GEOTECHNICAL INVESTIGATION

SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

TONY SHORES
EL CAJON, CALIFORNIA

SDC PDS RCVD 09-20-18
STP18-012

DECEMBER 21, 2017
PROJECT NO. G2217-52-01

GEOCON
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GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G2217-52-01
December 21, 2017

Tony Shores
2020 Hillsdale Road
El Cajon, California 92019

Subject: GEOTECHNICAL INVESTIGATION
SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

Dear Mr. Shores:

In accordance with your authorization of our Proposal No. LG-17283 dated November 6, 2017, we herein submit the results of our geotechnical investigation for the subject site. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards and to assist in the design of the proposed improvements. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of the proposed development. The site is considered suitable for development provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON INCORPORATED

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Attention: Mr. Tom Sheehan

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed commercial improvements to a Shell Gas Station in the Alpine area of the County of San Diego, California. The purpose of this geotechnical investigation is to evaluate the surface and subsurface soil conditions, general site geology and to identify geotechnical constraints that may impact development of the property.

To aid in preparing this report we reviewed:

1. *[Preliminary] Site Plan – Phase 1C, Tavern Road Gas Station, Alpine, California,* prepared by ZAAP Architectural + Planning, dated October 17, 2017 (Project No. 17010).
2. *[Preliminary] Alta/ NSPS Land Title Survey, Alpine Shell, Alpine, California,* prepared by Omega Land Surveying Incorporated, plot dated June 26, 2017.

The scope of this investigation included the review of aerial photographs and readily available published and unpublished geologic literature (see List of References), excavating nine exploratory trenches to a maximum depth of 12 feet, soil sampling, laboratory testing, engineering analyses and preparing this geotechnical investigation report.

Appendix A presents the exploratory trench logs and details of the field investigation. We performed laboratory tests on selected soil samples obtained during the field investigation to evaluate pertinent physical and chemical properties for engineering analyses and to assist in providing geotechnical engineering recommendations for project design. Details of the laboratory tests and a summary of the test results are presented in Appendix B and on the trench logs in Appendix A. The Storm Water Management Investigation is presented in Appendix C.

2. SITE AND PROJECT DESCRIPTION

The project site consists of two parcels located north of Interstate 8 and west of Tavern Road in the community of Alpine, California (see Vicinity Map, Figure 1). The site is bordered by Tavern Road to the north and the east, undeveloped land and the Interstate 8 westbound onramp to the south, and undeveloped land and a dirt storage yard of an adjacent property to the west. The eastern parcel of the site (Parcel 2) is currently occupied by a Shell Gas Station consisting of a convenience store structure, gas pumps, miscellaneous ancillary structures, asphalt parking and driveways, and other associated improvements. The Shell Gas Station is accessed by driveway entrances from Tavern Road. The majority of the western parcel (Parcel 1) is undeveloped sloping land consisting of a dirt access road along the eastern perimeter of the parcel, rock and soil stockpiles, and brush/trees. The

Parcel 1 area of the site generally descends to the west with a naturally occurring canyon drainage traversing the central portion of the parcel.

The Parcel 2 area where the existing Shell Station is located was likely previously graded with approximately 1½:1 to 2:1 (horizontal to vertical) descending fill slopes with a maximum height of about 30 feet along the west and south perimeters. Elevations at the site range from approximately 1,715 feet above Mean Sea Level (MSL) within the central-western portion of Parcel 1 to 1,765 feet MSL in the northeastern end of the site on Parcel 2. The gas station area slopes gently to the south with a total relief of approximately 3 feet.

Based on review of the preliminary architectural plans prepared by ZAAP Architectural + Planning, we understand the planned project will consist of the reconfiguration and expansion of the existing gas station. The improvements will consist of the removal of the existing convenience store structure and constructing a new structure consisting of a new restaurant, gas station convenience store, and drive-thru restaurant located within the south-central portion of the site. We expect the existing gas station convenience store structure will be removed to allow the proper grading for the planned structures. The canopy and gas pumps can likely remain during the construction operations. The new structure will have a building coverage of approximately 7,000 square feet. In addition, improvements will consist of the construction of a new coffee kiosk, asphalt parking and driveways, two bioswales and other associated improvements. The grading for the planned expansion will require fills up to approximately 30 feet with 2:1 (horizontal to vertical) fill slopes up to approximately 45 feet high along the west and southwest ends of the site. Retaining walls may be installed along the south-central portion of the site to accommodate the planned drive-thru lane. We used the referenced preliminary site plan and land title survey as a base for our Geologic Map, Figure 2.

The site location, descriptions, and proposed development discussed herein are based on a site investigation, review of the preliminary architectural site plans and land title survey and our discussions with you. If development plans differ from those described herein, Geocon Incorporated should be contacted for review of the plans and possible revisions to this report.

3. SOIL AND GEOLOGIC CONDITIONS

We encountered topsoil, undocumented fill, older alluvium and Granitic Rock during our field investigation. The occurrence and distribution of the units are presented on the trench logs in Appendix A, and on the Geologic Map, Figure 2. Figure 3 presents a Geologic Cross-Section providing our subsurface geologic interpretation. The geologic units are described in order of increasing age.

3.1 Undocumented Fill (Qudf)

We encountered undocumented fill to a maximum depth of 8 feet in the exploratory trenches located within the existing fill slope. The undocumented fill is likely associated with the grading required to construct the existing Shell Station pad. We expect the undocumented fill thickest at the southern end of the existing gas station at the top of the existing fill slope with an estimated thickness of up to approximately 20 to 25 feet. We expect the southern portion of the proposed structure is underlain by up to approximately 15 feet of undocumented fill. The fill thickness is likely less than 5 feet adjacent to Tavern Road. The fill encountered in the exploratory trenches generally consists of loose to medium dense, dry to moist, yellowish to reddish brown, silty sand. A geotechnical report documenting the testing and observation of the fill soils was not available, therefore, the existing fill soil is designated as undocumented. The undocumented fill is considered unsuitable for support of additional fill and/or structural loads in its present condition and will require compete removal where possible in areas of planned development. The majority of this material will likely be suitable for reuse as compacted fill provided debris and vegetation is removed.

3.2 Topsoil (unmapped)

We observed topsoil overlying Older Alluvium within the Parcel 1 area with a thickness of up to approximately 1½ feet. The topsoil consists of loose, yellowish to reddish brown, silty sand. The topsoil is unsuitable in its present condition and will require remedial grading in areas of planned improvement. This material is suitable for use as compacted fill from a geotechnical engineering standpoint provided the soil is relatively free of vegetation.

3.3 Older Alluvium (Qoal)

We encountered older alluvium underlying topsoil or undocumented fill with a thickness ranging from approximately 1 to 2½ feet within the exploratory trenches T-2 through T-9. The older alluvium generally consists of medium dense to very dense, dry to damp, reddish brown, silty sand. The older alluvium is prone to hydroconsolidation and is considered unsuitable for support of fill and/or structural loads and will require removal in areas of planned improvement. This material is suitable for use as compacted fill from a geotechnical engineering standpoint provided the soil is relatively free of vegetation.

3.4 Granitic Rock (Kgr)

Early Cretaceous-aged granitic rock underlies the topsoil and/or older alluvium at depths ranging from approximately 1 to 9½ feet below existing grades. The granitic rock generally is highly weathered and moderately weak consisting of a medium- to coarse-grained tonalite. Deeper excavations into the granitic rock may encounter refusal. The granitic rock is considered suitable for support of the proposed building and improvements.

4. GROUNDWATER

We did not encounter groundwater during our field investigation and we do not expect groundwater to have a significant influence on construction operations or the performance of the improvements. It is not uncommon for seepage conditions to develop where none previously existed. Seepage is dependent on seasonal precipitation, irrigation and land use, among other factors, and varies as a result. Proper surface drainage will be critical 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 indicate 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 site is not located within a State of California Earthquake Fault Zone.

According to the computer program *EZ-FRISK* (Version 7.65), 9 known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database to evaluate the fault parameters. The nearest known active fault is the Elsinore Fault system located approximately 21 miles northeast of the site, and is the dominant source of potential ground motion. Earthquakes that might occur on the Elsinore fault or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Elsinore fault are 7.9 and 0.18g, respectively. Table 5.1.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships.

TABLE 5.1.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Elsinore	21	7.9	0.18	0.13	0.17
Newport-Inglewood	24	7.5	0.14	0.10	0.12
Rose Canyon	24	6.9	0.11	0.09	0.08
Earthquake Valley	25	6.8	0.10	0.08	0.07
Coronado Bank	35	7.4	0.10	0.07	0.08
Palos Verdes Connected	35	7.7	0.12	0.08	0.10
San Jacinto	41	7.9	0.11	0.08	0.10
Brawley Gridded, Strike Slip	44	6.5	0.05	0.04	0.04
Brawley Gridded, Normal	44	6.5	0.04	0.04	0.03

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) in the analysis. Table 5.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 5.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2007 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.35	0.38	0.43
5% in a 50 Year Period	0.27	0.28	0.30
10% in a 50 Year Period	0.21	0.21	0.22

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

5.2 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the earth surface. The potential for ground rupture is considered to be very low due to the absence of active faults at the subject site.

5.3 Slope Stability

Slope stability analyses for the proposed fill slopes and the existing descending fill slope along the south end of the site with inclinations as steep as 2:1 (horizontal to vertical) indicate a calculated factor of safety of at least 1.5 under static conditions for both deep-seated and surficial failure. Figures 4 and 5 present the slope stability calculations for deep-seated and surficial failures for the proposed and existing fill slopes, respectively.

We performed the slope stability analyses based on the interpretation of geologic conditions encountered during our field investigation. Additional analyses may be required during the grading operations if the geologic conditions vary significantly.

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

5.4 Tsunamis and Seiches

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 site is located approximately 25 miles from the Pacific Ocean, at elevations ranging from approximately 1,715 to 1,765 feet MSL; therefore, the risk of tsunami impacting the site is considered negligible.

Seiches are standing wave oscillations of an enclosed water body after the original driving force has dissipated. Driving forces are typically caused by seismic ground shaking. The site is not located adjacent to a body of water; therefore, the risk of seiches impacting the site is considered negligible.

5.5 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, on-site soils are cohesionless/silt or clay with low plasticity, groundwater is encountered, and soil relative densities are less than about 70 percent. If the four previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. Due to the dense nature of the underlying older alluvium and granitic rock, the planned compacted fill and the lack of a near surface groundwater table, the potential for liquefaction occurring within the site soil is considered negligible.

5.6 Landslides

Based on the examination of aerial photographs in our files, review of published geologic maps for the site vicinity, and the nature of the formational material, it is our opinion that landslides are not present at the property or at a location that could impact the subject site.

5.7 Hydroconsolidation

Hydroconsolidation is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the affected soil and overlying foundations or improvements supported thereon. A potential for settlement due to hydroconsolidation of the older alluvium exists. The potential for hydroconsolidation can be reduced by remedial grading and the use of stiffer foundation systems. Based on the laboratory test results, the potential for hydroconsolidation of the older alluvium is 0.8 percent. The older alluvium should be removed and replaced with properly compacted fill due to the hydroconsolidation potential.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 From a geotechnical engineering standpoint, it is our opinion that the site is suitable for development provided the recommendations presented herein are implemented in design and construction of the project.

6.1.2 Our field investigation indicates the site is underlain by undocumented fill, topsoil, older alluvium and granitic rock. Remedial grading of the fill, topsoil and older alluvium should be performed within the limits of grading where possible. The fill, topsoil and older alluvium should be generally free of vegetation and debris prior to placement as compacted fill or, if unable to be cleaned, exported offsite. The granitic rock is considered adequate for the support of compacted fill and/or structural loads. We expect that the proposed structure and surface improvements will be supported by properly compacted fill.

6.1.3 Potential geologic and geotechnical hazards at the site include moderate to strong seismic ground shaking and settlements of surficial soils left-in-place. Based on our investigation and available geologic information, active or potentially active faults are not present underlying or trending toward the site.

6.1.4 We expect the limits of the remedial grading operations will be contained within the property limits as shown on the grading plans. In addition, we expect grading may be limited to the areas outside existing structures that will be remain including the underground storage area and gas pumps. However, the existing convenience store will require removal to allow for the planned grading operations. We should be contacted to provide additional recommendations if the convenience store will not be removed before the grading operations. Temporary excavations for remedial grading along the project margins and around structures to remain should extend into the site at inclinations of 1:1 (horizontal to vertical).

