GEOTECHNICAL INVESTIGATION

NEIGHBORHOOD HOUSE ASSOCIATION (NHA) MODULAR RELOCATION 4110 41ST STREET SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

PRAVA
CONSTRUCTION SERVICES INCORPORATED
ESCONDIDO, CALIFORNIA

FEBRUARY 15, 2019
PROJECT NO. G2354-52-01

Project No. G2354-52-01
February 15, 2019

Prava Construction Services Incorporated
344 North Vinewood Street
Escondido, California 92104

Attention: Mr. George Estrema

Subject: GEOTECHNICAL INVESTIGATION
NEIGHBORHOOD HOUSE ASSOCIATION (NHA) MODULAR RELOCATION
4110 41ST STREET
SAN DIEGO, CALIFORNIA

Dear Mr. Estrema:

In accordance with your request and our Proposal No. LG-18458, dated December 13, 2018, we herein submit the results of our geotechnical investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards to assist in the design of the proposed building and improvements. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The site is considered suitable for the proposed building and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project.

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,

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed Neighborhood House Association (NHA) modular relocation project in the City Heights area of San Diego, California (see Vicinity Map, Figure 1). 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 the planned improvements to the property. In addition, this report provides recommendations for 2016 California Building Code (CBC) seismic design criteria, grading, concrete slab-on-grade, shallow foundations, mat foundation, deep foundation, retaining walls and lateral loads. We also include discussions regarding the local geologic hazards including faulting and seismic shaking.

This report is limited to the area proposed for the construction of the new development and associated improvements as shown on the Geologic Map, Figure 2. We used the preliminary site plan prepared by Masson & Associates Incorporated as the base for the Geologic Map.

The scope of this investigation included reviewing readily available published and unpublished geologic literature (see List of References), performing engineering analyses and preparing this geotechnical investigation report. We also drilled four geotechnical borings to a maximum depth of 13 feet (due to refusal), sampled soil and performed laboratory testing. Appendix A presents the exploratory boring logs. The results of the laboratory tests are presented on the boring logs in Appendix A and in Appendix B. Appendix C present the results of the storm water evaluation for the property.

2. SITE AND PROJECT DESCRIPTION

The subject property is located north of Polk Avenue, west of 41st Street, east of an existing alleyway and south of a residential structure in San Diego, California. The rectangular property is currently a dirt lot previously used for parking. The property is relatively flat at an elevation of about 362 to 365 feet above Mean Sea Level (MSL) at the south and north ends of the site, respectively.

Based on the referenced preliminary plan, we understand a rectangular-shaped, 2,880 square-foot building will be constructed within the south-central portion of the property. In addition, a concrete playground including a 600-square-foot shade structure with turf below will be constructed on the east side of the property. We expect the complex will be supported at-grade (i.e. subterranean levels are not planned). The remainder of the property will consist of driveways, parking stalls, a trash enclosure, hardscape areas and landscaping. We understand a stormwater bioretention basin is proposed along the southern border of the property.

The locations and descriptions of the site and proposed development are based discussions with you and observations during our field investigations. If project details vary significantly from those described herein, Geocon Incorporated should be contacted to evaluate the necessity for review and revision of this report.

3. GEOLOGIC SETTING

The site is located in the coastal plain within the southern portion of the Peninsular Ranges Geomorphic Province of southern California. The Peninsular Ranges is a geologic and geomorphic province that extends from the Imperial Valley to the Pacific Ocean and from the Transverse Ranges to the north and into Baja California to the south. The coastal plain of San Diego County is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary rocks that thicken to the west and range in age from Late Cretaceous through the Pleistocene with intermittent deposition. The sedimentary units are deposited on bedrock Cretaceous to Jurassic age igneous and metavolcanic rocks. Geomorphically, the coastal plain is characterized by a series of twenty-one, stair-stepped marine terraces (younger to the west) that have been dissected by west flowing rivers. The coastal plain is a relatively stable block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault Zone and the active Rose Canyon Fault Zone. The Peninsular Ranges Province is also dissected by the Elsinore Fault Zone that is associated with and sub-parallel to the San Andreas Fault Zone, which is the plate boundary between the Pacific and North American Plates.

The site is located on the central portion of the coastal plain roughly two miles south of Mission Valley in the City of San Diego. Marine sedimentary units make up the geologic sequence encountered on the site and consist of the Upper Pleistocene-age Normal Heights Mudstone which is the upper portion of the Pleistocene-age Very Old Paralic Deposits Unit 8, and then a lower conglomerate member of the Very Old Paralic Deposits. The mudstone unit was deposited within a quite marine near shore lagoonal environment that is located in the East San Diego City area and can reach thicknesses up to 13 feet. The Very Old Paralic Deposits were deposited roughly 930k years ago and has been named the Terra Santa Terrace. The lower members of the Very Old Paralic Deposits generally consist of sandstone units with abundant cobbles and boulders and occasional layers containing silt and clay. The geologic unit is generally reported to be 35 to 40 feet thick at the site. The site is located on a natural marine formed terrace and is roughly 380 feet MSL. The site slopes gently to the south with a topographic relief of roughly 3 feet.

4. SOIL AND GEOLOGIC CONDITIONS

Our field investigation indicates the site is underlain by one surficial soil type (undocumented fill) and one geologic unit (Pleistocene-age Very Old Paralic Deposits, which includes the Normal Heights Mudstone). The boring logs in Appendix A and the Geologic Map, Figure 2, show the occurrence, distribution, and description of each unit encountered during our field investigation. The

Geologic Cross-Section, Figure 3, presents a profile view of the underlying geologic conditions. The surficial soil and geologic units are described herein in order of increasing age.

4.1 Undocumented Fill (Qudf)

We encountered undocumented fill to a depth ranging from approximately ½ to 3½ feet in Borings B-1 through B-5. We expect the fill is associated with previous improvements at the site. The fill consists of a gravel layer at the surface with a thickness of 2 to 6 inches across the site and soil fill exists below the gravel in Boring B-2. The undocumented fill was likely not tested or observed during placement and should be considered highly variable. The soil fill material encountered in Boring B-2 generally consists of stiff, moist, reddish brown, sandy clay with trace gravel. The fill soil likely possesses a "medium" to "high" expansion potential (expansion index of 51 to 130). The existing fill is not considered suitable for support of the proposed building structure and adjacent improvements and remedial grading will be required. The existing fill material can be reused as properly compacted new fill if relatively free from vegetation, debris, and contaminants.

Storm water that is allowed to migrate within the undocumented fill soil cannot be controlled due to lateral migration potential, would destabilize support for the existing improvements and would shrink and swell. The undocumented fill will be removed and replaced with properly compacted fill to support the planned improvements. Therefore, full and partial infiltration should be considered infeasible within the undocumented fill.

4.2 Very Old Paralic Deposits (Qm/Qvop)

Quaternary-age Very Old Paralic Deposits, Unit 8 (formerly called the Lindavista Formation) underlies the existing fill soil and extended to the maximum depth explored of 13 feet. During drilling operations we encountered the "Normal Heights Mudstone" (Qm), as described by Reed (1990), within the Very Old Paralic Deposits unit which varies in depths from 7 to 10 feet across the site. The mudstone unit consists within the Very Old Paralic Deposits consist of firm to very stiff, moist to saturated, fat clay. This mudstone unit typically possesses gypsum crystals which increases the water-soluble sulfate content. In addition, the Normal Height Mudstone typically possesses a "very high" expansion potential (expansion index greater than 130).

We encountered, dense to very dense, cemented, sandstone and cobble conglomerate is present below the mudstone unit within the Very Old Paralic Deposits. We encountered practical drilling refusal in the dense sandstone and cobble conglomerate materials in each of the exploratory borings. We did not perform expansion index tests on samples of the underlying cobble and sandstone conglomerate. However, based on previous laboratory testing with similar material in the area we expect this to unit to possess a "very low" to "low" expansive potential (expansion index of 50 or less). Excavations within this unit will likely encounter difficult digging and/or drilling conditions in the cemented zones and oversize material with abundant cobbles will be generated. In addition, coring and rock breaking equipment may be required to excavate the very dense and cemented sandstone and cobble layers.

The infiltration rates within the Very Old Paralic Deposits are considered to be extremely low due to the fine-grained makeup of the Normal Heights Mudstone unit and the cemented/very dense nature of the underlying sandstone and conglomerate materials. Therefore, full and partial storm water infiltration is considered infeasible within the Very Old Paralic Deposits.

5. GROUNDWATER

We did not encounter groundwater in our geotechnical borings to the maximum depth explored of 13 feet or an elevation of roughly 350 feet above MSL. We expect groundwater exists deeper than 200 feet below existing grade. We do not expect groundwater to be encountered during construction of the proposed development. It is possible that perched seepage layers may be encountered during excavation and drilling operations due to adjacent irrigation and drainage practices. It is not uncommon for perched groundwater conditions to develop where none previously existed. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 21 defines the site with a *Hazard Category 52,* identified as an area of favorable geologic structure and low geologic hazard risk. Based on a review of the map, a fault does not traverse the planned development area. Unnamed faults are mapped about 9,000 feet east and west of the site.

6.2 Faulting and Seismicity

Based on our site investigation and a review of published geologic maps and reports, the site is not located on known active, potentially active or inactive fault traces as defined by the California Geological Survey (CGS). The CGS considers a fault seismically active when evidence suggests seismic activity within roughly the last 11,000 years.

According to the computer program EZ-FRISK (Version 7.65), 6 known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. The Rose Canyon Fault Zone and the Newport-Inglewood Fault are the closest known active faults, located approximately 4 miles west of the site. Earthquakes that might occur on the Newport-Inglewood or

Rose Canyon Fault Zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.43g, respectively. Table 6.2.1 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 USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008 acceleration-attenuation relationships.

TABLE 6.2.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2007 (g)
Newport-Inglewood	4	7.5	0.34	0.35	0.43
Rose Canyon	4	6.9	0.30	0.34	0.36
Coronado Bank	16	7.4	0.18	0.14	0.16
Palos Verdes Connected	16	7.7	0.20	0.15	0.19
Elsinore	38	7.9	0.12	0.08	0.10
Earthquake Valley	42	6.8	0.06	0.05	0.04

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 (2007) NGA USGS 2008 in the analysis. Table 6.2.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.2.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2007 (g)
2% in a 50 Year Period	0.43	0.45	0.50
5% in a 50 Year Period	0.29	0.30	0.32
10% in a 50 Year Period	0.20	0.21	0.21

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 structures should be evaluated in accordance with the 2016 California Building Code (CBC) guidelines currently adopted by the City of San Diego.

The site could be subjected to moderate to severe ground shaking in the event of a major earthquake on any of the referenced faults or other faults in Southern California. With respect to seismic shaking, the site is considered comparable to the surrounding developed area.

