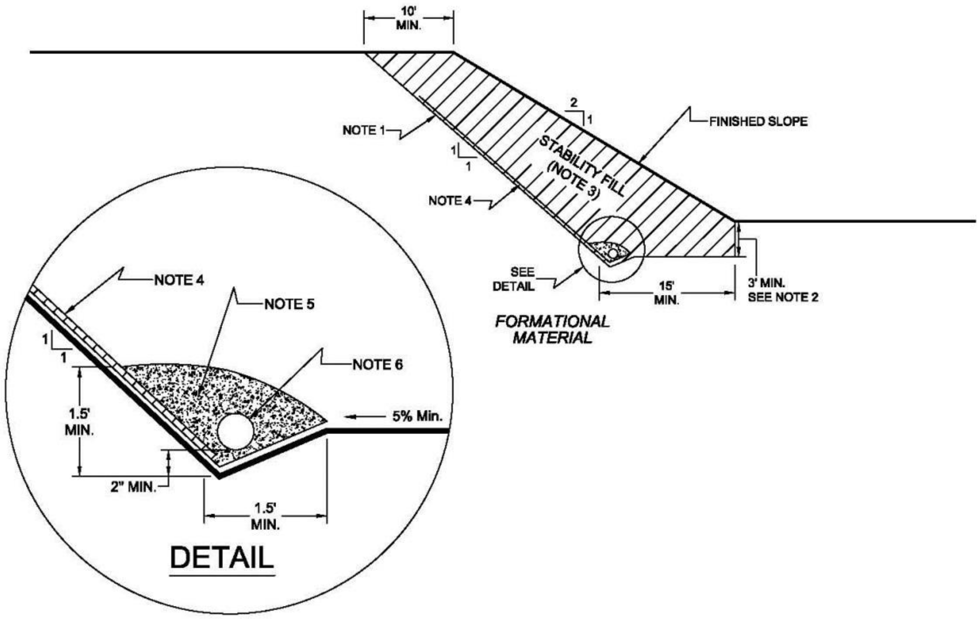
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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.

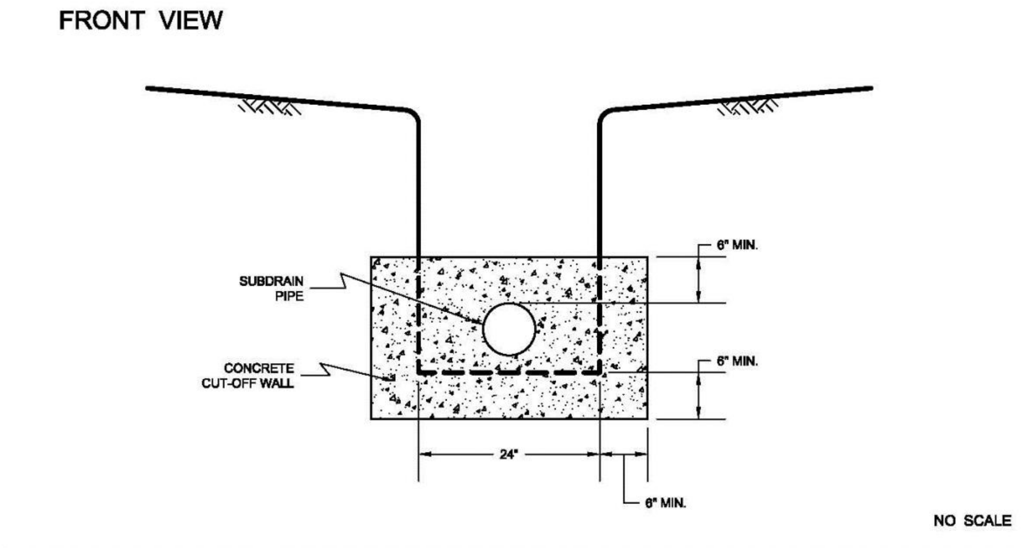
NO SCALE

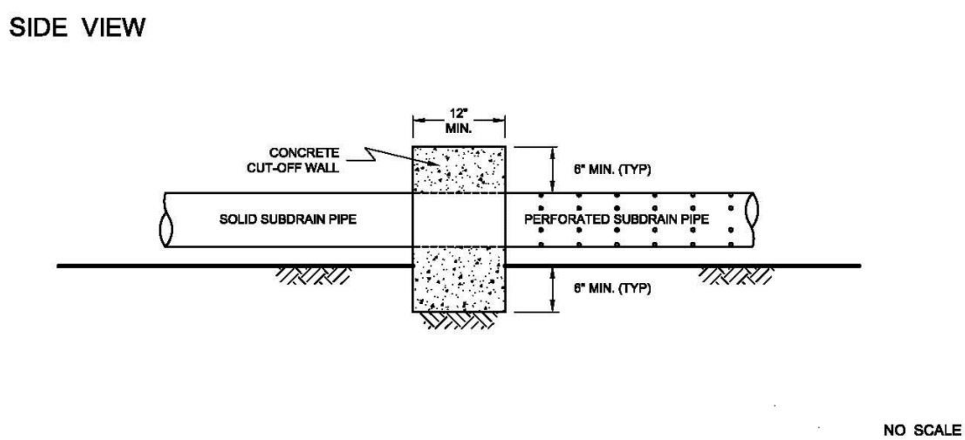
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