6.1.5 Excavation of the undocumented fill, topsoil, older alluvium and underlying granitic rock should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. Gravel, cobble and some boulders should be expected during the grading operations.

6.1.6 The site is located approximately 21 miles from the nearest active fault. Based on our background research, it is our opinion active, potentially active, or inactive faults do not extend across or trend toward the site. Risks associated with seismic activity consist of the potential for strong seismic shaking.

6.1.7 We did not encounter groundwater during our investigation and do not expect groundwater would impact site development. However, wet conditions and seepage could affect proposed development if grading operations occur during the rainy season or where seepage occur. In addition, we encountered a septic system in our trenches T-6 and T-9. Some saturated soil may exist where the septic system is located.

6.1.8 We expect the proposed structures can be supported on a conventional shallow foundation system with concrete slabs-on-grade embedded into properly compacted fill. Other foundation types may be required if the convenience store is not removed before the grading operations.

6.1.9 Proper drainage should be maintained in order to preserve the engineering properties of the fill in the sheet-graded pad and slope areas. Recommendations for site drainage are provided herein.

6.1.10 Surface settlement monuments will not be required prior to or during site development.

6.2 Soil Characteristics

6.2.1 The soil encountered in the field investigation is considered to be "non-expansive" and "expansive" (expansion index [EI] less of 20 or less and greater than 20, respectively) as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 6.2.1 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a "very low" to "low" expansion potential (EI of 50 or less) in accordance with ASTM D 4829.

TABLE 6.2.1
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

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

6.2.2 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 location tested possesses “S0” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. Concrete placed at the site should be designed using an “S0” sulfate exposure. 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.

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

6.3 Seismic Design Criteria – California Building Code

6.3.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS to evaluate the seismic design criteria. Table 6.3.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements should be designed using a Site Class D. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 6.3.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.3.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
Spectral Response – Class B (short), S_S	0.968g	Figure 1613.3.1(1)
Spectral Response – Class B (1 sec), S_1	0.360g	Figure 1613.3.1(2)
Site Coefficient, F_a	1.113	Table 1613.3.3(1)
Site Coefficient, F_v	1.681	Table 1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.077g	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.604g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.718g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.403g	Section 1613.3.4 (Eqn 16-40)

6.3.2 Table 6.3.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.3.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 or 2016 CBC Reference
Site Class	D	Section 1613.3.2 (CBC)
Mapped MCE_G Peak Ground Acceleration, PGA	0.364g	Figure 22-7 (ASCE)
Site Coefficient, F_{PGA}	1.136	Table 11.8-1 (ASCE)
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.413g	Section 11.8.3 (Eqn 11.8-1, ASCE)

6.3.3 Conformance to the criteria in Tables 6.3.1 and 6.3.2 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 Temporary Excavations

6.4.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor to provide a safe excavation during the construction of the proposed project.

6.4.2 Temporary slopes should be made in conformance with OSHA requirements. The topsoil and undocumented fill should be considered a Type C soil, new compacted fill should be considered a Type B soil (Type C if seepage is encountered) and the older alluvium and granitic rock should be considered a Type A soil (Type B soil if seepage or groundwater is encountered) in accordance with OSHA requirements. In general, special shoring requirements will not be necessary if temporary excavations will be less than 4 feet in height. Temporary excavations greater than 4 feet in height, however, should be sloped back at an appropriate inclination. 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.

6.5 Grading

6.5.1 Grading should be performed in accordance with the *Recommended Grading Specifications* in Appendix D. Where the recommendations herein conflict with Appendix D, the recommendations of this section take precedence.

6.5.2 Geocon Incorporated should provide testing and observation services during the earthwork operations.

6.5.3 A pre-construction meeting with a county representative, owner, architect, grading contractor, civil engineer, and a representative of Geocon Incorporated should be held prior to the beginning of grading and development operations. Grading requirements and building construction methods can be discussed at that time.

6.5.4 Grading of the site should commence with the removal and export of the existing concrete slabs, vegetation, debris and rock stockpiles from the areas of development. If existing underground improvements require removal, the resulting depressions should be properly backfilled in accordance with the procedures described herein.

6.5.5 The surficial materials (undocumented fill, topsoil and older alluvium) within the limits of the planned structures and within areas of planned fill should be removed to expose granitic rock materials, where possible, and replaced with compacted fill. In addition, the upper 1 to 2 feet of the existing materials in new pavement areas should be removed and replaced with properly compacted fill. We expect removals will be limited at the property lines because off-site grading will not be performed. In addition, we expect grading may be limited to the areas outside existing structures that will be remain including the underground storage area and gas pumps.

6.5.6 We expect the southern portion of the proposed building is underlain by up to approximately 15 feet of surficial materials, and there may be portions of the building underlain by shallow fill less than a few feet thick. The surficial materials within the limits of the planned building should be removed to granitic rock and replaced with compacted fill. The building pad should be undercut a minimum of 3 feet and replaced with properly compacted fill where the granitic rock is encountered within 3 feet of finish grade. The limit of the removals should extend at least 5 feet outside of the planned building limits, where possible.

6.5.7 The base of the removals and undercut areas should be scarified, moisture conditioned as necessary, and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content, as determined in accordance with ASTM D 1557.

6.5.8 Recommendations for the handling and disposal of oversized rock in fill areas are presented in Figure 6 and in Appendix D. In general, structural fill placed and compacted at the site should consist of material that can be classified into four zones:

Zone A: Material placed within 3 feet from building pad grade, 8 feet from roadway grade, and to at least 1 foot below the deepest utility within roadways should consist of “soil” fill with an approximate maximum particle dimension of 6 inches with a minimum of 40 percent of the soil passing the ¾-inch sieve. In addition, the upper 3 feet of pad grade should have at least 20 percent of the soil passing the No. 4 sieve.

Zone B: Material placed below 8 feet from grade (below *Zone A* and *C*) may consist of “rock” fill or “soil/rock” fill (as defined in Appendix D). Blasted rock should generally consist of 2 foot minus rock material with occasional rock up to 4 foot in maximum dimension. Alternatively, “soil” fill may be placed in Zone B containing rock with a maximum dimension of 2 feet. Rocks up to 4 feet in maximum dimension can be individually placed in a properly compacted soil matrix with rocks separated at least 8 feet apart.

Zone C: Within 3 to 8 feet of pad grade and between 5 and 15 feet from face of slope, fill material should consist of “soil” fill with an approximate maximum particle dimension of 1 foot. Rocks up to 2 feet in maximum dimension may be placed, provided they are distributed in a matrix of compacted “soil” fill.

Zone D: Within the outer 5 feet of fill slopes, the fill should consist of rock up to 1 foot in maximum dimension in a matrix of compacted “soil” fill.

6.5.9 The site should then be brought to final subgrade elevations with structural fill compacted in layers. In general, soil native to the site is suitable for use 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, as determined 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.

6.5.10 Import fill (if necessary) should consist of granular materials with a “very low” to “low” expansion potential (EI of 50 or less) free of deleterious material or rock larger than 3 inches and should be compacted as recommended herein. 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.

6.5.11 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular "soil" fill to reduce the potential for surficial sloughing. In general, soil with an expansion index of 50 or less and at least 35 percent sand-size particles should be acceptable as "soil" fill. Soil of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength. The use of cohesionless soil in the outer portion of fill slopes should be avoided. Fill slopes should be overbuilt at least 2 feet and cut back or be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet to maintain the moisture content of the fill. The slopes should be track-walked at the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content to the face of the finished slope.

6.5.12 Placement of rock fills should be planned in the deeper fill areas to facilitate rock disposal. Overexcavation of fill areas may be required to accommodate the necessary rock volumes generated during blasting. Capping material used for placement near finish grade within roadways, building pads, and slope zones should be stockpiled during excavation and remedial grading operations. Overexcavation of units that generate capping material may be necessary to achieve sufficient volumes to achieve finish grade.

6.5.13 Rock fill placement should be performed in accordance with the Recommended Grading Specifications provided in Appendix D. We do not expect blasting would be required due to the planned import necessary for the proposed site elevations. Blasting of rock material, if necessary, should be performed to maximize rock breakage to 2-foot minus material. Rock fill placement should generally be limited to 2-foot-thick horizontal layers and compacted using rock trucks and bulldozers. Significant volumes of water are typically required during rock fill placement. The downstream areas can generate large volumes of water that can be re-used during construction.

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

6.6 Settlement Due to Fill Loads

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

6.6.2 Due to the variable fill thickness, a potential for differential settlement across the proposed building exists. Based on measured settlement of similar fill depths on this and other sites and the time period since the fill was placed, we estimate that maximum settlement of the compacted fill will be approximately 0.4 percent of the fill thickness soon after the grading operations are complete.

6.6.3 The proposed building will be underlain by a maximum differential thickness of compacted fill on the order of 20 feet. The settlement of compacted fill is expected to continue over a relatively extended time period resulting from both gravity loading and hydrocompression upon wetting from rainfall and/or landscape irrigation.

6.6.4 The amount of differential settlement and the estimated maximum angular distortion that could occur for the building is 1 inch and 1/360, respectively.

6.6.5 The estimated differential settlement for fill underlying the building should be considered in the design of improvements and adjacent flatwork. Additionally, the total and differential settlement should be incorporated into the design for pavement areas.

6.6.6 Deep foundations are the most effective means of reducing the ultimate settlement potential of the proposed structure to a negligible amount. However, installing deep foundations may not be economically feasible. Highly reinforced shallow foundation systems and slabs-on-grade may be used for support of the structure; however, the shallow foundation systems would not eliminate the potential for distress related to differential settlement of the underlying fill. Some cosmetic distress should be expected over the life of the structure as a result of long-term differential settlement. The building owner, tenants, and future owners should be made aware that cosmetic distress, including separation of caulking at wall joints, small, non-structural wall panel cracks, and separation of concrete flatwork, is likely to occur. Recommendations for deep foundations can be provided to evaluate the comparative risks and costs upon request.

6.7 Shallow Foundations and Concrete Slabs-On-Grade

6.7.1 The proposed structures can be supported on a shallow foundation system founded in the compacted fill. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Continuous footings should be at least 12 inches wide and extend at least 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 2 feet and should also extend at least 24 inches below lowest adjacent pad grade. Figure 7 shows a wall/column footing dimension detail. In addition, 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

6.7.2 Steel reinforcement for continuous footings should consist of at least four No. 5 steel reinforcing bars placed horizontally in the footings, two near the top and two near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

6.7.3 The recommendations herein are based on soil characteristics only (EI of 50 or less) and is not intended to replace reinforcement required for structural considerations.

6.7.4 The recommended allowable bearing capacity for foundations with minimum dimensions described herein and bearing in properly compacted fill is 2,000 pounds per square foot (psf). The 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.

6.7.5 We estimate the total and differential settlements under the imposed allowable loads to be about ½ inch based on a 5-foot square footing.

6.7.6 Concrete floor slabs should possess a thickness of at least 5 inches and reinforced with a minimum of No. 4 steel reinforcing bars at 18 inches on center in both horizontal directions. The structural engineer should design the steel required for the planned loading conditions.

6.7.7 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

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

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

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

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

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

6.7.13 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal to 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 to 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.

6.7.14 The foundation and concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting expected loads.

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

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

6.7.17 Where exterior flatwork abuts the structure at entrant or exit areas, 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.

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

6.8 Concrete Flatwork

6.8.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and, when in excess of 8 feet square, should be reinforced with 6 x 6 - W2.9/W2.9 (6 x 6 - 6/6) welded wire mesh or No. 3 reinforcing bars spaced at least 18 inches center-to-center in both directions to reduce the potential for cracking. In addition, 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 checked prior to placing concrete.

6.8.2 Even with the incorporation of the recommendations within this report, the exterior concrete flatwork has a likelihood of experiencing some uplift due to expansive soil beneath grade; therefore, 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 Retaining Walls

6.9.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at 2:1 (horizontal to vertical), we recommend an active soil pressure of 50 pcf. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

6.9.2 Retaining walls should be designed to ensure stability against overturning sliding, 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.

6.9.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure 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. 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. Figure 8 presents the retaining wall loading diagram.