6.3 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 negligible due to the absence of active faults at the subject site.

6.4 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soil is cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four of the 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. The potential for liquefaction and seismically induced settlement occurring within the site soil is considered to be very low due to the age and dense nature of the Very Old Paralic Deposits.

6.5 Hydroconsolidation

Hydroconsolidation is the tendency of unsaturated soil structure to collapse after saturation resulting in the overall settlement of the effected soil and overlying foundations and improvements. Dry to damp (with a degree of saturation less than about 70 percent), loose to dense sand are typically prone to hydroconsolidation. Potentially compressible soil underlying the proposed structures and existing fill is typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydroconsolidation of the soil exists. The potential for hydroconsolidation can be mitigated by remedial grading and the use of stiffer foundation systems. Based on the laboratory test results, it appears the potential for hydroconsolidation within the Very Old Paralic Deposits to be negligible.

6.6 Landslides

Based on observations during our field investigation and review of published geologic maps for the site vicinity, it is our opinion that potential landslides are not present at the subject property or at a location that could impact the proposed development.

6.7 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 6.5 miles from the Pacific Ocean at elevations greater than 350 feet MSL. Therefore, we consider the risk of a tsunami hazard at the site to be very low.

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

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 With the exception of possible moderate to strong seismic shaking, we did not observe significant geologic hazards or are known to exist on the site that would adversely affect the proposed project. Special consideration will be necessary due to the existing highly expansive Normal Heights Mudstone.

7.1.3 Our field investigation indicates the site is underlain by undocumented fill overlying Very Old Paralic Deposits. The Very Old Paralic Deposits consist of the Normal Heights Mudstone unit (fat claystone) underlain by sandstone/cobble conglomerate. The sandstone and cobble conglomerate materials comprising the Very Old Paralic Deposits are considered suitable for the support of settlement-sensitive structures.

7.1.4 We did not encounter groundwater during our field investigation to the maximum depth explored of 13 feet below the ground surface. We do not expect groundwater will be encountered during construction of the proposed development.

7.1.5 The proposed building can be supported on a post-tensioned foundation or mat slab system bearing in properly compacted fill with associated settlements. We expect the proposed shade structure will be supported on drilled piers founded in the sandstone and cobble conglomerate unit of the Very Old Paralic Deposits. We expect the dense sandstone and cobble conglomerate are present at elevations ranging from approximately 354 to 357½ feet MSL across the site.

7.1.6 Due to the presence of the clayey materials, the potential for expansion and the expected impermeable rates, we opine full or partial infiltration on the property should be considered infeasible due to the very low infiltration rates on the property.

7.1.7 Surface settlement monuments and canyon subdrains will not be required on this project.

7.1.8 The proposed project will not impact the structural integrity of adjacent properties or the existing public improvements and street right-of-ways located adjacent to the site if the recommendations of this report are incorporated into project design.

7.2 Excavation and Soil Conditions

7.2.1 Excavations within the undocumented fill and the Normal Heights Mudstone should generally be possible with moderate to heavy effort using conventional heavy-duty equipment. The sandstone and cobble conglomerate materials within the Very Old Paralic Deposits will likely require very heavy effort to excavate during drilling operations due to its cemented nature and presence of oversize cobble and possible refusal may be encountered. The Very Old Paralic Deposits also can contain also contain cohesionless sand layers. The contractors should be prepared to handle the potential for seepage and caving during the construction operations.

7.2.2 The existing fill and Normal Heights Mudstone unit within the Very Old Paralic Deposits encountered in our field investigation is considered to be “expansive” (expansion index [EI] of greater than 20) as defined by 2016 California Building Code (CBC) Section 1803.5.3. However, the sandstone and cobble conglomerate materials located within the Very Old Paralic Deposits is anticipated to be “non-expansive” (EI of 20 or less). Table 7.2.1 presents soil classifications based on the expansion index. Based on the results of our laboratory testing, presented in Appendix A, we expect the on-site materials possess a “medium” to “high” expansion potential (expansion index of 51 to 130) in accordance with ASTM D 4829.

TABLE 7.2.1
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	

7.2.3 We performed a laboratory test on a sample of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content test. The test results indicate the on-site materials at the location tested possesses “S1” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. Additionally, gypsum is present within the mudstone portion of the Very Old Paralic Deposits that may possess “S1” to “S3” sulfate exposures. Therefore, special concrete mix designs will be needed during construction of the building foundations and slabs and surface concrete pavement and flatwork that is in contact with the existing soils. Table 7.2.2 presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration. We recommend the concrete that will be in contact with site soil to be designed for an "S2" sulfate exposure class.

TABLE 7.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

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

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

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

7.3 Seismic Design Criteria

7.3.1 We used the SEAOL web application program *OSHPD Seismic Design Maps*. Table 7.3.1 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 C. 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 7.3.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.3.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.018g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.389g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.093	Table 1613.3.3(1)
Site Coefficient, F_V	1.622	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.112g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.631g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.742g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.421g	Section 1613.3.4 (Eqn 16-40)

7.3.2 Table 7.3.2 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 (MCEG).

TABLE 7.3.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.427	Figure 22-7
Site Coefficient, F_{PGA}	1.073	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.580g	Section 11.8.3 (Eqn 11.8-1)

7.3.3 Conformance to the criteria in Tables 7.3.1 and 7.3.2 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.

7.3.4 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Rick Category of I, II or III and resulting in a Seismic Design Category D.

7.4 Grading

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

7.4.2 A pre-construction meeting with the city inspector, owner, general contractor, civil engineer, and geotechnical engineer should be held at the site prior to the beginning of grading, excavation and possible utility shoring operations. Special soil handling requirements can be discussed at that time.

7.4.3 Earthwork should be observed and compacted fill tested by representatives of Geocon Incorporated.

7.4.4 Grading of the site should commence with the removal of existing improvements, vegetation, and deleterious debris. Deleterious debris should be exported from the site and should not be mixed with the fill. Existing underground improvements within the proposed structure area should be removed.

7.4.5 The upper soil to a depth of at least 2 feet below the proposed foundations should be removed and replaced with properly compacted fill. The removals should extend at least 5 feet outside the perimeter of the proposed footings, where possible. The upper 2 to 3 feet of undocumented fill and/or Normal Heights mudstone outside the building pad should be removed and replaced with properly compacted fill. The undocumented fill and Normal Heights Mudstone can be reused for compacted fill. We expect the existing materials will need to be exported and import material may be required. Otherwise, the existing materials can be cement treated with at least 5 percent Type II/V cement.

7.4.6 Some areas of overly wet and saturated soil should be expected. The saturated soil would require additional effort prior to placement of compacted fill or additional improvements. Stabilization of the soil would include scarifying and air-drying, removing and replacement with drier soil, use of stabilization fabric (e.g. Tensar TX7, Mirafi HP 370 or other approved fabric), or chemical treating (i.e. cement or lime treatment).

7.4.7 The contractor should be careful during the remedial grading operations to avoid a "pumping" condition at the base of the removals. Where recompaction of the excavated bottom will result in a "pumping" condition, the bottom of the excavation should be tracked with low ground pressure earthmoving equipment prior to placing fill. If needed to improve the stability of the excavation bottoms, reinforcing fabric or 2- to 3-inch crushed rock can be placed prior to placement of compacted fill. A filter fabric should be placed over the rock to help prevent fines migration and settlement.

7.4.8 Fill and backfill materials that will require placement for elevators or adjacent surface improvements should be placed in loose thicknesses of 6 to 8 inches and compacted to a dry density of at least 90 percent of the laboratory maximum dry density 2 to 5 percent greater than the optimum moisture content as determined by ASTM Test Method D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

7.4.9 Import fill (if necessary) should consist of granular materials with a "very low" to "medium" expansion potential (EI of 90 or less) free of deleterious material or stones larger than 3 inches and should be compacted as recommended herein. Geocon Incorporated should be notified of the import source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

7.5 Excavation Slopes

7.5.1 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.

7.5.2 Temporary excavations should be made in conformance with OSHA requirements. Undocumented fill and the Normal Heights Mudstone should be considered a Type C soil in accordance with OSHA requirements. Compacted fill materials can be considered a Type B soil (Type C soil if seepage or groundwater is encountered) and the sandstone/ cobble conglomerate portion of the Very Old Paralic Deposits can be considered a Type A soil (Type B soil if seepage or groundwater is encountered). The contractor should evaluate the proper soil type during excavation.

7.5.3 In general, special shoring requirements will not be necessary if temporary excavations will be less than 4 feet in height and raveling of the excavations does not occur. 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.

7.5.4 The upper mudstone can be very weak in areas and proper shoring or slope inclinations will be required. Therefore, consideration should be given to a maximum of 2- to 3-foot verticals within the clayey materials to help prevent caving. In addition, additional shoring may be required to support deeper excavations.

7.6 Conventional Shallow Foundations/Jacks

7.6.1 The proposed structure can be supported on jacks supported on a conventional shallow foundation system bearing on properly compacted fill if the parameters presented herein are incorporated into design. Foundations for the structures should consist of isolated spread footings. Isolated spread footings should have a minimum width of 36 inches and depth of 30 inches. Figure 4 presents a footing dimension detail depicting the depth to lowest adjacent grade. The jacks can be adjusted if expansion or settlement is observed during the life of the structures.

7.6.2 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. The minimum reinforcement recommended herein is based on soil characteristics only (expansion index of 130 or less) and is not intended to replace reinforcement required for structural considerations.

7.6.3 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. Foundation modifications may be required if unexpected soil conditions are encountered.

7.6.4 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 standard concrete placement. Desiccation cracking should not form in the foundation excavations or slab-on-grade subgrade soil prior to placing concrete.

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

7.7 Post-Tensioned Foundations

7.7.1 The proposed building can be supported on a post-tensioned foundation system founded in properly compacted fill. The post-tensioned system should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC10.5 as required by the 2016 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 7.7. The parameters presented in Table 7.7. are based on the guidelines presented in the PTI, DC10.5 design manual.

TABLE 7.7
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) DC10.5 Design Parameters	Value
Thornthwaite Index	-20
Equilibrium Suction	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	3.8
Edge Lift, y_M (inches)	3.40
Center Lift Moisture Variation Distance, e_M (feet)	7.0
Center Lift, y_M (inches)	1.07

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

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

- The criteria presented in Table 7.7 are still applicable.
- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 24 inches. The embedment depths should be measured from the lowest adjacent pad grade.

7.7.4 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.

7.7.5 We estimate the total and differential settlements under the imposed allowable loads to be about ½ inch based on the minimum dimensions discussed herein. We estimated the total and differential settlement under the imposed allowable loads based on a 10-foot square footing to be about 1 and ½ inch, respectively. We expect the differential static settlement is one-half of the total settlement in a distance of 40 feet.