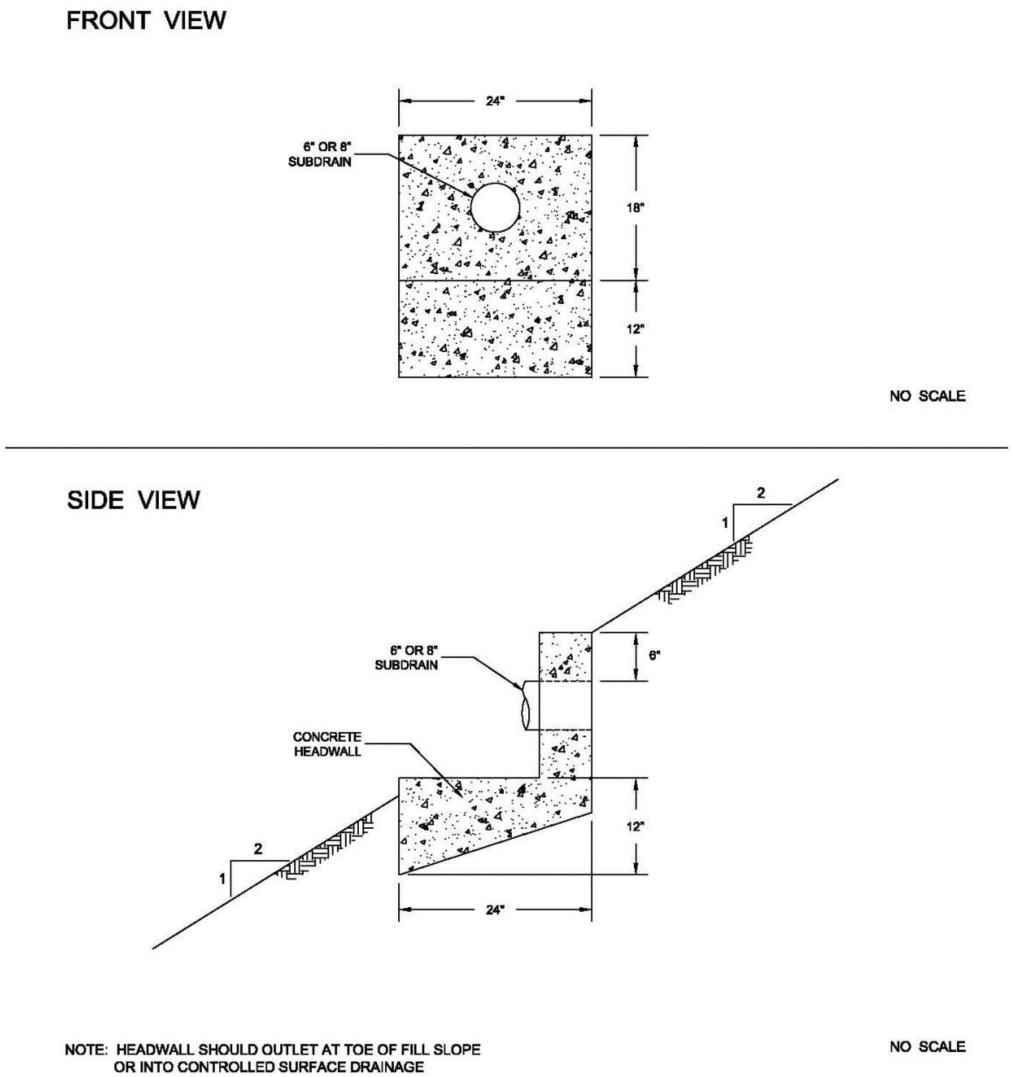
TYPICAL CUT OFF WALL DETAIL





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

TYPICAL HEADWALL DETAIL



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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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

LIST OF REFERENCES

1. FEMA (2012), *Flood Map Service Center*, FEMA website, https://msc.fema.gov/portal/home, flood map number 06073C2159G, effective May 16, 2012, accessed March 13, 2020;

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

3. Kennedy, M. P., and S. S. Tan, 2007, *Geologic Map of the Oceanside 30'x60' Quadrangle, California*, USGS Regional Map Series Map No. 1, Scale 1:100,000.

4. SEAOC (2019), *OSHPD Seismic Design Maps:* Structural Engineers Association of California website, http://seismicmaps.org/, accessed March 13, 2020;

5. USGS (2019), *Quaternary Fault and Fold Database of the United States*: U.S. Geological Survey website, https://www.usgs.gov/natural-hazards/earthquake-hazards/faults, accessed March 13, 2020;

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