6.9.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2016 CBC or Section 11.6 of ASCE 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 17H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.413g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.3.

6.9.5 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.

6.9.6 In general, wall foundations having a minimum depth and width of 1 foot may be designed for an allowable soil bearing pressure of 2,000 psf. The allowable soil bearing pressure may be increased by an additional 300 psf for each additional foot of depth and width, to a maximum allowable bearing capacity of 3,000 psf. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, 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.

6.9.7 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.

6.9.8 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. Figure 9 presents a 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.

6.9.9 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.

6.9.10 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.

6.10 Lateral Loading

6.10.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid density of 350 pounds per cubic foot (pcf) should be used 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.

6.10.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.4 should be used for design. The friction coefficient may be reduced depending on the vapor barrier or waterproofing material used for construction in accordance with the manufacturer's recommendations (normally about 0.2 to 0.25).

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

6.11 Preliminary Pavement Recommendations

6.11.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We understand that import soils may be required to achieve finish grade, therefore, we have assumed an R-Value of 30 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 6.11.1 presents the preliminary flexible pavement sections.

TABLE 6.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	30	3	6
Driveways for automobiles and light-duty vehicles	5.5	30	3	7
Medium truck traffic areas	6.0	30	3.5	8
Driveways for heavy truck traffic	7.0	30	4	10

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

6.11.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*.

6.11.4 The base thickness can be reduced if a reinforcement geogrid is used during the installation of the pavement. Geocon should be contact for additional recommendations, if required.

6.11.5 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway entrance aprons, trash bin loading/storage areas and loading dock areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 6.11.2.

TABLE 6.11.2
RIGID PAVEMENT DESIGN PARAMETERS

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

6.11.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 6.11.3.

TABLE 6.11.3
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Areas (TC=A)	5.5
Heavy Truck and Fire Lane Areas (TC=C)	7.0

6.11.7 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

6.11.8 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., a 7-inch-thick slab would have a 9-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

6.11.9 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 12.5 feet and 15 feet for the 5.5 and 6-inch-thick slabs and thicker, respectively, and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report. The depth of the crack-control joints should be at least ¼ of the slab thickness when using a conventional saw, or at least 1 inch when using early-entry saws on slabs 9 inches or less in thickness, as determined by the referenced ACI report discussed in the pavement section herein. Cuts at least ¼ inch wide are required for sealed joints, and a ⅜ inch wide cut is commonly recommended. A narrow joint width of $^{1}/_{10}$ to $^{1}/_{8}$-inch wide is common for unsealed joints.

6.11.10 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 at the as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

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

6.12 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 2016 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

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

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

6.12.4 We understand detention basins, bioswales, retention basins, water infiltration, low impact development (LID), or storm water management devices are being considered. The recommendations provided herein pertain to the geotechnical aspects of design and possible impacts of implementation. Appendix C presents the results of the storm water investigation.

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 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.

2. 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 that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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

4. 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.

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NO SCALE

VICINITY MAP

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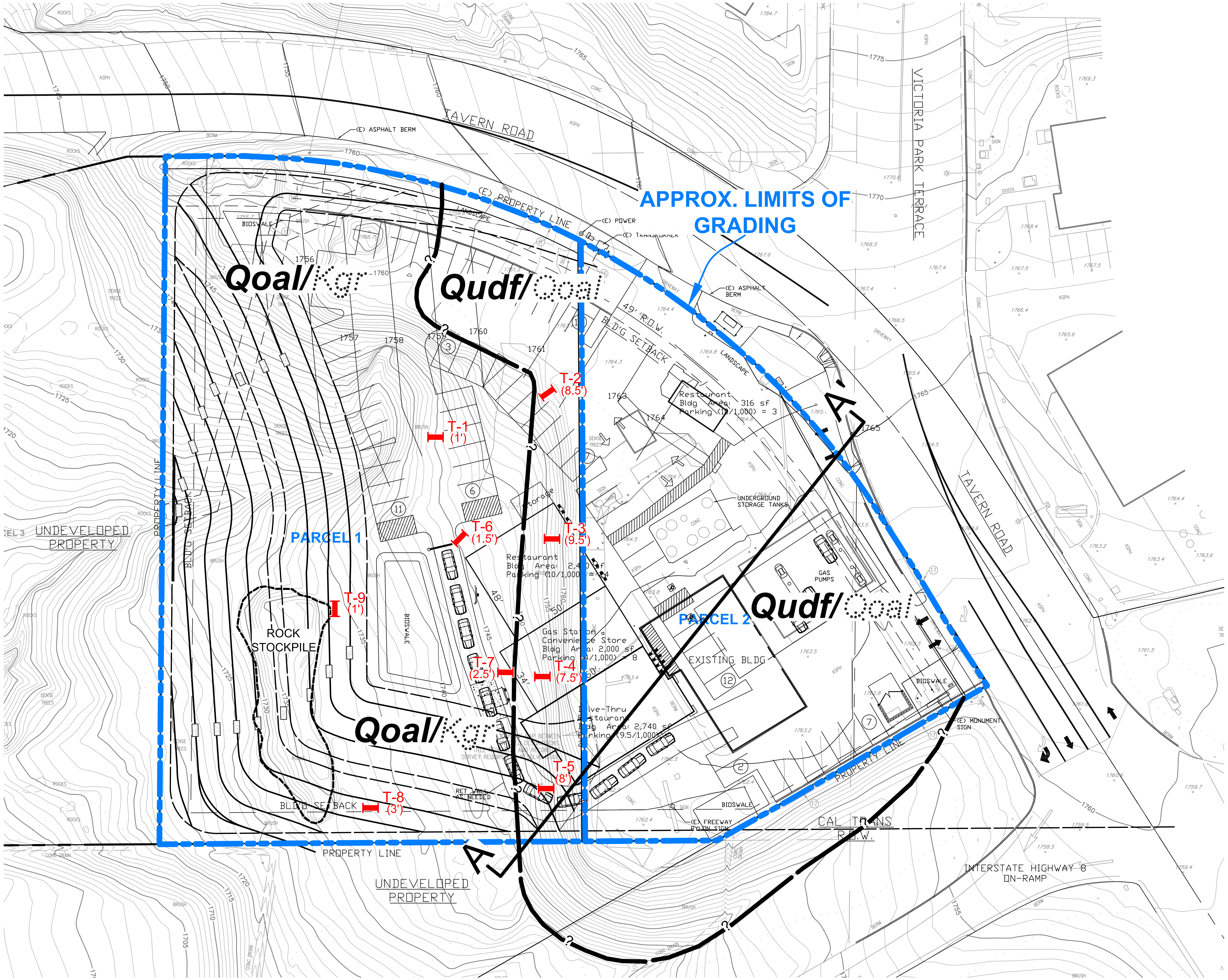
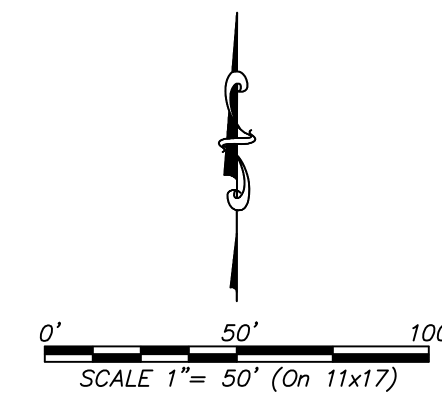
SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

LR / RA		DSK/GTYPD	DATE 12 - 21 - 2017	PROJECT NO. G2217 - 52 - 01	FIG. 1

Plotted:12/21/2017 9:53AM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\G2217-52-01 Shell Station Expansion\DETAILS\G2217-52-01 VicinityMap.dwg

SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA





GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qoal ....OLDER ALLUVIUM (Dotted Where Buried)

Kgr ....GRANITIC ROCK (Tonalite; Dotted Where Buried)

T-9 ....APPROX. LOCATION OF EXPLORATORY TRENCH

(9.5') ....APPROX. DEPTH TO GRANITIC ROCK (Feet)

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

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

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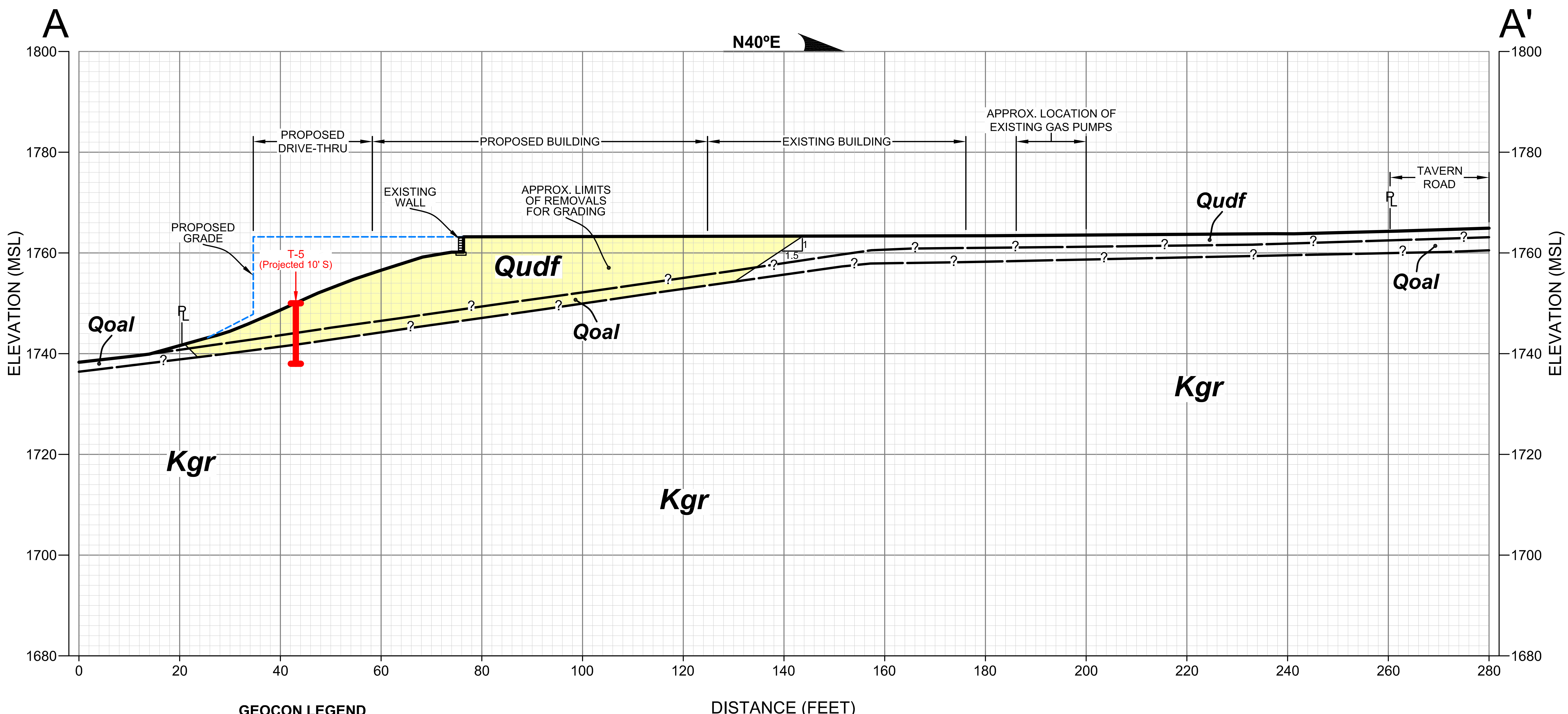
PROJECT NO. G2217 - 52 - 01

GEOLOGIC MAP

FIGURE 2
DATE 12 - 21 - 2017

SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA



GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qoal ....OLDER ALLUVIUM

Kgr ....GRANITIC ROCK (Tonalite)

T-5 ....APPROX. LOCATION OF EXPLORATORY TRENCH

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

GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 20' (Vert. = Horiz.)