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

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

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

7.8 Drilled Pier Recommendations

7.8.1 We understand the shade structure may be supported on drilled piers. Drilled piers can be designed to develop support by end bearing and skin friction within the sandstone portion of the Old Paralic Deposits. The drilled piers should be embedded at least 2 feet into the sandstone portion of the Very Old Paralic Deposits; therefore, we expect the drilled piers will be at least 10 to 15 feet long. An allowable end bearing pressure of 18,000 psf can be used for the design of the drilled piers. An allowable skin friction resistance of 300 can be used for that portion of the drilled pier embedded in sandstone portion of the Very Old Paralic Deposits. These allowable values possess a factor of safety of at least 2 and 3 for skin friction and end bearing, respectively. We estimate the settlement of the drilled piers will be approximately ½ inch.

7.8.2 The diameter of the piers should be a minimum of 18 inches. The design length of the drilled piers should be determined by the designer based on the elevation of the pile cap or grade beam and the elevation of the top of the formational materials obtained from the Geologic Map and Geologic Cross-Sections presented herein. It is difficult to evaluate the exact length of the proposed drilled piers due to the variable thickness of the existing fill and Normal Heights Mudstone; therefore, some variation should be expected during drilling operations.

7.8.3 Piers should be spaced at least three-pile diameters, center-to-center. If they are spaced closer than this, the efficiency of the group will be less than 100 percent. Standard reductions for lateral capacity should be applied to piles groups spaced closer than 7 diameters on center. We can provide an analysis of group lateral capacity using the computer program GROUP once foundation plans are available, if necessary.

7.8.4 Because a significant portion of the pier capacity will be developed by end bearing, the bottom of the borehole should be cleaned of loose cuttings prior to the placement of steel and concrete. Experience indicates that backspinning the auger does not remove loose material and a flat cleanout plate or hand cleaning is necessary. Concrete should be placed within the pier excavation as soon as possible after the auger/cleanout plate is withdrawn to reduce the potential for discontinuities or caving. Pier sidewall instability may randomly occur if cohesionless soils are encountered. We do not expect seepage will be encountered during the drilling operations. However, casing may be required to maintain the integrity of the pier excavation, particularly if seepage or sidewall instability is encountered. The fill and the formational materials contain gravel, cobble and some boulders. The formational materials may possess very dense and cemented zones, and difficult drilling conditions during excavations for the piers should be anticipated.

7.8.5 In general, ground conditions are moderately suited for drilled pier construction techniques. However, gravel, cobble, oversized material and cemented zones may be encountered in the Very Old Paralic Deposits that could be difficult to drill. Additionally, some raveling may result along the unsupported portions of excavations in the existing clay materials. Seepage, if encountered during the drilling operations, may cause caving.

7.9 Mat Foundation Recommendations

7.9.1 The proposed structure may be supported on a mat foundation. A mat foundation consists of a thick, rigid concrete mat that allows the entire footprint of the structure to carry building loads. In addition, the mat can tolerate significantly greater differential movements such as those associated with expansive soils or differential settlement. We expect the mat foundation would be supported on compacted fill.

7.9.2 The allowable bearing capacity can be taken as 500 pounds per square foot (psf). The modulus of subgrade reaction for design of the mat can range from 50 to 75 pounds per cubic inch (pci) for the compacted fill and formational materials. These values should be modified as necessary using standard equations for mat size as required by the structural engineer. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

7.9.3 We expect total and differential settlements to be ½ inch and ½ inches in 40 feet, respectively, under static loads.

7.9.4 A mat foundation system will allow the structure to settle with the ground and should have sufficient rigidity to allow the structure to move as a single unit. Re-leveling of the mat foundation could be performed through the use of mud jacking, compaction grouting or other similar techniques if differential settlement occurs, if necessary.

7.9.5 Foundation and bottom excavations should be observed by the Geotechnical Engineer (a representative of Geocon Incorporated) prior to the placement of reinforcing steel and concrete to observe that the exposed soil conditions are consistent with those expected and have been extended to appropriate bearing strata. If expected soil conditions are encountered, foundation modifications may be required.

7.10 Concrete Flatwork

7.10.1 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 4 x 4 – W4.0/W4.0 (4 x 4 - 4/4) welded wire mesh or No. 4 reinforcing bars at 12 inches on 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.

7.10.2 The Normal Heights Mudstone portion of the Very Old Paralic Deposits possesses a "medium" to "very high" expansion potential (expansion index of greater than 50). Flatwork placed above the mudstone will likely experience movement during the lifetime of the improvements. Consideration should be given to removing the upper 2 feet of material and replacing it with a non-expansive material (i.e. sand or base) or lime treating the upper 12 to 24 inches. We expect 5 percent lime can be used for lime treatment, if desired.

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

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

7.10.5 The recommendations presented herein are intended to reduce the potential for cracking of slabs and foundations as a result of differential movement. However, even with the incorporation of the recommendations presented herein, foundations and slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

7.11 Retaining Walls

7.11.1 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 40 pounds per cubic foot (pcf) for select backfill with a "very low" to "medium" expansion potential (expansion index of 90 or less). Where the backfill will be inclined at 2:1 (horizontal to vertical), an active soil pressure of 55 pcf is recommended. Soil with an expansion index (EI) of greater than 90 should not be used as backfill material behind retaining walls. Geocon should test the soil proposed for wall backfill prior to use to check with conformance with these recommendations. Import soils may be required for wall backfill to achieve the proper soil characteristics.

7.11.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure of 7H psf should be added to the active soil pressure for walls 8 feet or less. For walls greater than 8 feet tall, an additional uniform pressure of 13H psf should be applied to the wall starting at 8 feet from the top of the wall to the base of the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

7.11.3 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure. A seismic load of 17H psf should be used for design of walls that support more than 6 feet of backfill 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. We used the site-specific peak ground acceleration, PGA_M, of 0.458g calculated from ASCE 7-10 Section 11.8.3. Figure 5 presents a retaining wall loading diagram.

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

7.11.5 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

7.11.6 The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 90 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. Figure 6 presents typical retaining wall drain details for conventional walls. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.11.7 In general, wall foundations having a minimum depth and width of 1 foot may be designed for an allowable soil bearing pressure of 1,500 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.

7.11.8 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. We should be contacted to provide additional recommendations if other types of walls (such as mechanically stabilized earth [MSE] walls, soil nail walls, or soldier pile walls) are planned.

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

7.12 Lateral Loading

7.12.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 300 pounds per cubic foot (pcf) should be used for the design of footings or shear keys poured neat in compacted fill. The 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.

7.12.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.25 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 (typically a reduced friction coefficient of about 0.2 to 0.25).

7.12.3 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.

7.13 Preliminary Pavement Recommendations

7.13.1 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 used an R-Value of 3 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 7.13.1 presents the preliminary flexible pavement sections.

TABLE 7.13.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete Thickness (inches)		
			3	3 ½	4
			Class 2 Aggregate Base (inches)		
Parking stalls for automobiles and light-duty vehicles	5.0	3	10	9	8
Driveways for automobiles and light-duty vehicles	5.5	3	12	11	10
Medium truck traffic areas	6.0	3	---	13	12
Driveways for heavy truck traffic	7.0	3	---	---	16

7.13.2 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 materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

7.13.3 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)*.

7.13.4 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.

7.13.5 A rigid Portland Cement concrete (PCC) pavement section should be placed in driveway entrance aprons, trash bin loading/storage areas and the alleyway. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 7.13.2.

TABLE 7.13.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	50 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 and 100

7.13.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.13.3.

TABLE 7.13.3
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Areas (TC=A)	6.0
Heavy Truck and Fire Lane Areas (TC=C)	7.5*

*Conforms with City of San Diego Schedule J for Traffic Index of 6.5.

7.13.7 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).

7.13.8 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 7.5-inch-thick slab would have a 9.5-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

7.13.9 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 15 feet for the 6-inch-thick slabs and thicker and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report. The depth of the crack-control joints should be at least ¼ of the slab thickness when using a conventional saw, or at least 1 inch when using early-entry saws on slabs 9 inches or less in thickness, as determined by the referenced ACI report discussed in the pavement section herein. Cuts at least ¼ inch wide are required for sealed joints, and a ⅜ inch wide cut is commonly recommended. A narrow joint width of $^1/_{10}$ to $^1/_8$ inch wide is common for unsealed joints.

7.13.10 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.

7.13.11 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.

7.14 Site Drainage and Moisture Protection

7.14.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure. Appendix C presents the storm water management recommendations.

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

7.14.3 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.

7.15 Grading and Foundation Plan Review

7.15.1 Geocon Incorporated should review the final improvement/grading plans and foundation plans prior to finalization to check their compliance with the recommendations of this report and evaluate the need for additional comments, recommendations, and/or analyses.

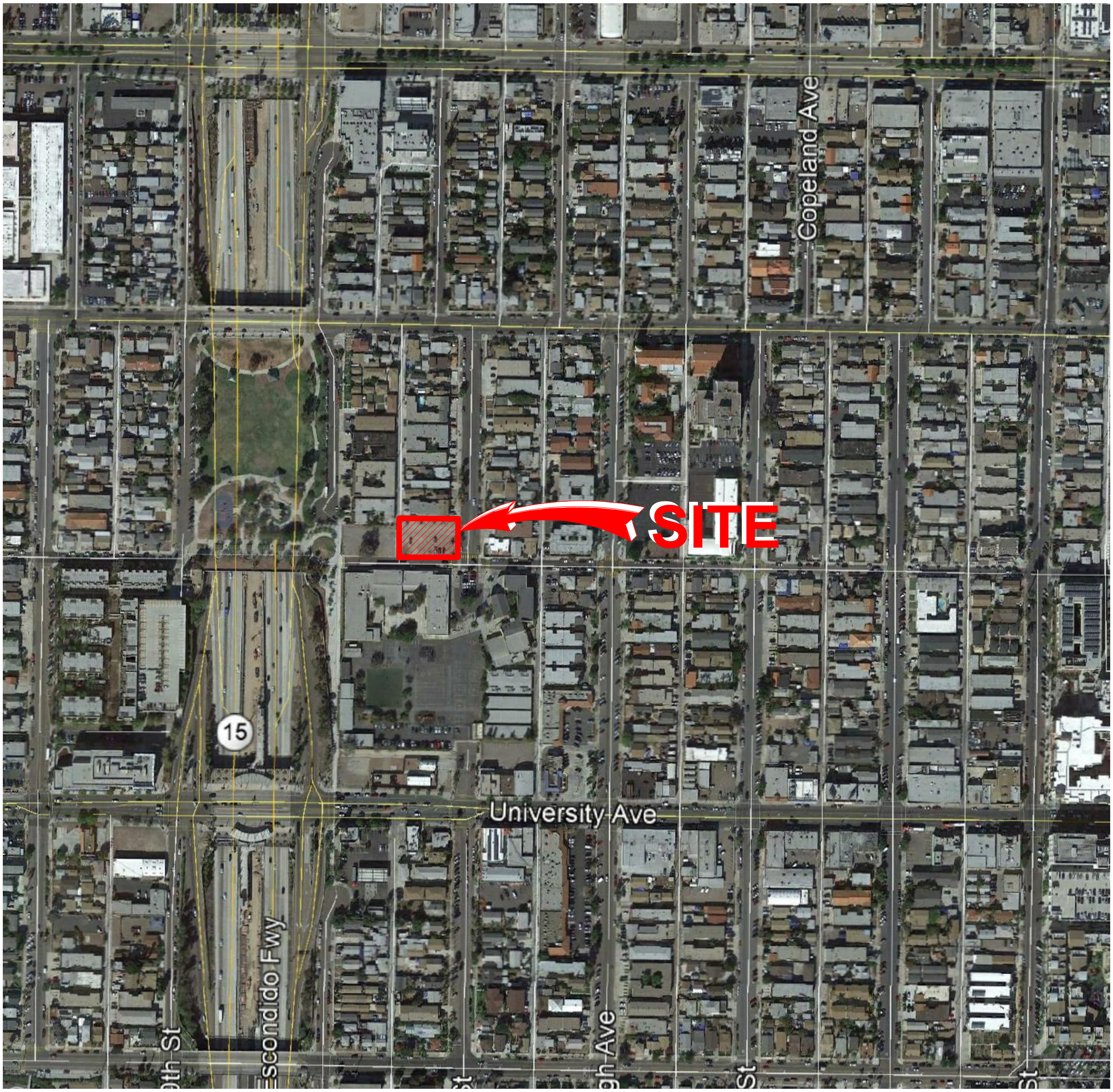
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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