DRAFT

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GEOLOGIC CROSS - SECTION

PROJECT NO. G2217 - 52 - 01
FIGURE 3
DATE 12 - 21 - 2017

Plotted:12/21/2017 9:49AM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\G2217-52-01 Shell Station Expansion\SHEETS\G2217-52-01 Cross-Section.dwg

Surficial Slope Stability Evaluation

Slope Height, H (feet)	∞	
Vertical Depth of Stauration, Z (feet)	3	
Slope Inclination	2.00	:1
Slope Inclination, I (degrees)	26.6	
Unit Weight of Water, γW (pcf)	62.4	
Total Unit Weight of Soil, γ_T (pcf)	125	
Friction Angle, ϕ (degrees)	28	
Cohesion, C (psf)	500	
Factor of Safety = $(C+(\gamma_T-\gamma_W)Z \cos^2 i \tan\phi)/(\gamma_T Z \sin i \cos i)$	3.87	

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

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

Slope Stability Evaluation

Slope Height, H (feet)	45	
Slope Inclination	2.0	:1
Total Unit Weight of Soil, γ_T (pcf)	125	
Friction Angle, ϕ (degrees)	28	
Cohesion, C (psf)	500	
$\gamma_{C\phi} = (\gamma H \tan\phi)/C$	6.0	
N_{Cf} (from Chart)	22.5	
Factor of Safety = $(N_{Cf}C)/(\gamma H)$	2.00	

References: (1) Janbu, N. *Stability Analysis of Slopes with Dimensionless Parameters,* Harvard Soil Mechanics, Series No. 46, 1954.

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

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SW / SW

SLOPE STABILITY ANALYSIS - PROPOSED FILL SLOPES

SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

DATE 12-21-2017 | PROJECT NO. G2217-52-01 | FIG. 4

Surficial Slope Stability Evaluation

Slope Height, H (feet)	∞	
Vertical Depth of Stauration, Z (feet)	3	
Slope Inclination	2.00	:1
Slope Inclination, I (degrees)	26.6	
Unit Weight of Water, γW (pcf)	62.4	
Total Unit Weight of Soil, γ_T (pcf)	125	
Friction Angle, ϕ (degrees)	26	
Cohesion, C (psf)	200	
Factor of Safety = $(C+(\gamma_T-\gamma_W)Z\cos^2 i\tan\phi)/(\gamma_T Z\sin i\cos i)$	1.82	

References:

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

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

Slope Stability Evaluation

Slope Height, H (feet)	30	
Slope Inclination	2.0	:1
Total Unit Weight of Soil, γ_T (pcf)	125	
Friction Angle, ϕ (degrees)	26	
Cohesion, C (psf)	200	
$\gamma_{C\phi} = (\gamma H\tan\phi)/C$	9.1	
N_{Cf} (from Chart)	30	
Factor of Safety = $(N_{Cf}C)/(\gamma H)$	1.60	

References:

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

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

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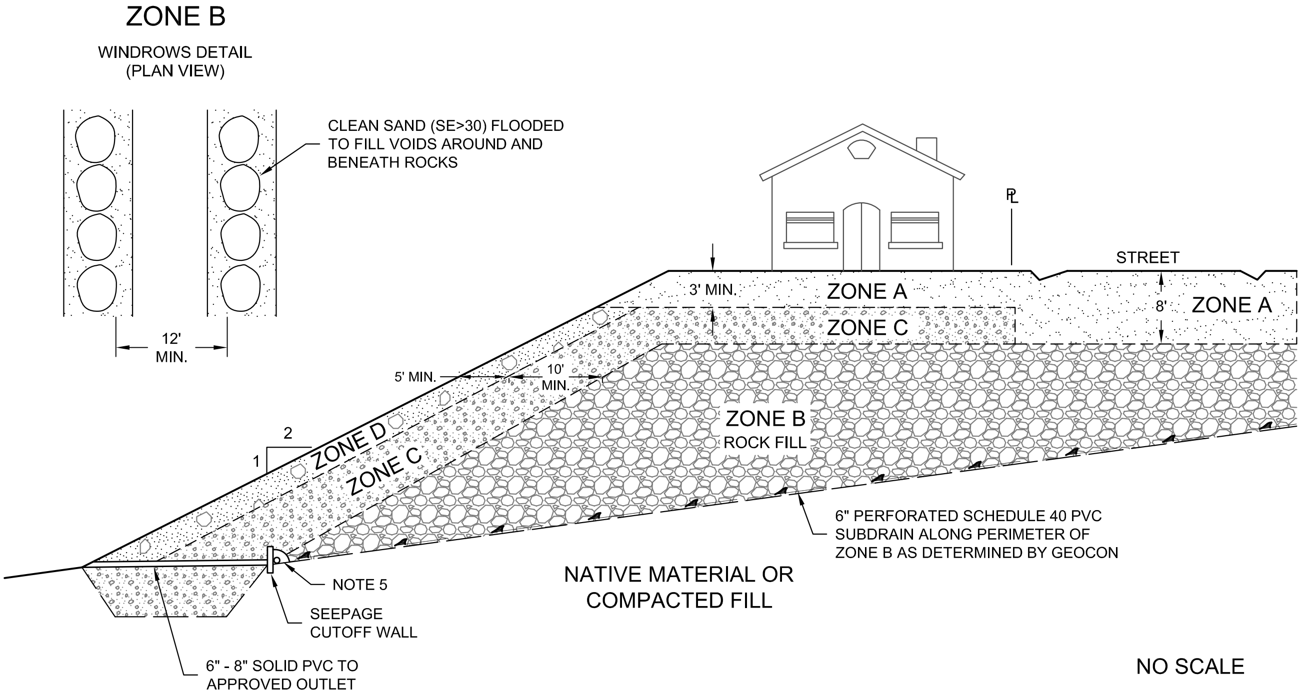
SLOPE STABILITY ANALYSIS - EXISTING FILL SLOPES

SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

SW / SW	DATE 12-21-2017	PROJECT NO. G2217-52-01	FIG. 5



LEGEND

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

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

ZONE C: ROCKS UP TO 2 FEET IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED SOIL FILL WITHIN BUILDING PADS AND SLOPE AREAS ONLY.

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

NOTES

1. COMPACTED SOIL FILL IN UPPER 8 FEET SHALL CONTAIN AT LEAST 40 PERCENT SOIL PASSING THE 3/4 - INCH SIEVE (BY WEIGHT) AND IN THE UPPER 3 FEET OF PAD GRADE AT LEAST 20% SOIL PASSING THE NO. 4 SIEVE (BY WEIGHT) AND COMPACTED IN ACCORDANCE WITH SPECIFICATIONS FOR STRUCTURAL FILL.
2. CONTINUOUS OBSERVATION REQUIRED BY GEOCON DURING ROCK PLACEMENT.
3. ROCK FILL (LESS THAN 40 PERCENT SOIL SIZES) MAY BE PERMITTED IN DESIGNATED AREAS UPON THE RECOMMENDATION OF THE GEOTECHNICAL ENGINEER.
4. DEPTH OF ZONE A SHOULD BE AT LEAST 8 FEET AND EXTENDED TO AT LEAST 2 FEET BELOW DEEPEST UTILITY WITHIN ROADWAYS.
5. 6" PERFORATED SCHEDULE 40 PVC SUBDRAIN ALONG THE TOE AND PORTIONS OF THE PERIMETER OF ZONE B.
6. BASE OF ZONE B SHOULD SLOPE A MINIMUM OF 3 PERCENT.

OVERSIZE ROCK DISPOSAL DETAIL

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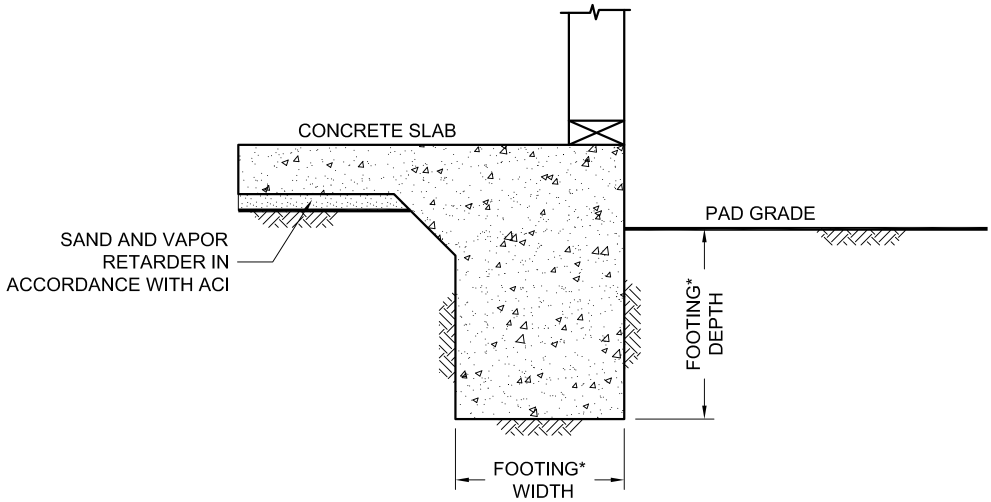
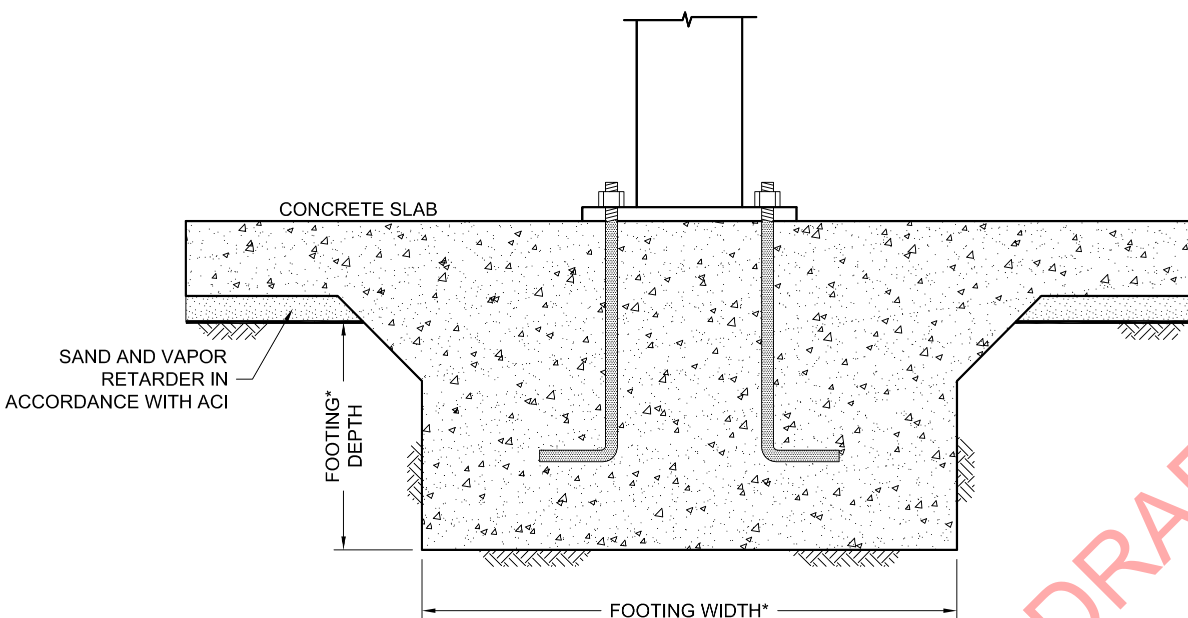
SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

LR / RA		DSK/GTYPD	DATE 12 - 21 - 2017	PROJECT NO. G2217 - 52 - 01	FIG. 6





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

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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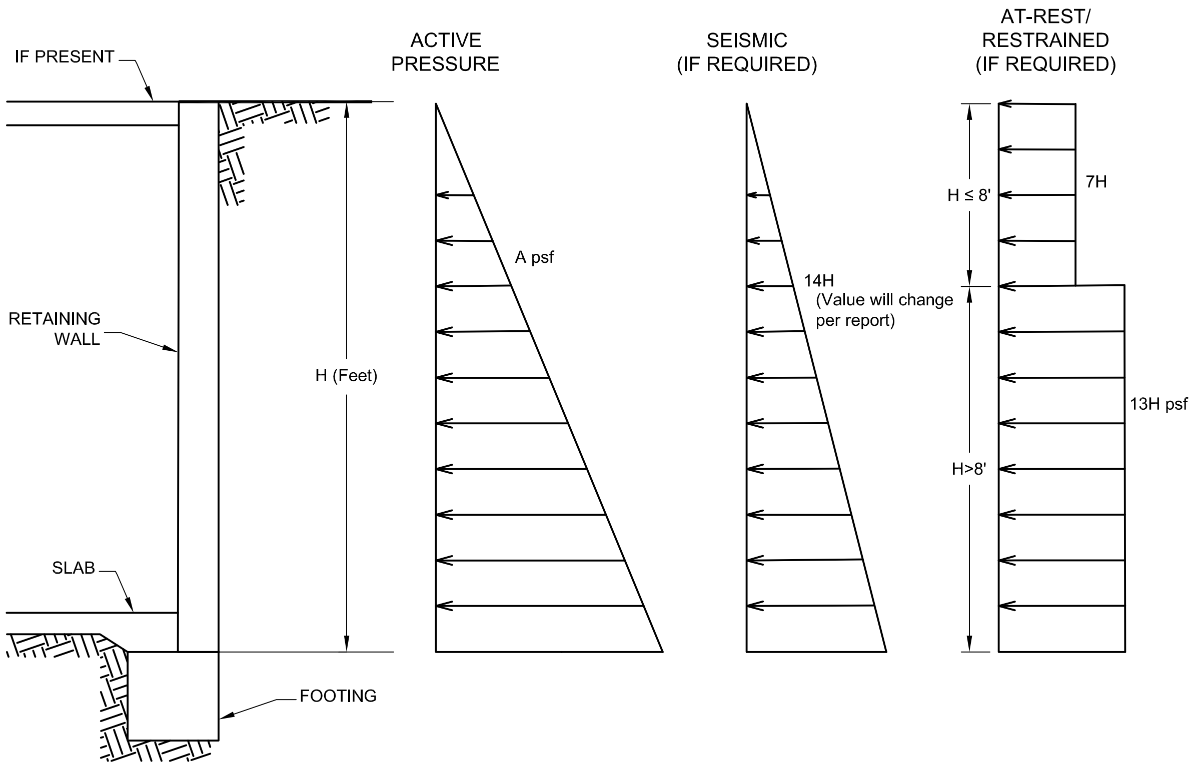
SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

LR / RA		DSK/GTYPD	DATE 12 - 21 - 2017	PROJECT NO. G2217 - 52 - 01	FIG. 7



ACTIVE PRESSURE, A (psf)		
EXPANSION INDEX, EI	LEVEL BACKFILL	2:1 SLOPING BACKFILL
EI ≤ 50	35	50
EI ≤ 90	40	55

NOTES:

1..... A SURCHARGE OF 2 FEET OF SOIL (250 PSF VERTICAL LOAD) SHOULD BE ADDED TO THE DESIGN OF THE WALL WHERE TRAFFIC LOADS ARE WITHIN A HORIZONTAL DISTANCE EQUAL TO ⅔ THE WALL HEIGHT. OTHER SURCHARGES SHOULD BE APPLIED, AS APPLICABLE.

2..... EXPANSION INDEX GREATER THAN 50/90 SHOULD NOT BE USED FOR WALL BACKFILL PER REPORT.

3..... RETAINING WALLS SHOULD BE PROPERLY DRAINED AND WATER PROOFED.

4..... THE PROJECT STRUCTURAL ENGINEER SHOULD EVALUATE THE WALL LOADING COMBINATIONS.

DRAFT

NO SCALE

RETAINING WALL LOADING DIAGRAM

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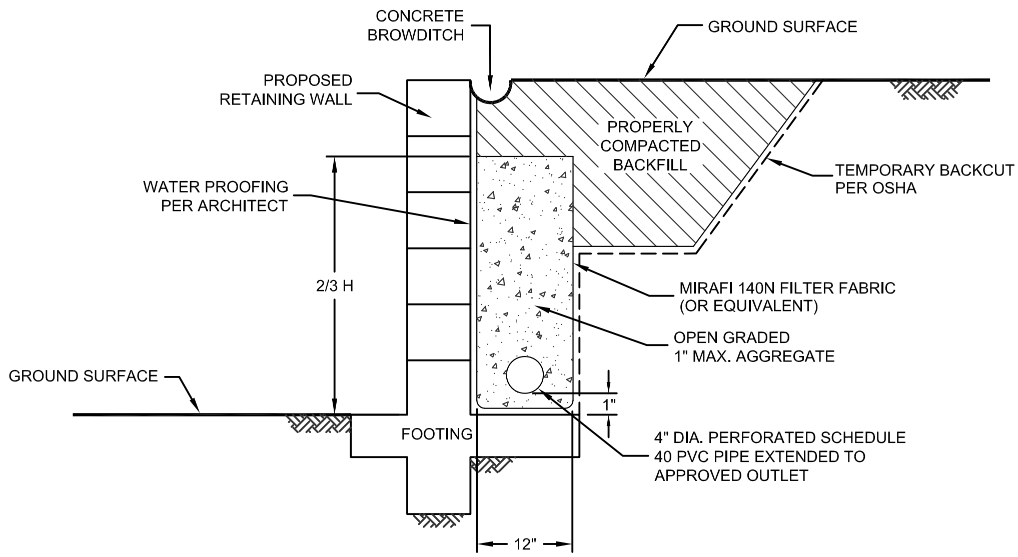
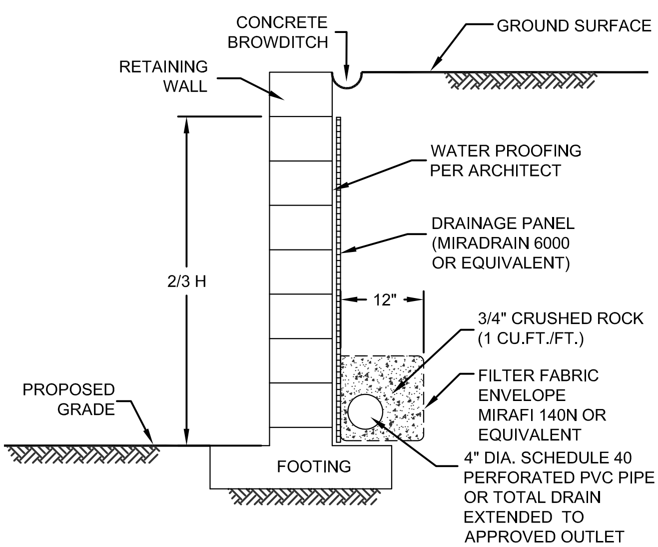
SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

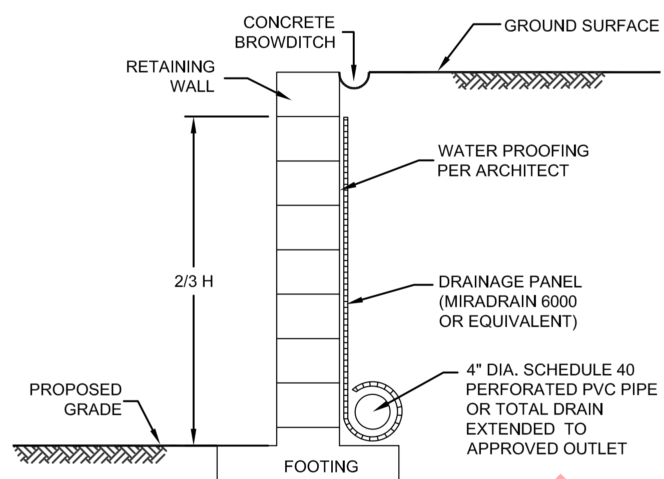
LR / RA		DSK/GTYPD	DATE 12 - 21 - 2017	PROJECT NO. G2217 - 52 - 01	FIG. 8





NOTE :

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



NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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SHELL STATION EXPANSION
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LR / RA		DSK/GTYPD	DATE 12 - 21 - 2017	PROJECT NO. G2217 - 52 - 01	FIG. 9

APPENDIX

A

APPENDIX A

FIELD INVESTIGATION

We performed the fieldwork for our investigation on November 29, 2017. Our subsurface exploration consisted of excavating nine exploratory trenches to a maximum depth of approximately 12 feet using a track-mounted John Deere 555G backhoe. The locations of the trenches are shown on the Geologic Map, Figure 2. The trench logs, and an explanation of the geologic units encountered are presented on Figures A-1 through A-9. We located the trenches in the field using existing reference points; therefore, actual locations may deviate slightly.

We obtained bulk and chunk samples at appropriate intervals and transported them to the laboratory for testing. We estimated elevations shown on the trench logs from the provided topographic map. Each excavation was backfilled with the soil cuttings generated during excavation.

We visually examined, classified, and logged the soil encountered in the trenches in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). The logs depict the soil and geologic conditions observed and the depth at which samples were obtained.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 1 ELEV. (MSL.) 1746' DATE COMPLETED 11-29-2017 EQUIPMENT TRACK-MOUNTED JD 555G BACKHOE BY: L. RODRIGUEZ	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Loose, dry to damp, yellowish to reddish brown, Silty, fine to coarse SAND; trace rootlets			
2					**GRANITIC ROCK (Kgr)** Completely weathered, yellowish to grayish brown, moderately weak to weak TONALITE; excavates as fine to coarse SAND with gravel-sized rock fragments			
					TRENCH TERMINATED AT 3 FEET Groundwater not encountered			

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

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

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

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TRENCH T 2

ELEV. (MSL.) **1764'** DATE COMPLETED **11-29-2017**

EQUIPMENT **TRACK-MOUNTED JD 555G BACKHOE** BY: **L. RODRIGUEZ**

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	**UNDOCUMENTED FILL (Qudf)** Loose, dry to damp, light grayish to yellowish brown, Silty, fine to coarse SAND; organics, gravel, roots, metal and plastic debris in upper 4 feet			
2					-Mostly angular gravel up to 1" in diameter from 2 to 3 feet			
					-Trace boulders up to approx. 6" in diameter			
4					-Becomes loose to medium dense, reddish brown			
6	T2-1			SM	**OLDER ALLUVIUM (Qoal)** Dense, damp, reddish brown, Silty, fine to coarse SAND			
8					**GRANITIC ROCK (Kgr)** Completely weathered, moist, yellowish to grayish brown, moderately weak TONALITE; excavates as fine to coarse SAND with gravel sized rock fragments			
10								

TRENCH TERMINATED AT 10.5 FEET
Groundwater not encountered

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

G2217-52-01.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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TRENCH T 3

ELEV. (MSL.) 1759' DATE COMPLETED 11-29-2017

EQUIPMENT TRACK-MOUNTED JD 555G BACKHOE BY: L. RODRIGUEZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	T3-1			SM	**UNDOCUMENTED FILL (Qudf)** Loose to medium dense, dry to damp, light yellowish to reddish brown, Silty, fine to coarse SAND; trace clasts granitic rock; trace rootlets			
2					-H.P. approx. 4.5 tsf			
4					-Becomes light yellowish to grayish brown; H.P. approx. 2.5 tsf			
6								
8				SM	**OLDER ALLUVIUM (Qoal)** Dense to very dense, dry to damp, reddish brown, Silty, fine to coarse SAND; micaceous			
10					**GRANITIC ROCK (Kgr)** Highly weathered, yellowish to grayish brown, moderately weak TONALITE; excavates as fine to coarse SAND with gravel sized rock fragments			
12					TRENCH TERMINATED AT 12 FEET Groundwater not encountered			

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

G2217-52-01.GPJ

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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 4 ELEV. (MSL.) 1753' DATE COMPLETED 11-29-2017 EQUIPMENT TRACK-MOUNTED JD 555G BACKHOE BY: L. RODRIGUEZ	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED FILL (Qudf)** Loose to medium dense, damp, light reddish brown, Silty, fine to coarse SAND; trace organics; trace gravel sized rock fragments; H.P.=3.0 tsf			
2	T4-1				-Becomes loose, light grayish brown; H.P.=0.5 tsf			
4					-Becomes medium dense, dark brown; H.P.=4.0 tsf			
6								
	T4-2			SM	**OLDER ALLUVIUM (Qoal)** Dense, dry to damp, reddish brown, Silty, fine to coarse SAND; pinhole voids; micaceous		109.7	9.0
8					**GRANITIC ROCK (Kgr)** Highly weathered, yellowish to grayish brown, moderately weak TONALITE; excavates as Silty, fine to coarse SAND with gravel sized rock fragments			
					TRENCH TERMINATED AT 9 FEET Groundwater not encountered			