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NO SCALE

VICINITY MAP

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NEIGHBORHOOD HOUSE ASSOCIATION (NHA)
MODULAR RELOCATION
4110 41ST STREET
SAN DIEGO, CALIFORNIA

LR / CW		DSK/GTYPD	DATE 02 - 15 - 2019	PROJECT NO. G2354 - 52 - 01	FIG. 1

Plotted:02/15/2019 7:35AM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\G2354-52-01 NHA Modular\DETAILS\G2354-52-01_Vicinity Map.dwg

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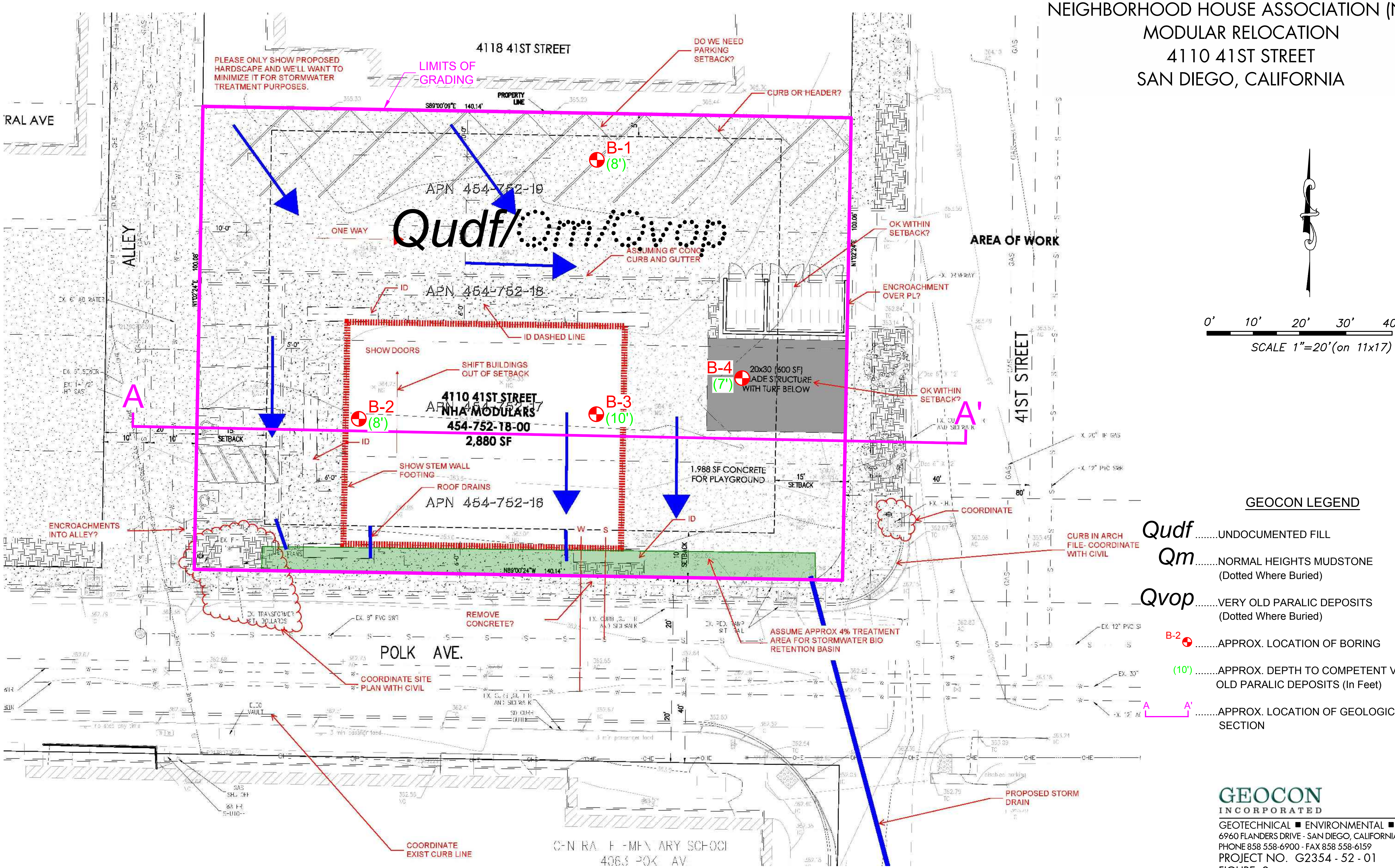

GEOCON LEGEND

- **Qudf** ........UNDOCUMENTED FILL
- **Qm** ....NORMAL HEIGHTS MUDSTONE (Dotted Where Buried)
- **Qvop** ....VERY OLD PARALIC DEPOSITS (Dotted Where Buried)
- B-2 ....APPROX. LOCATION OF BORING
- (10') ....APPROX. DEPTH TO COMPETENT VERY OLD PARALIC DEPOSITS (In Feet)
- A A' ....APPROX. LOCATION OF GEOLOGIC CROSS SECTION

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GEOLOGIC MAP

PROJECT NO. G2354 - 52 - 01
FIGURE 2
DATE 02 - 15 - 2019

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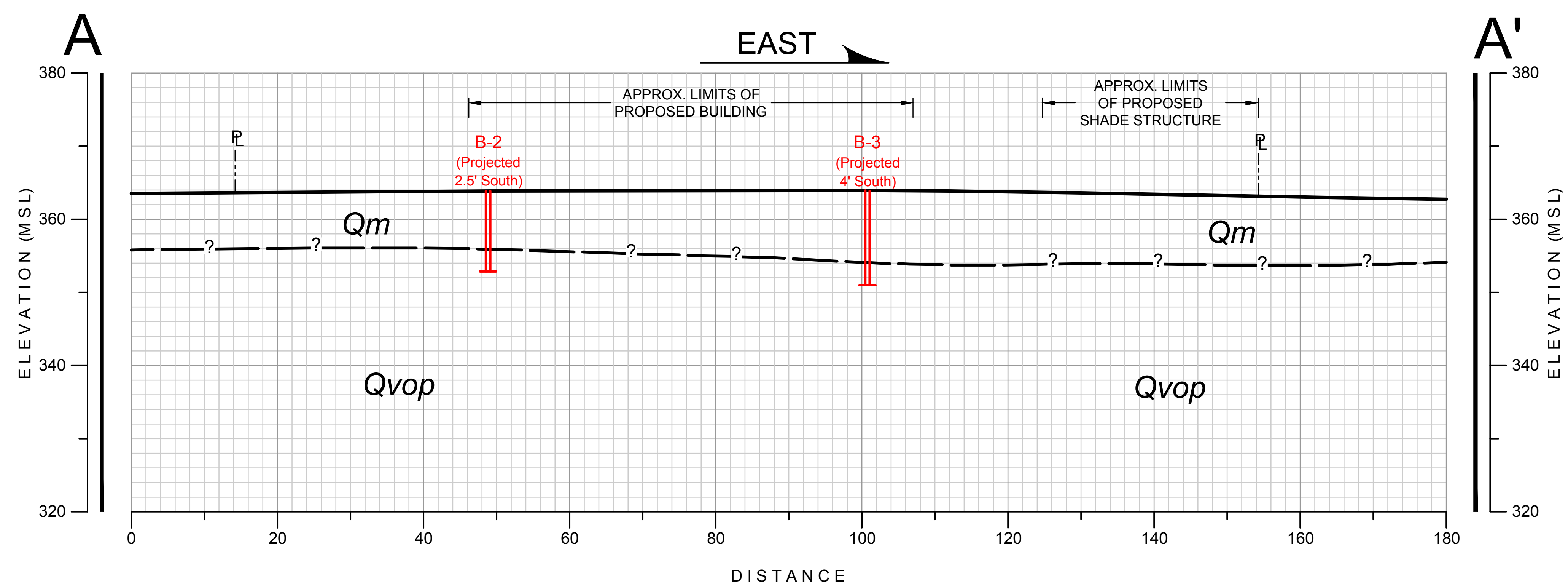

GEOLOGIC CROSS-SECTION A-A'

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

GEOCON LEGEND

Qm........NORMAL HEIGHTS MUDSTONE

Qvop........VERY OLD PARALIC DEPOSITS

B-2 ....APPROX. LOCATION OF BORING

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

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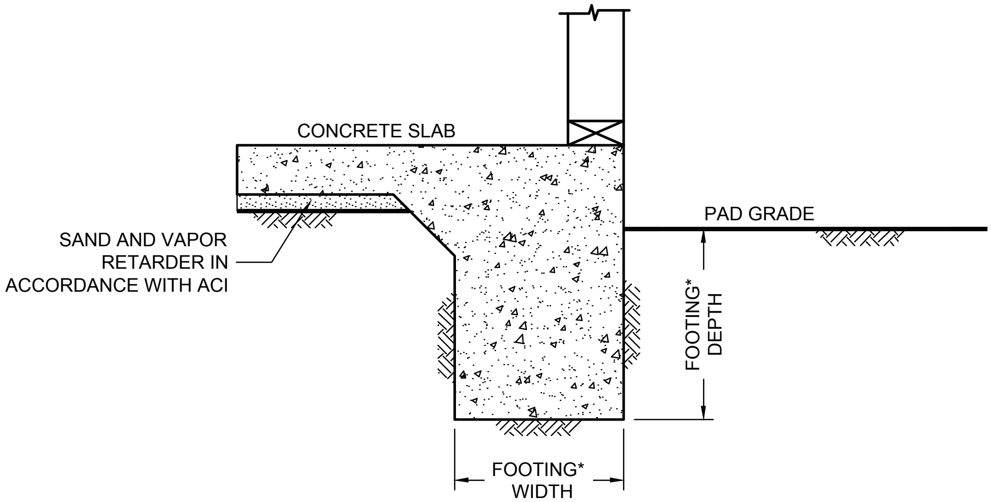
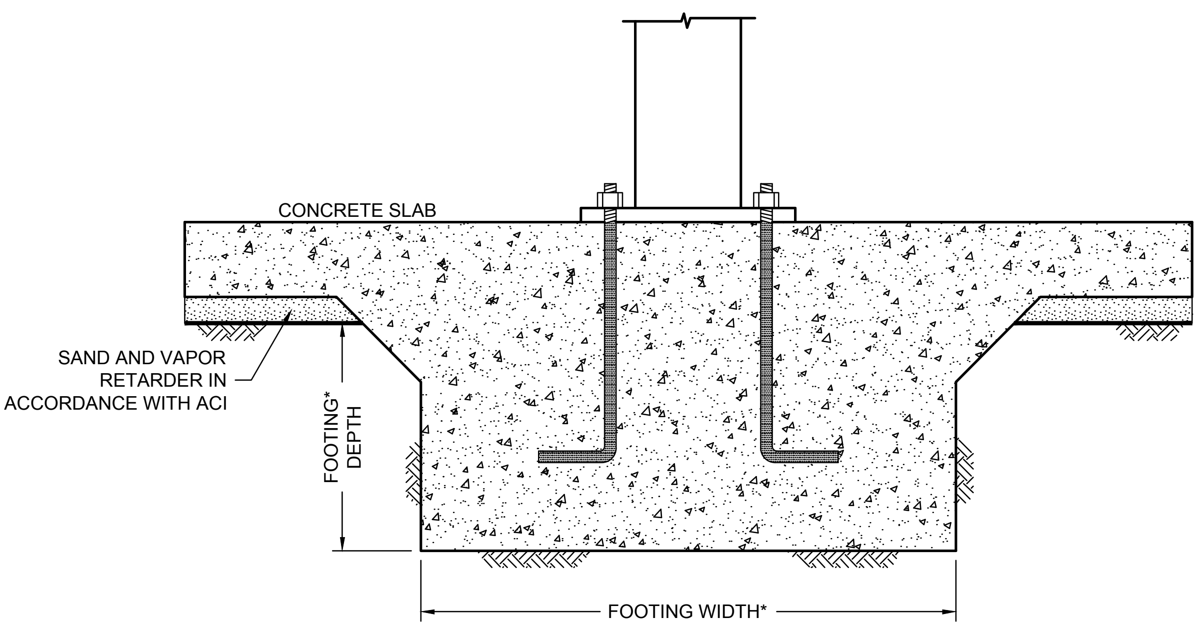
PROJECT NO. G2354 - 52 - 01
FIGURE 3
DATE 02 - 15 - 2019





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

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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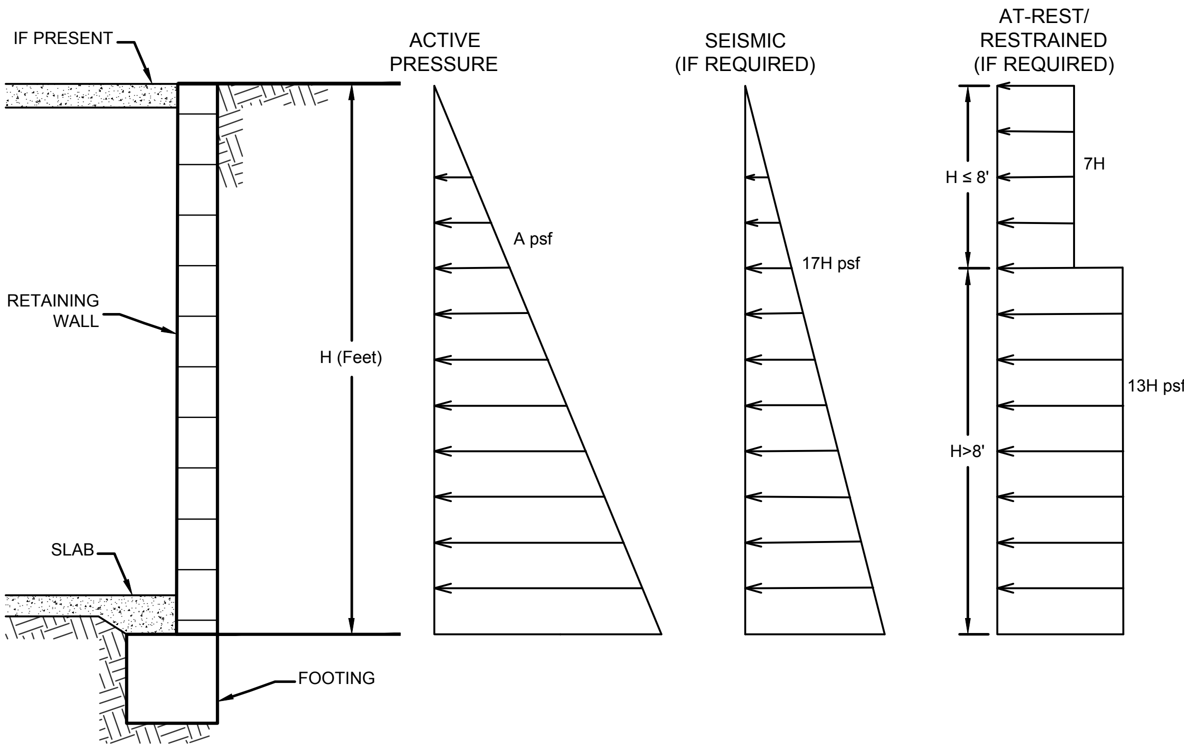
LR / CW		DSK/GTYPD	DATE 02 - 15 - 2019	PROJECT NO. G2354 - 52 - 01	FIG. 4



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 $\frac{2}{3}$ 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 WALLS LOADING COMBINATIONS.

NO SCALE

RETAINING WALL LOADING DIAGRAM

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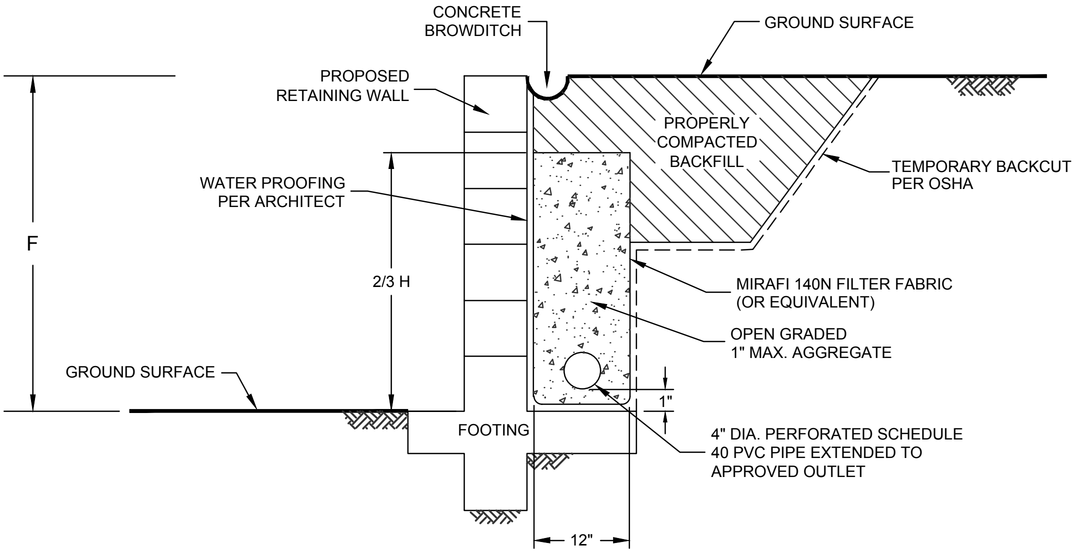
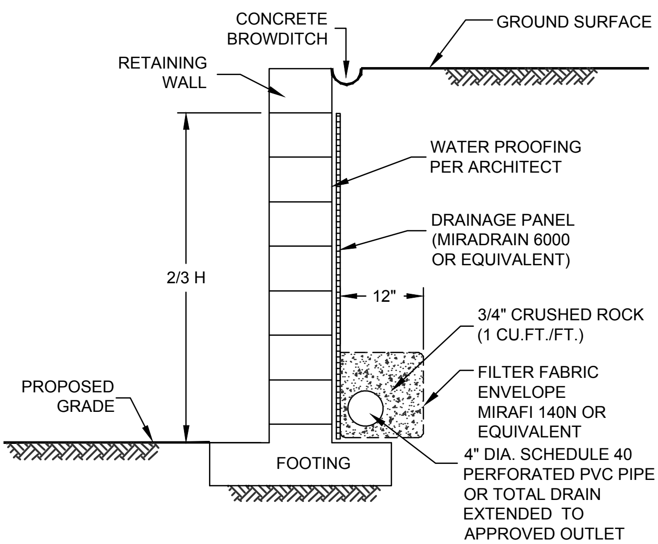
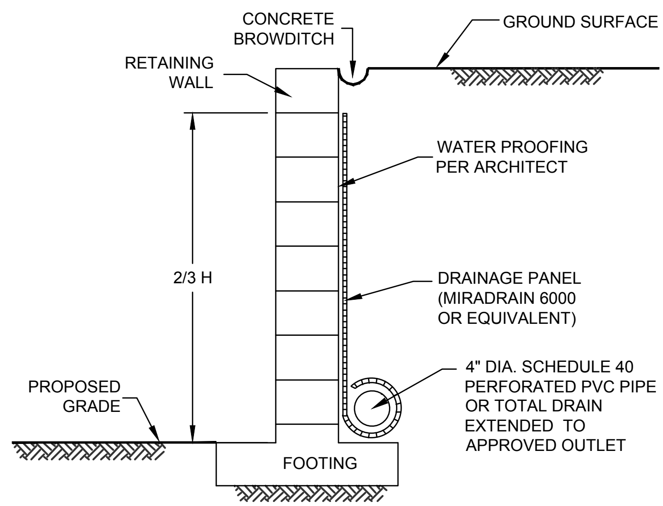
LR / CW		DSK/GTYPD	DATE 02 - 15 - 2019	PROJECT NO. G2354 - 52 - 01	FIG. 5





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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LR / CW		DSK/GTYPD	DATE 02 - 15 - 2019	PROJECT NO. G2354 - 52 - 01	FIG. 6

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on January 4, 2019, that consisted of a visual site reconnaissance and drilling four exploratory borings. The Geologic Map, Figure 2, shows the approximate locations of the borings.