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

G2217-52-01.GPJ

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

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

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PROJECT NO. G2217-52-01

TRENCH T 5

ELEV. (MSL.) **1750'** DATE COMPLETED **11-29-2017**

EQUIPMENT **TRACK-MOUNTED JD 555G BACKHOE** BY: **L. RODRIGUEZ**

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	**UNDOCUMENTED FILL (Qudf)** Loose to medium dense, damp, light reddish brown, Silty, fine to coarse SAND; trace clasts granitic rock; trace organics			
	T5-1							
2								
	T5-2				-Becomes medium dense, moist, dark reddish brown			
4								
				SM	**OLDER ALLUVIUM (Qoal)** Dense, moist, reddish brown, Silty, fine to coarse SAND			
6								
					-Trace cobble approx. 7" in diameter			
8					**GRANITIC ROCK (Kgr)** Completely weathered, reddish to grayish brown, weak TONALITE; excavates as Silty, fine to coarse SAND with gravel-sized rock fragments			
10								
					-Becomes highly weathered, yellowish to grayish brown			
12					TRENCH TERMINATED AT 12 FEET Groundwater not encountered			

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

G2217-52-01.GPJ

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

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

TRENCH T 6

ELEV. (MSL.) **1746'** DATE COMPLETED **11-29-2017**

EQUIPMENT **TRACK-MOUNTED JD 555G BACKHOE** BY: **L. RODRIGUEZ**

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	**OLDER ALLUVIUM (Qoal)** Dense to very dense, damp, reddish brown, Silty, fine to coarse SAND; clasts granitic rock			
2					**GRANITIC ROCK (Kgr)** Highly weathered, yellowish to grayish brown, moderately weak TONALITE; excavates as fine to coarse SAND with gravel sized rock fragments -Septic system with 2" diameter gravel encountered at 2' at southwest end of trench			
4					TRENCH TERMINATED AT 4 FEET Groundwater not encountered			

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

G2217-52-01.GPJ

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

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

PROJECT NO. G2217-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 7 ELEV. (MSL.) 1746' DATE COMPLETED 11-29-2017 EQUIPMENT TRACK-MOUNTED JD 555G BACKHOE BY: L. RODRIGUEZ	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 – 2				SM	**OLDER ALLUVIUM (Qoal)** Dense to very dense, damp, reddish brown, Silty, fine to coarse SAND; clasts granitic rock			
2 – 4					**GRANITIC ROCK (Kgr)** Highly weathered, yellowish to grayish brown, moderately weak TONALITE; excavates as fine to coarse SAND with gravel sized rock fragments			
4					TRENCH TERMINATED AT 4 FEET Groundwater not encountered			

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

G2217-52-01.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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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 8** ELEV. (MSL.) **1733.5'** DATE COMPLETED **11-29-2017** EQUIPMENT **TRACK-MOUNTED JD 555G BACKHOE** BY: **L. RODRIGUEZ**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Loose, dry, light brown, Silty, fine to coarse SAND; some organics			
2				SM	**OLDER ALLUVIUM (Qoal)** Dense to very dense, dry to damp, reddish brown, Silty, fine to coarse SAND; trace gravel			
4					**GRANITIC ROCK (Kgr)** Completely weathered, yellowish to grayish brown, weak TONALITE; excavates as fine to coarse SAND with gravel sized rock fragments -Septic system with 2" diameter gravel encountered at 3 feet at east end of trench			
					TRENCH TERMINATED AT 5 FEET Groundwater not encountered			

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

G2217-52-01.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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TRENCH T 9

ELEV. (MSL.) **1734.5'** DATE COMPLETED **11-29-2017**

EQUIPMENT **TRACK-MOUNTED JD 555G BACKHOE** BY: **L. RODRIGUEZ**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	T9-1			SM	**OLDER ALLUVIUM (Qoal)** Dense to very dense, damp, reddish brown, Silty, fine to coarse SAND; micaceous			
2					**GRANITIC ROCK (Kgr)** Completely to moderately weathered, grayish to yellowish brown, moderately strong TONALITE; excavates as gravel to cobble sized rock fragments			
					TRENCH TERMINATED AT 3 FEET Groundwater not encountered			

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

G2217-52-01.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.

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally currently accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for their in-place density and moisture content, maximum dry density and optimum moisture content, shear strength, expansion index, pH, resistivity, water-soluble sulfate characteristics, grain size and consolidation. Tables B-I through B-V and Figures B-1 and B-2 present the results of our laboratory tests. In addition, the in-place dry density and moisture content test results are presented on the trench logs in Appendix A.

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

Sample No.	Depth (Feet)	Description (Geologic Unit)	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T3-1	½-3	Yellowish to reddish brown, Silty fine to coarse SAND (Qudf)	131.9	9.2
T9-1	0-1½	Reddish brown, Silty fine to coarse SAND (Qvop/Kgr)	132.9	9.0

TABLE B-II
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080

Sample No.	Depth (feet)	Geologic Unit	Dry Density (pcf)	Moisture Content (%)		Peak [Ultimate[1]] Cohesion (psf)	Peak [Ultimate[1]] Angle of Shear Resistance (degrees)
				Initial	Final		
T3-1[2]	½-3	Qudf	118.8	8.9	15.3	625 [500]	30 [30]
T9-1[2]	0-1½	Qoal/Kgr	119.7	8.6	15.0	600 [525]	28 [28]

[1] Ultimate measured at 0.2-inch deflection.
[2] Sample Remolded.

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

Sample No.	Depth (feet)	Geologic Unit	Moisture Content (%)		Dry Density (pcf)	Expansion Index	ASTM Expansion Classification	2016 CBC Expansion Classification
			Before Test	After Test				
T3-1	½-3	Qudf	7.5	15.1	119.3	11	Non-Expansive	Very Low

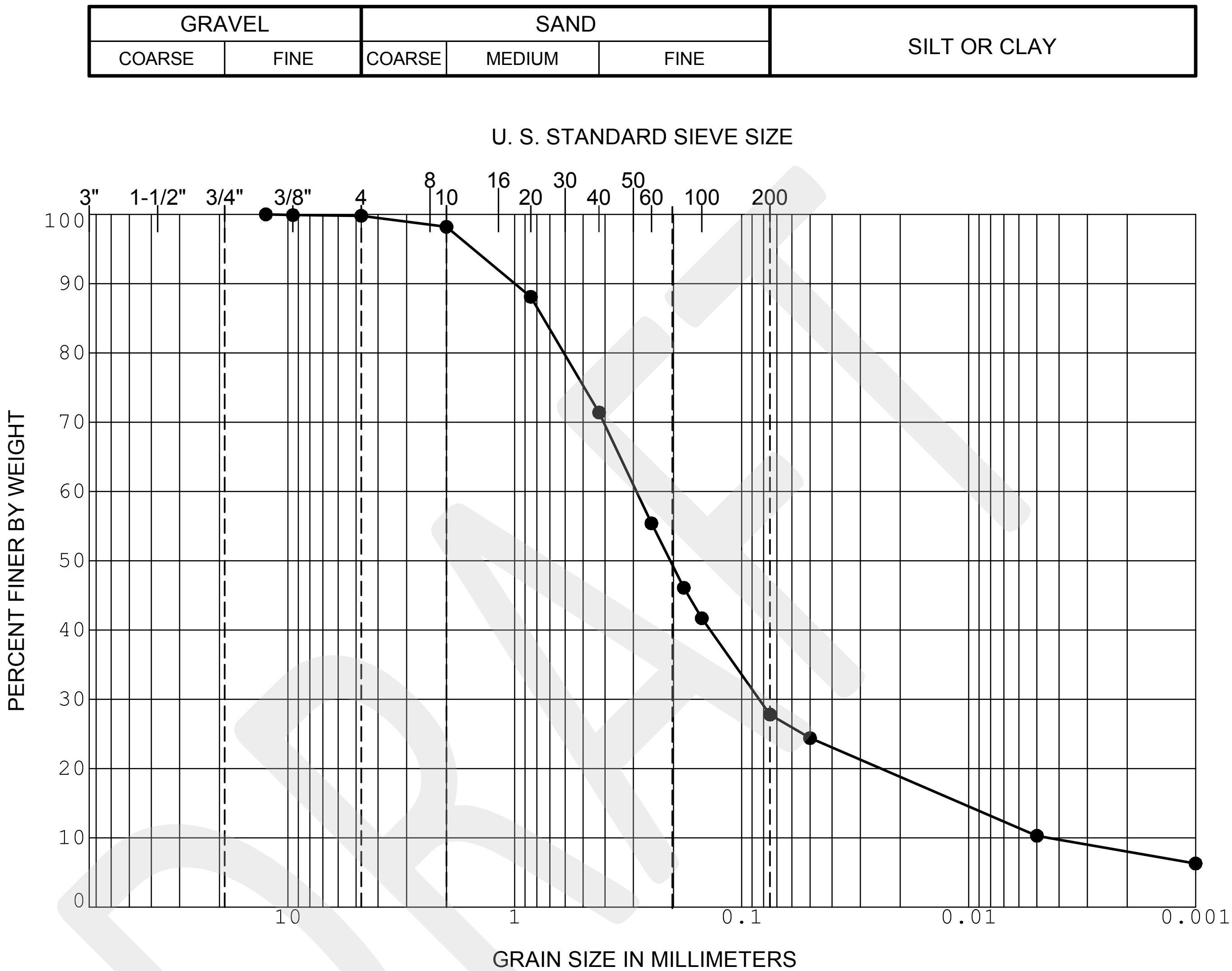
TABLE B-IV
SUMMARY OF LABORATORY PH AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	Depth (Feet)	Geologic Unit	pH	Minimum Resistivity (ohm-centimeters)
T3-1	½-3	Qudf	8.0	2800

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

Sample No.	Depth (Feet)	Geologic Unit	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
T3-1	½-3	Qudf	0.005	S0

PROJECT NO. G2217-52-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	T3-1	0.5	SM - Silty SAND				
⊠							
▲							

GRADATION CURVE

SHELL STATION EXPANSION

COUNTY OF SAN DIEGO, CALIFORNIA

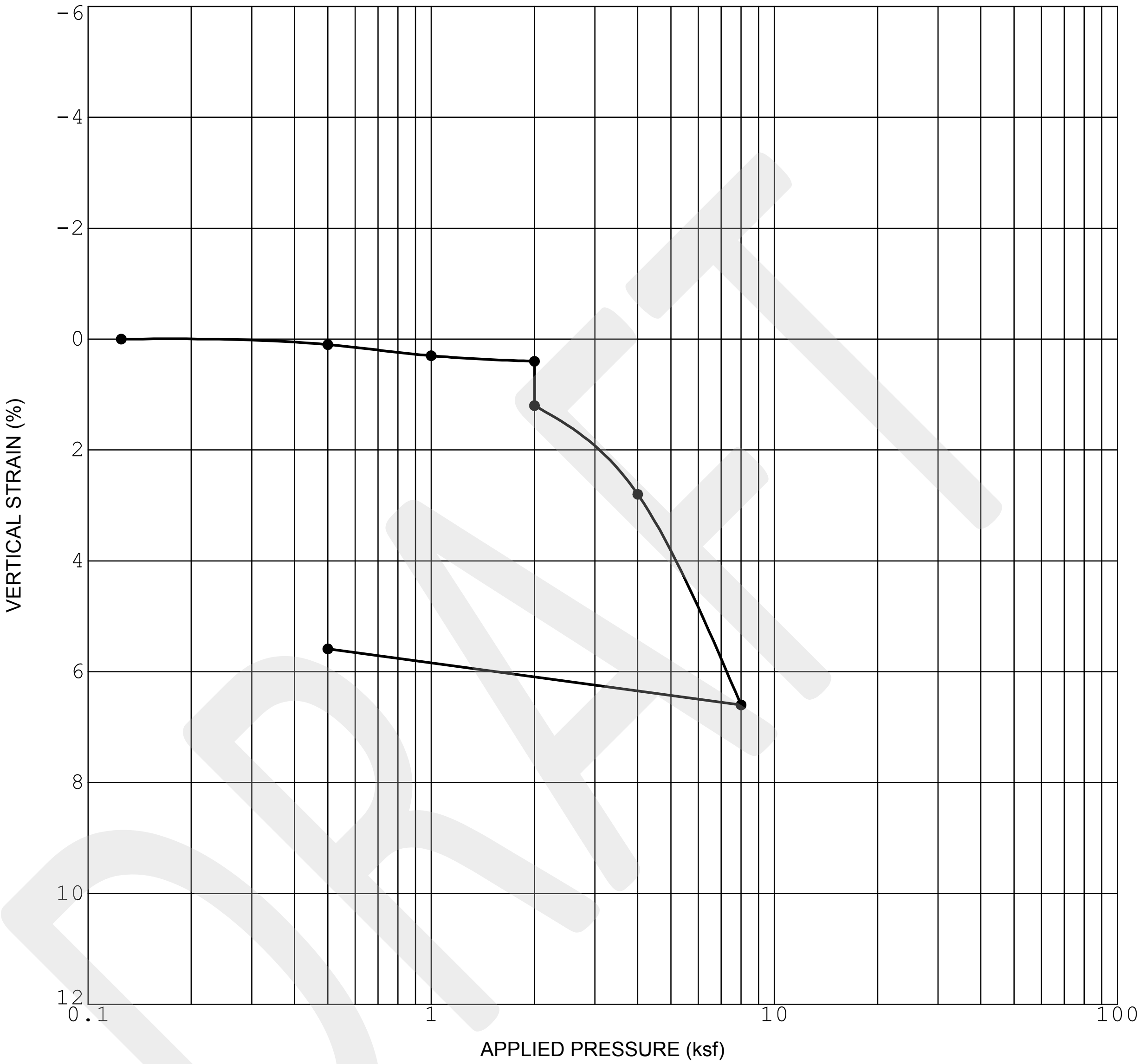
G2217-52-01.GPJ

Figure B-1

GEOCON

PROJECT NO. G2217-52-01

SAMPLE NO. T4-2



Initial Dry Density (pcf)	109.7
Initial Water Content (%)	9.0

Initial Saturation (%)	46.5
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