The exploratory borings, performed by Baja Exploration, were advanced to depths of 10 to 13 feet using a CME 75 truck-mounted drill rig equipped with 8-inch diameter augers. We obtained samples during our subsurface exploration using a California split-spoon sampler. The sampler is composed of steel and are driven to obtain the soil samples. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 2.875 inches. Up to 18 rings are placed inside the sampler that is 2.4 inches in diameter and 1 inch in height. We obtained ring samples in moisture-tight containers at appropriate intervals and transported them to the laboratory for testing. We also obtained disturbed bulk soil samples from the borings for laboratory testing. The type of sample is noted on the exploratory boring logs.

The samplers were driven 12 inches into the bottom of the excavations with the use of a down-hole hammer. The sampler is driven into the bottom of the excavation by dropping a 140-pound hammer from height of 30 inches. Blow counts are recorded for every 6 inches the sampler is driven. The penetration resistances shown on the boring logs are shown in terms of blows per foot. The values indicated on the boring logs are the sum of the last 12 inches of the sampler if driven 18 inches. If the sampler was not driven for 18 inches, an approximate value is calculated in terms of blows per foot or the final 6-inch interval is reported. These values are not to be taken as N-values, adjustments have not been applied.

We visually classified and logged the soil encountered in the excavations 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 of the exploratory borings are presented on Figures A-1 through A-4 included herein. The logs depict the soil and geologic conditions observed and the depth at which samples were obtained.

PROJECT NO. G2354-52-01

BORING B 1

ELEV. (MSL.) **365'** DATE COMPLETED **01-04-2019**

EQUIPMENT **CME 75** 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					**6 INCHES GRAVEL**			
	B1-1			CH	**NORMAL HEIGHTS MUDSTONE (Qm)** Firm, wet, brown, Sandy lean CLAY			
2								
	B1-2					12	102.1	22.0
4								
	B1-3					12	100.5	20.8
6								
	B1-4							
8				SC/GM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, moist, reddish to yellowish brown, Clayey, fine- to medium-grained SANDSTONE to Sandy COBBLE CONGLOMERATE	40		11.0
10	B1-5					81/9"		9.1
					PRACTICAL REFUSAL AT 11 FEET No groundwater			

Figure A-1,
Log of Boring B 1, Page 1 of 1

G2354-52-01.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. G2354-52-01

BORING B 2

ELEV. (MSL.) 364' DATE COMPLETED 01-04-2019

EQUIPMENT CME 75 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					**6 INCHES GRAVEL**			
	B2-1			CL	**UNDOCUMENTED FILL (Qudf)** Medium dense, wet, reddish brown, Sandy CLAY; trace gravel			
2								
	B2-2					21	100.5	22.1
4				CH	**NORMAL HEIGHTS MUDSTONE (Qm)** Stiff, wet, brown, Sandy lean CLAY			
	B2-3					17	93.2	28.3
6								
	B2-4							
8				SC/GM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, yellowish to reddish brown, Clayey, fine to medium-grained SANDSTONE to Sandy COBBLE CONGLOMERATE	68/10"	114.7	11.5
10	B2-5					50/5"		10.4
					PRACTICAL REFUSAL AT 11 FEET No groundwater			

Figure A-2,
Log of Boring B 2, Page 1 of 1

G2354-52-01.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.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 3 ELEV. (MSL.) 364' DATE COMPLETED 01-04-2019 EQUIPMENT CME 75 BY: L. RODRIGUEZ	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**6 INCHES GRAVEL**			
				CH	**NORMAL HEIGHTS MUDSTONE (Qm)** Stiff, moist, brown, Sandy CLAY			
2								
	B3-1					16		15.6
4								
	B3-2				-Becomes firm, saturated	11	94.6	30.7
6								
8	B3-3				-Becomes stiff	15	112.3	19.6
10	B3-4			SC/GM	**VERY OLD PARALIC DEPOSITS (Qvop)** Medium dense, moist, reddish brown, Clayey fine-grained SANDSTONE to Sandy COBBLE CONGLOMERATE -Becomes very dense	38	111.9	16.1
12								
	B3-5					50/4"		
					PRACTICAL REFUSAL AT 13 FEET No groundwater			

Figure A-3,
Log of Boring B 3, Page 1 of 1

G2354-52-01.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.

GEOCON

PROJECT NO. G2354-52-01

BORING B 4

ELEV. (MSL.) 364' DATE COMPLETED 01-04-2019

EQUIPMENT CME 75 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					**2 INCHES GRAVEL**			
	B4-1			CH	**NORMAL HEIGHTS MUDSTONE (Qm)** Firm, wet, brown, Sandy lean CLAY; trace gravel			
2								
	B4-2					13	104.5	19.3
4								
6	B4-3				-Becomes very stiff	34	101.0	20.3
				SM/GM	**VERY OLD PARALIC DEPOSITS (Qvop)** Dense, damp, reddish brown, Silty fine- to coarse-grained SANDSTONE to Sandy COBBLE CONGLOMERATE			
8								
	B4-4					81/9"		
10					PRACTICAL REFUSAL AT 10 FEET No groundwater			

Figure A-4,
Log of Boring B 4, Page 1 of 1

G2354-52-01.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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APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current and generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We selected samples to test for in-place density and moisture content, maximum density and optimum water content, shear strength, expansion potential, plasticity index, water-soluble sulfate content, R-Value, unconfined compressive strength, gradation and consolidation characteristics. The results of our laboratory tests are summarized on Tables B-I through B-VII, Figures B-1 through B-3, and on the boring logs in Appendix A.

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

Sample No.	Description (Geologic Unit)	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B4-1	Brown, Sandy CLAY (Qm)	126.3	11.0

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

Sample No.	Depth (feet)	Geologic Unit	Dry Density (pcf)	Moisture Content (%)		Unit Peak [Ultimate[1]] Cohesion (psf)	Angle of Peak [Ultimate[1]] Shear Resistance (degrees)
				Initial	Final		
B2-3	5	Qm	94.6	30.7	31.0	650 [650]	6 [6]
B4-1[2]	0-5	Qm	112.3	11.4	19.4	400 [400]	15 [15]

[1] Ultimate at end of test at 0.2-inch deflection.
[2] Samples remolded to approximately 90 percent of the laboratory maximum dry density near optimum moisture content.

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

Sample No.	Geologic Unit	Moisture Content (%)		Dry Density (pcf)	Expansion Index	ASTM Soil Expansion Classification	2016 CBC Expansion Classification
		Before Test	After Test				
B4-1	Qm	10.6	23.7	106.7	69	Medium	Expansive

TABLE B-IV
SUMMARY OF LABORATORY PLASTICITY INDEX TEST RESULTS
ASTM D 4318

Sample No.	Geologic Unit	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification
B4-1	Qm	50	15	35	CL-CH

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 318-14 Sulfate Class
B4-1	0-5	Qm	0.106	S1

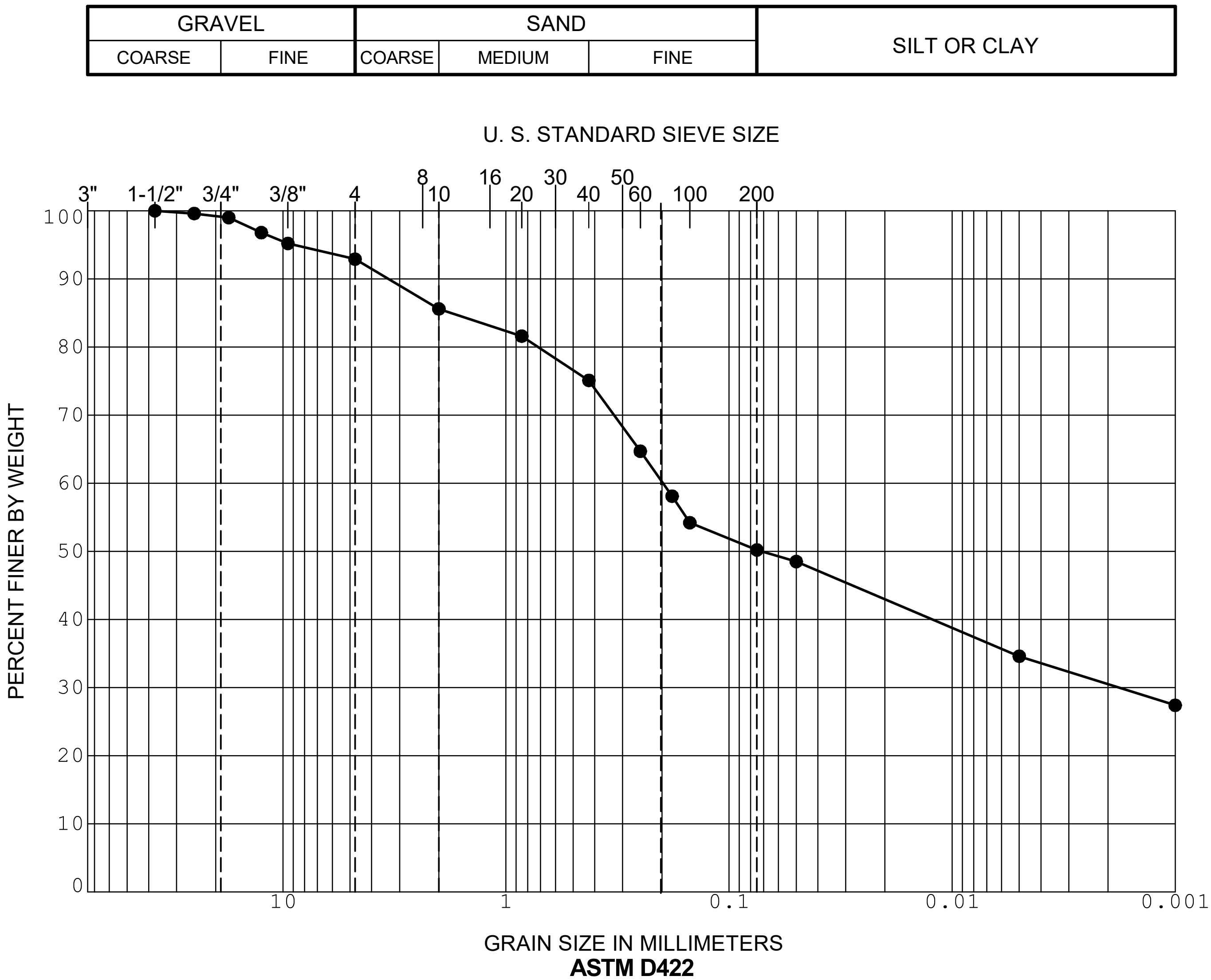
TABLE B-VI
SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS
ASTM D 2844-01

Sample No.	R-Value
B1-1	3

TABLE B-VII
SUMMARY OF LABORATORY UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS
ASTM D 1558

Sample No.	Depth (feet)	Geologic Unit	Hand Penetrometer Reading, Unconfined Compression Strength (tsf)	Undrained Shear Strength (ksf)
B1-2	2.5	Qm	0.5	0.5
B1-3	5	Qm	2.0	2.0
B1-4	7.5	Qm	1.5	1.5
B2-2	2.5	Qudf	1.0	1.0
B2-3	5	Qm	1.5	1.5
B2-4	7.5	Qvop	4.5+	4.5+
B3-2	5	Qm	1.75	1.75
B3-3	7.5	Qm	3.0	3.0
B3-4	10	Qvop	4.5+	4.5+
B4-2	3	Qm	3.0	3.0
B4-3	6	Qm	3.0	3.0
B4-4	9	Qvop	4.5+	4.5+

PROJECT NO. G2354-52-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B4-1	0.0	CL - Sandy, lean CLAY				
⊠							
▲							

GRADATION CURVE

NEIGHBORHOOD HOUSING ASSOCIATION (NHA) MODULAR RELOCATION
4110 41ST STREET
SAN DIEGO, CALIFORNA

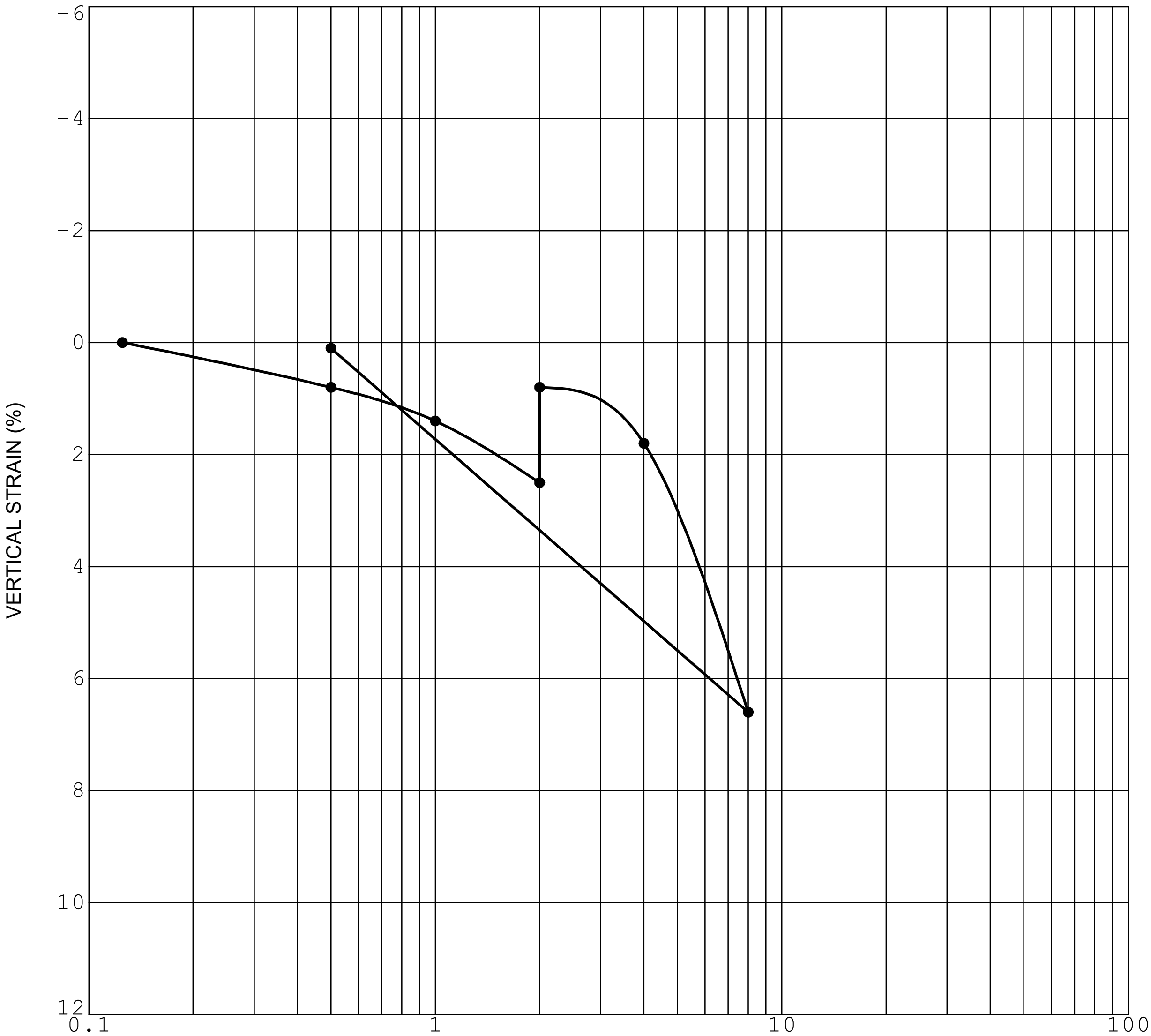
G2354-52-01.GPJ

Figure B-1

GEOCON

PROJECT NO. G2354-52-01

SAMPLE NO. B3-2



ASTM D2435

Initial Dry Density (pcf)	94.6
Initial Water Content (%)	30.7

Initial Saturation (%)	100
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

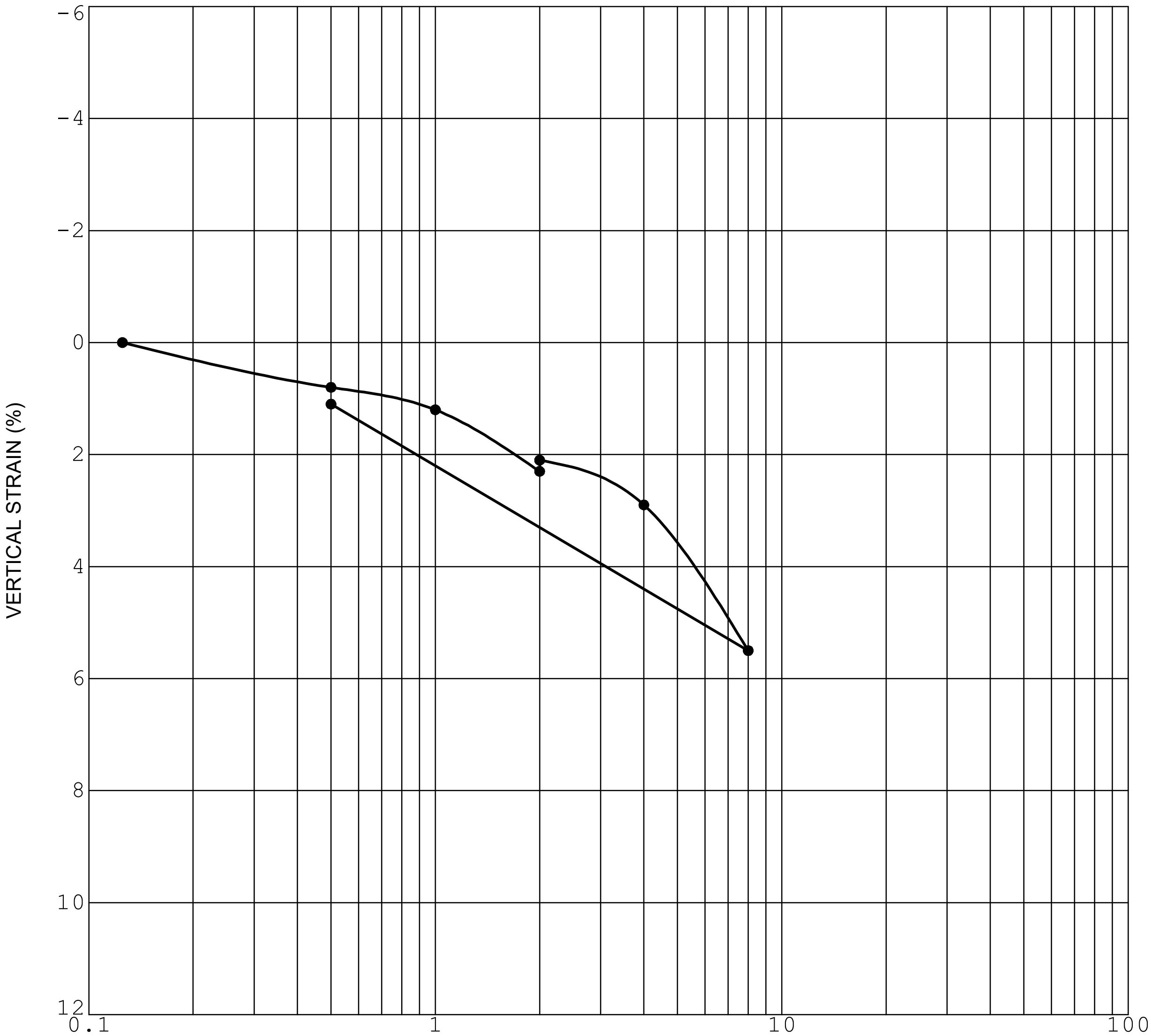
NEIGHBORHOOD HOUSING ASSOCIATION (NHA) MODULAR RELOCATION
4110 41ST STREET
SAN DIEGO, CALIFORNA

G2354-52-01.GPJ

GEOCON

PROJECT NO. G2354-52-01

SAMPLE NO. B3-3



ASTM D2435

Initial Dry Density (pcf)	112.3
Initial Water Content (%)	19.6

Initial Saturation (%)	100
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

NEIGHBORHOOD HOUSING ASSOCIATION (NHA) MODULAR RELOCATION
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Figure B-3

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APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

FOR

NEIGHBORHOOD HOUSE ASSOCIATION (NHA) MODULAR RELOCATION 4110 41ST STREET SAN DIEGO, CALIFORNIA

PROJECT NO. G2354-52-01

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

We prepared this section in accordance with Section C.1.1.1 of the *2017 City of San Diego Storm Water Standards* (SWS). If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table C-1 presents the descriptions of the hydrologic soil groups. 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-1
HYDROLOGIC SOIL GROUP DEFINITIONS

Soil Group	Soil Group Definition
A	Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.
B	Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.
C	Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.
D	Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

Based on the information from the USDA, the property is designated as Urban Land (Ur) and is classified as Soil Group D with a saturated hydraulic conductivity rate of 0.00 to 0.06 inches per hour.