SHELL STATION EXPANSION

COUNTY OF SAN DIEGO, CALIFORNIA

G2217-52-01.GPJ

Figure B-2

GEOCON

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

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

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table C-I presents the descriptions of the hydrologic soil groups. If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

TABLE C-I
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 man-made fill and natural materials consisting of topsoil, Older Alluvium and Granitic Rock. The fill should be classified as Soil Group D. Table C-II presents the information from the USDA website for the subject property. The Hydrologic Soil Group Map, provided at the end of this appendix, presents output from the USDA website showing the limits of the soil units.

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

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group
Cienaba Rocky Coarse Sandy Loam, 9 to 30 Percent Slopes	CmE2	100	D

In-Situ Testing

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

TABLE C-III
SOIL PERMEABILITY DEFINITIONS

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

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

We did not perform infiltration testing at the site because the site conditions and characteristics of the underlying soil at the site precludes allowing infiltration at the site. Full and partial infiltration is infeasible at the site due to the thickness and soil characteristics of the existing and planned fill at the site, the presence of existing slopes adjacent to the site, and the planned fill slopes. A detailed discussion is presented herein.

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

TABLE C-IV
INFILTRATION CATEGORIES

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

*Using a Factor of Safety of 2.

Groundwater Elevations

We did not encounter groundwater or seepage during the site investigation. We expect groundwater exists at depths greater than 100 feet below existing grades.

New or Existing Utilities

Utilities will be constructed within the site boundaries. Full or partial infiltration should not be allowed in the areas of the utilities to help prevent potential damage/distress to improvements. Mitigation measures to prevent water from infiltrating the utilities consist of setbacks, installing cutoff walls around the utilities and installing subdrains and/or installing liners.

Existing and Planned Structures

Existing roadways and Interstate 8 exist adjacent to the site. Water should not be allowed to infiltrate in areas where it could affect the neighboring properties and existing adjacent structures, improvements and roadways. Mitigation for existing structures consists of not allowing water infiltration within a 1:1 plane from existing foundations and extending the infiltration areas at least 10 feet below the existing adjacent improvements.

Slopes Hazards

Existing fill and naturally occurring descending slopes exist on the site and to the west and south of the property extending to an adjacent property and Interstate 8. If infiltration is allowed adjacent to the existing slopes at the site, water migration and the resulting seepage forces can negatively affect the stability of the slopes and cause erosion. The existing fill and formational materials possess limited vertical infiltration characteristics and water allowed to infiltrate on the site would migrate laterally to the adjacent properties, roadways and Interstate 8. Infiltration devices should not be installed adjacent to slopes unless they are lined, possess a minimum setback distance of 50 feet, or extend below the height of the slope. Due to the heights of the existing and proposed slopes, we expect the basins will be lined for the planned property.

Storm Water Evaluation Narrative

The western portion of the site (Parcel 1) is underlain by older alluvium overlying Granitic Rock, consists of naturally occurring descending slopes and an existing natural drainage, and is also bordered to the west and south by existing naturally occurring slopes. Infiltration should not be allowed adjacent to slopes. In addition, the existing older alluvium and Granitic Rock consists of very dense materials that are not typically conducive to infiltration. Therefore, due to the sloping conditions, the adjacent slopes to the west and south, and the very dense characteristics of the older alluvium and Granitic Rock, infiltration should not be allowed within a majority of the western half of the site.

The existing gas station site (Parcel 2) located on the eastern portion of the site is underlain by varying depths of undocumented fill overlying older alluvium and Granitic Rock. An existing fill slope with a height up to approximately 25 feet extends along a majority of the western edge and southern edge of the gas station site. We expect fills up to approximately 20 to 25 feet exist below the southern end of the gas station site. Infiltration should not be allowed within 50 feet of the existing fill slope or in areas underlain by greater than 5 feet of fill. In addition, the area on the gas station site outside the 50-foot setback area from the existing slope consists of existing improvements including an underground storage tank, gas pumps and several underground utilities that will remain. Several underground utilities also exist along the adjacent Tavern Road. Infiltration should not be allowed in areas of the existing improvements or adjacent to the existing roadway. Therefore, due to the fill slope, fill thicknesses, and presence of existing improvements, infiltration should not be allowed on the gas station site.

We opine the property is considered infeasible to full and partial infiltration. The planned storm water devices should be properly lined to prevent water migration into the underlying soil and to prevent distress to the adjacent existing and proposed slopes and adjacent properties and improvements.

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 or Form I-8) worksheet information to help evaluate the potential for infiltration on the property. The attached Worksheet C.4-1 presents the completed information for the submittal process.

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

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

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

Based on our geotechnical investigation and the previous table, Table C-VI 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-VI
FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	3	0.75
Predominant Soil Texture	0.25	1	0.25
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/ Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.75

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

Categorization of Infiltration Feasibility Condition			Worksheet C.4-1
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: We did not perform infiltration tests at the site because the site conditions and characteristics of the underlying soil at the site precludes allowing infiltration. The fill materials and existing formational older alluvium and Granitic Rock should be classified as Soil Group D. Materials characterized as a Hydrologic Soil Group D typically possess infiltration rates less than 0.5 inches per hour. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: The western portion of the site (Parcel 1) is underlain by older alluvium overlying Granitic Rock, consists of naturally occurring descending slopes and an existing natural drainage, and is also bordered to the west and south by existing naturally occurring slopes. Infiltration should not be allowed adjacent to slopes. In addition, the existing older alluvium and Granitic Rock consists of very dense materials that are not typically conducive to infiltration. Therefore, due to the sloping conditions, the adjacent slopes to the west and south, and the very dense characteristics of the older alluvium and Granitic Rock, infiltration should not be allowed within a majority of the western half of the site. The existing gas station site (Parcel 2) located on the eastern portion of the site is underlain by varying depths of undocumented fill overlying older alluvium and Granitic Rock. An existing fill slope with a height up to approximately 25 feet extends along a majority of the western edge and southern edge of the gas station site. We expect fills up to approximately 20 to 25 feet exist below the southern end of the gas station site. Infiltration should not be allowed within 50 feet of the existing fill slope or in areas underlain by greater than 5 feet of fill. In addition, the area on the gas station site outside the 50-foot setback area from the existing slope consists of existing improvements including an underground storage tank, gas pumps and several underground utilities that will remain. Several underground utilities also exist along the adjacent Tavern Road. Infiltration should not be allowed in areas of the existing improvements or adjacent to the existing roadway. Therefore, due to the fill slope, fill thicknesses, and presence of existing improvements, infiltration should not be allowed on the gas station site.			

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not encounter groundwater or seepage during the site investigation. We expect groundwater exists at depths greater than 100 feet below existing grades. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We do not expect infiltration will cause water balance issues such as seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2		No Full Infiltration

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

Worksheet C.4-1 Page 3 of 4

<u>Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria</u>

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

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

Provide basis:

We did not perform infiltration tests at the site because the site conditions and characteristics of the underlying soil at the site precludes allowing infiltration. The fill materials and existing formational older alluvium and Granitic Rock should be classified as Soil Group D. Materials characterized as a Hydrologic Soil Group D typically possess infiltration rates less than 0.05 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

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

Provide basis:

The western portion of the site (Parcel 1) is underlain by older alluvium overlying Granitic Rock, consists of naturally occurring descending slopes and an existing natural drainage, and is also bordered to the west and south by existing naturally occurring slopes. Infiltration should not be allowed adjacent to slopes. In addition, the existing older alluvium and Granitic Rock consists of very dense materials that are not typically conducive to infiltration. Therefore, due to the sloping conditions, the adjacent slopes to the west and south, and the very dense characteristics of the older alluvium and Granitic Rock, infiltration should not be allowed within a majority of the western half of the site.

The existing gas station site (Parcel 2) located on the eastern portion of the site is underlain by varying depths of undocumented fill overlying older alluvium and Granitic Rock. An existing fill slope with a height up to approximately 25 feet extends along a majority of the western edge and southern edge of the gas station site. We expect fills up to approximately 20 to 25 feet exist below the southern end of the gas station site. Infiltration should not be allowed within 50 feet of the existing fill slope or in areas underlain by greater than 5 feet of fill. In addition, the area on the gas station site outside the 50-foot setback area from the existing slope consists of existing improvements including an underground storage tank, gas pumps and several underground utilities that will remain. Several underground utilities also exist along the adjacent Tavern Road. Infiltration should not be allowed in areas of the existing improvements or adjacent to the existing roadway. Therefore, due to the fill slope, fill thicknesses, and presence of existing improvements, infiltration should not be allowed on the gas station site.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not encounter groundwater or seepage during the site investigation. We expect groundwater exists at depths greater than 100 feet below existing grades. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not provide a study regarding water rights. However, these rights are not typical in the San Diego County area. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**		No Infiltration

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

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

SHELL STATION EXPANSION
COUNTY OF SAN DIEGO, CALIFORNIA

PROJECT NO. G2217-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

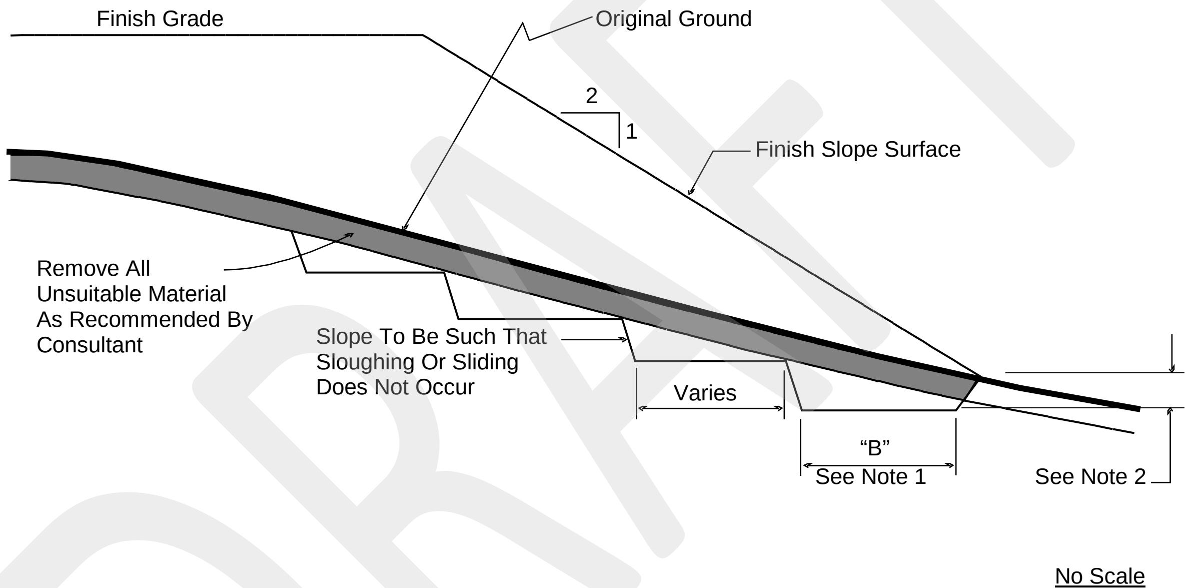
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

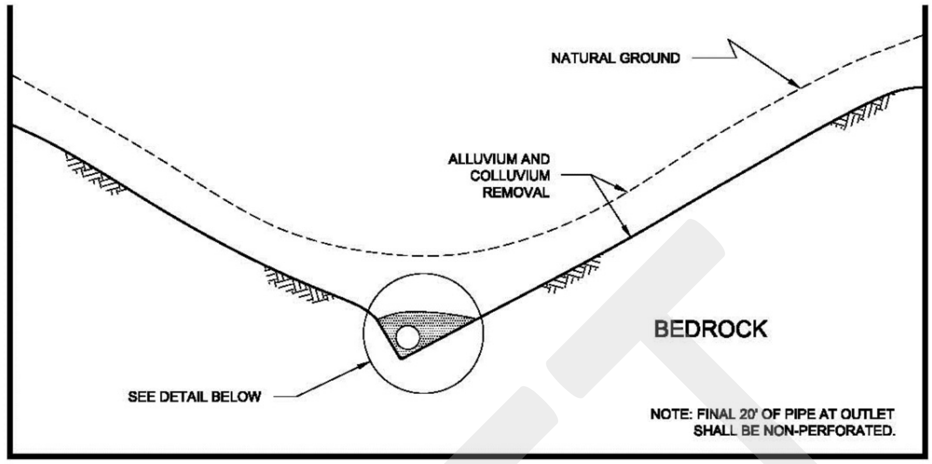
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

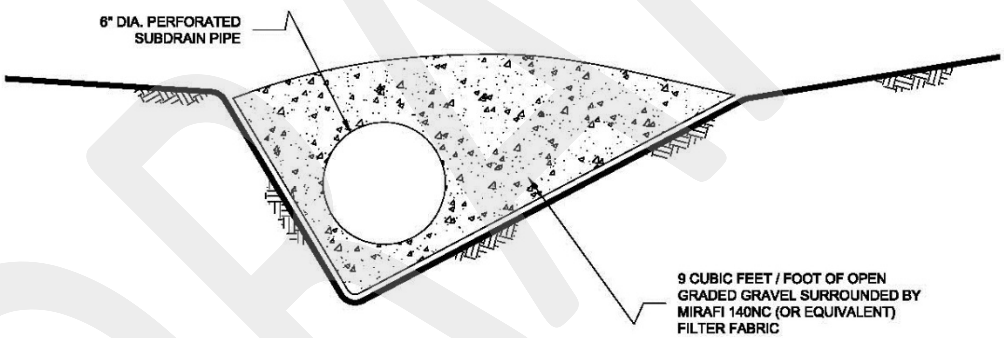
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

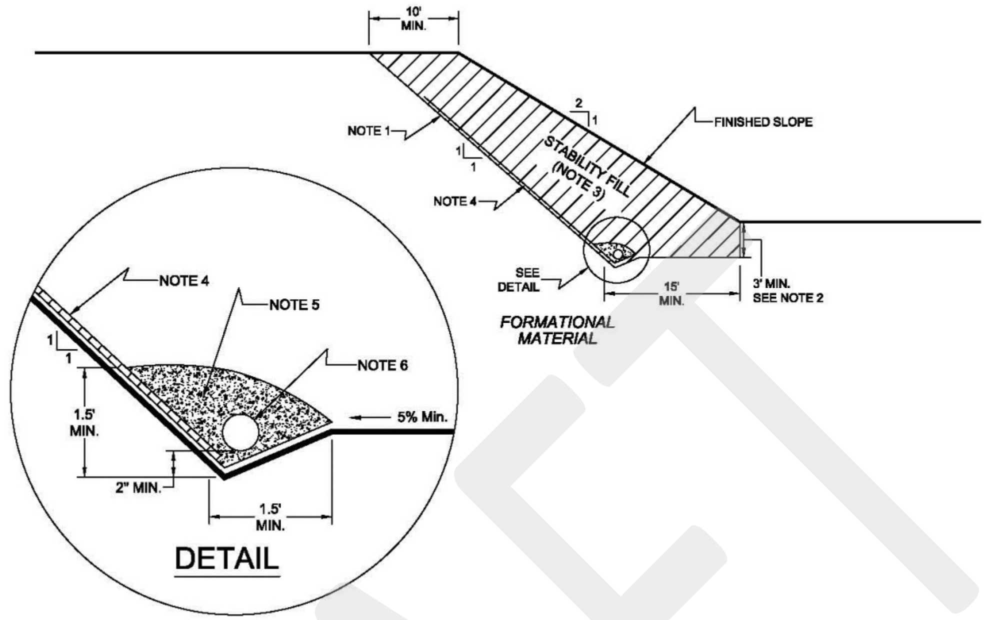
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

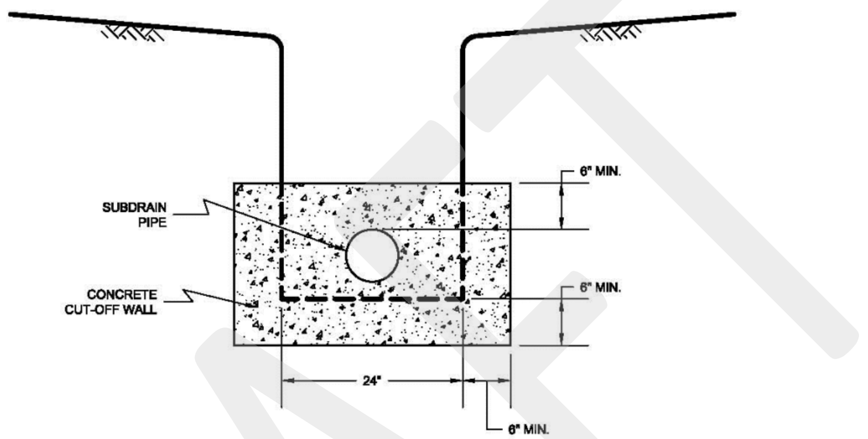
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

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

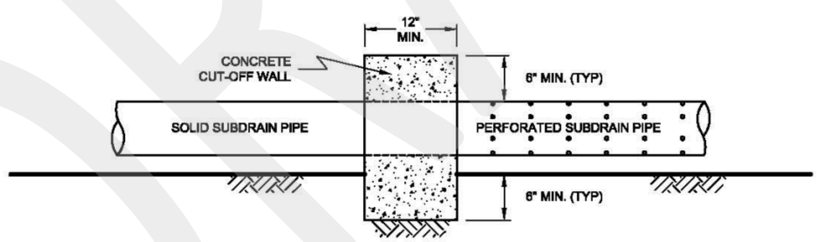
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

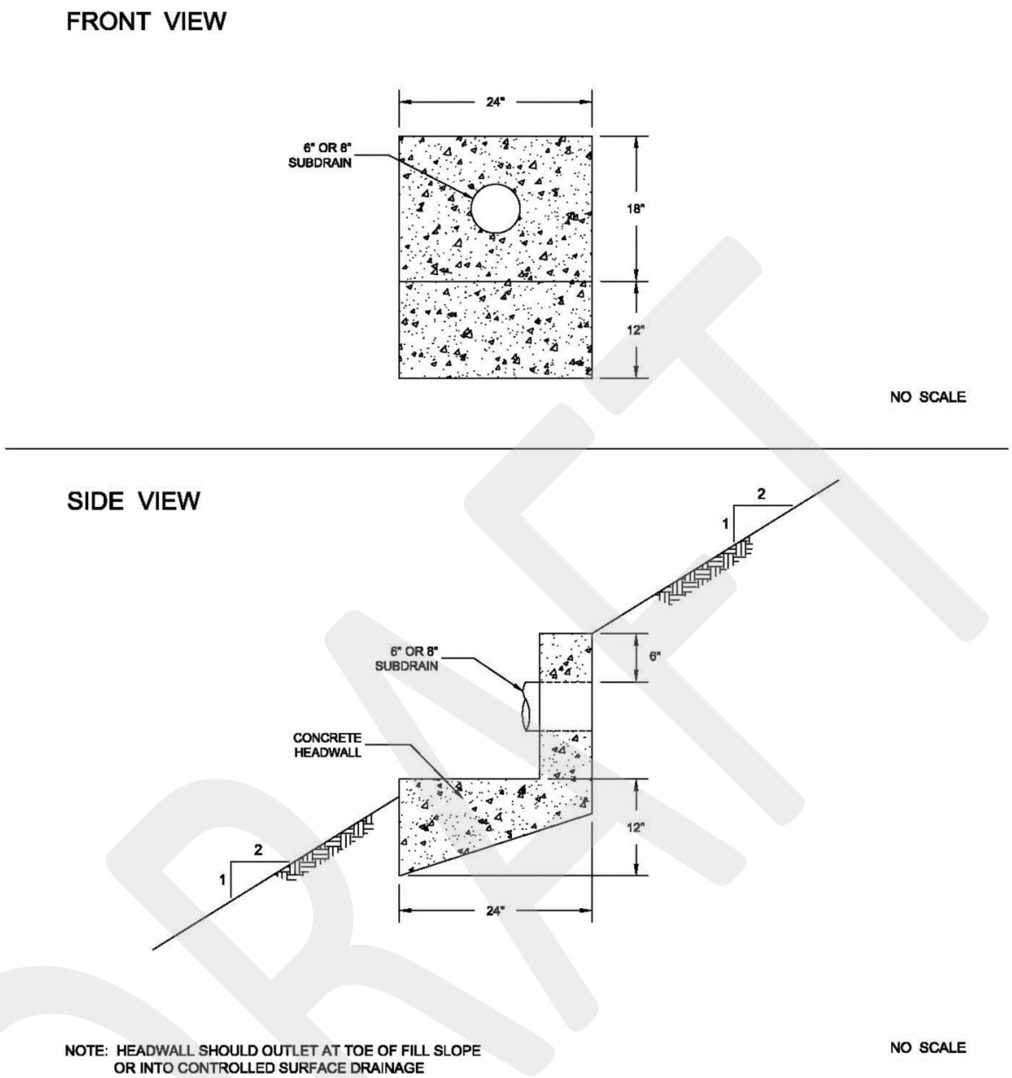
SIDE VIEW



NO SCALE

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

TYPICAL HEADWALL DETAIL



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. *2016 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2015 International Building Code,* prepared by California Building Standards Commission, dated July, 2016.

2. *ACI 318-14, Building Code Requirements for Structural Concrete* and *Commentary on Building Code Requirements for Structural Concrete,* prepared by the American Concrete Institute, dated September, 2014.

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

4. Anderson, J. G., T. K. Rockwell, and D. C. Agnew, *Past and Possible Future Earthquakes of Significance to the San Diego Region*: Earthquake Spectra, 1989, v.5, no. 2, p.299-333.

5. Boore, D. M. and G. M Atkinson, *Ground Motion Prediction Equations for the Average Horizontal Component of PGA, PVG, and 5%-Ramped PSA at Spectral Periods Between 0.01s and 10.0s,* Earthquake Spectra, Vol. 24, Issue I, February 2008.

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

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

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

9. Lindvall, S. C., T. K. Rockwell, and C. E. Lindvall, *The Seismic Hazard of San Diego Revised: New Evidence for Magnitude 6+ Holocene* Earthquakes *on the Rose Canyon Fault Zone*: Proceedings of the Fourth U.S. National Conference on Earthquake Engineering, 1990, 11 p.

10. Risk Engineering, *EZ-FRISK*, (Version 7.65) 2014.

11. Todd, Victoria, R., 2004 *Preliminary Geologic Map of the El Cajon 30' x 60' Quadrangle, Southern California,* USGS, Open File Report 2004-1361, Scale 1:100,000.

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

13. USGS computer program, *U.S. Seismic Design Maps*, http://earthquake.usgs.gov/designmaps/us/application.php.