In Situ Testing

The degree of soil compaction or in-situ density and soil type 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 on the property due to the large amount of clay in the existing soil.

Storm Water Design Narrative

The Normal Heights Mudstone underlies the property to a depth of about 7 to 10 feet below grade. As discussed herein, the mudstone is composed of saturated, fat clay (CH) and possesses a "medium" to "very high" expansion potential (expansion index greater than 50). These materials are considered impermeable from a geotechnical engineering standpoint. If the existing soil could take on more water, the soil would lose strength and cause settlement of the existing and proposed improvements. In addition, portions of the roadways, alleyway and sidewalk adjacent to the property have experienced excessive distress due to the expansive nature of the underlying material.

Conclusion

Based on the results of our research and our observations during the drilling operations, the existing geologic units on the property, and the discussion herein, it does not appear that the site conditions possess an opportunity for full and partial infiltration based on the underlying geologic conditions. Therefore, the property should be considered to possess a "No Infiltration" condition in accordance with Appendix C of the 2017 SWS.

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.

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

NEIGHBORHOOD HOUSE ASSOCIATION
(NHA) MODULAR RELOCATION
4110 41ST STREET
SAN DIEGO, CALIFORNIA

PROJECT NO. G2354-52-01

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

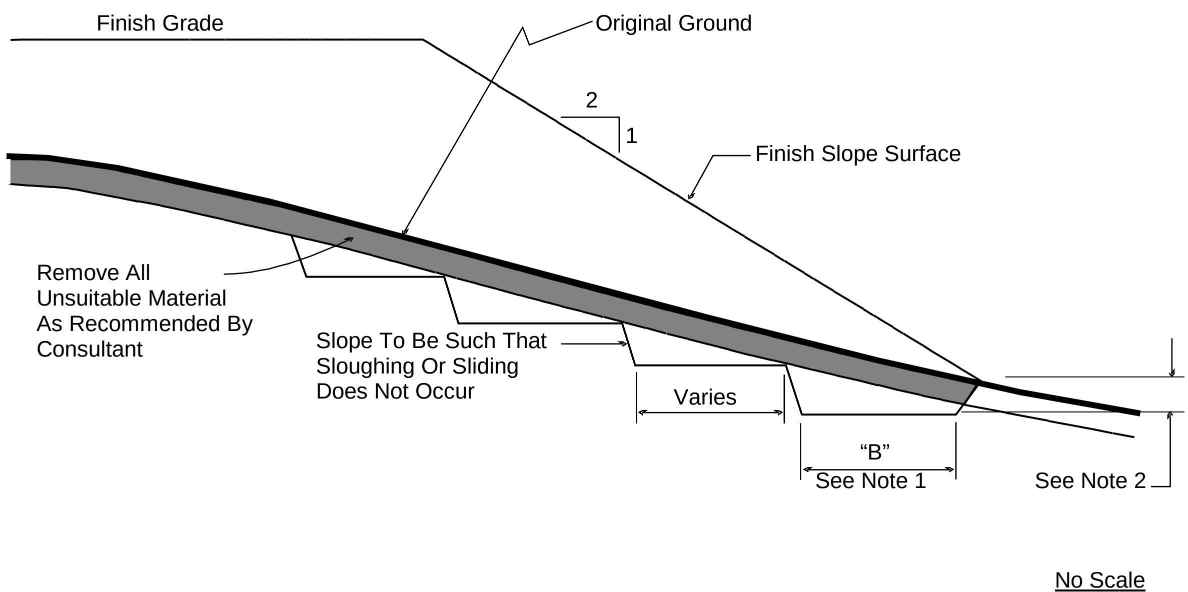
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

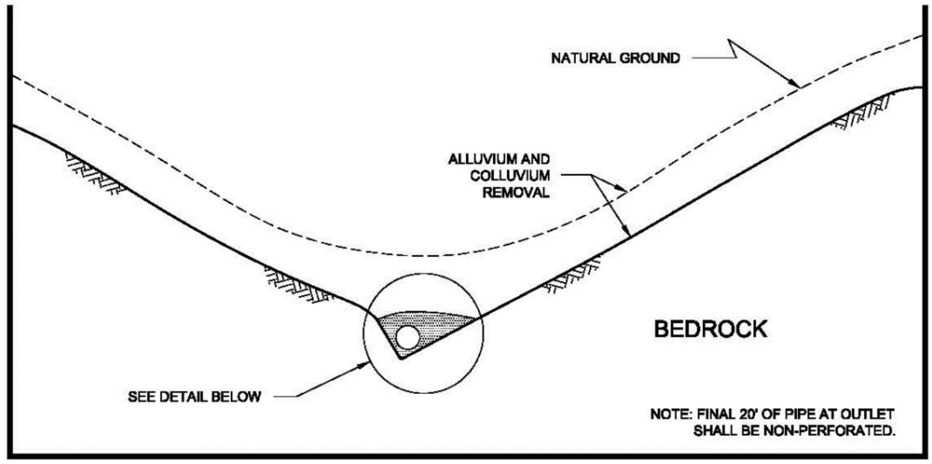
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

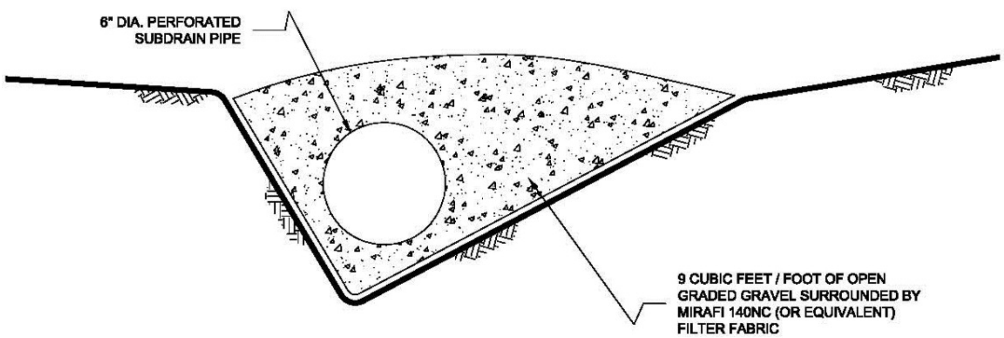
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

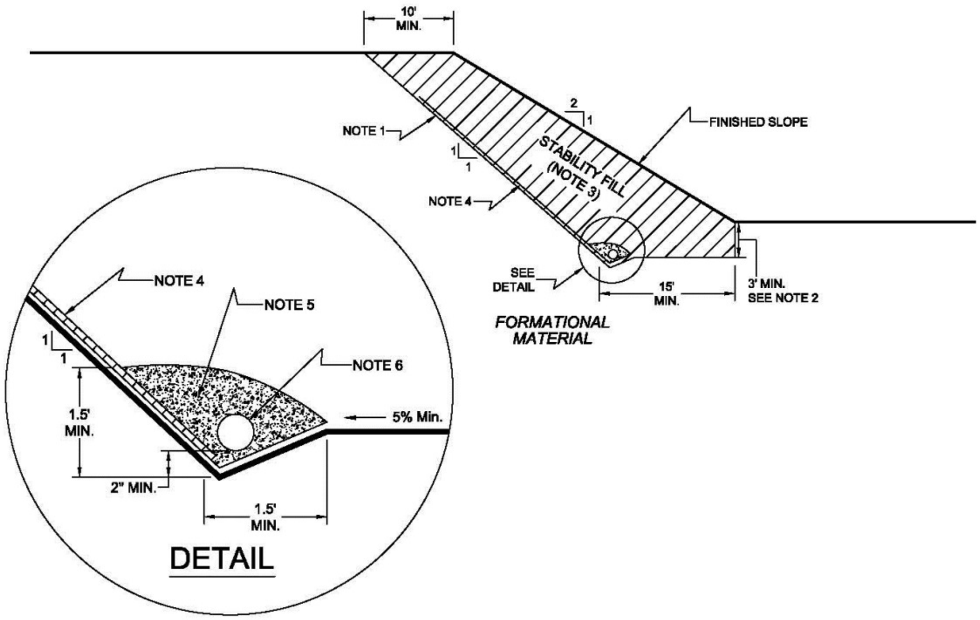
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

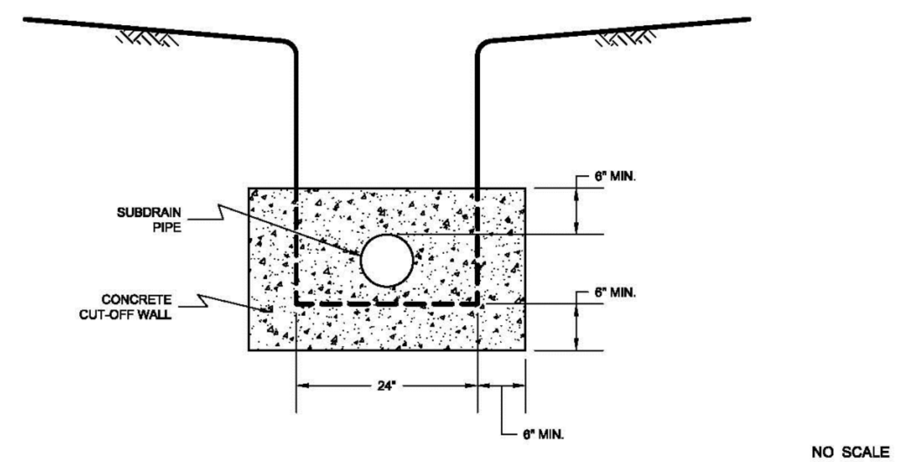
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

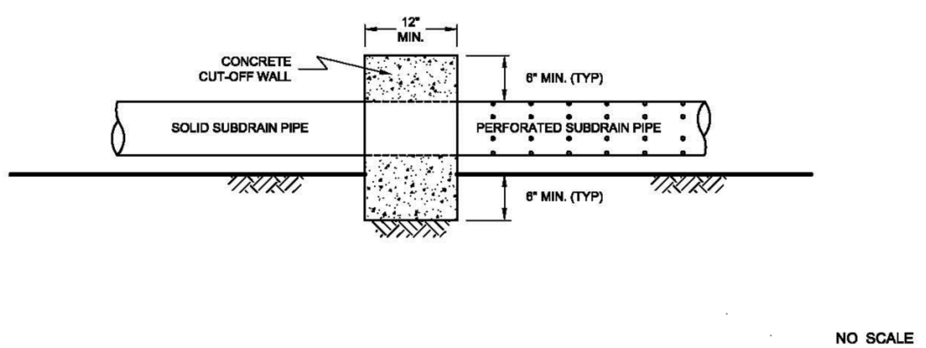
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

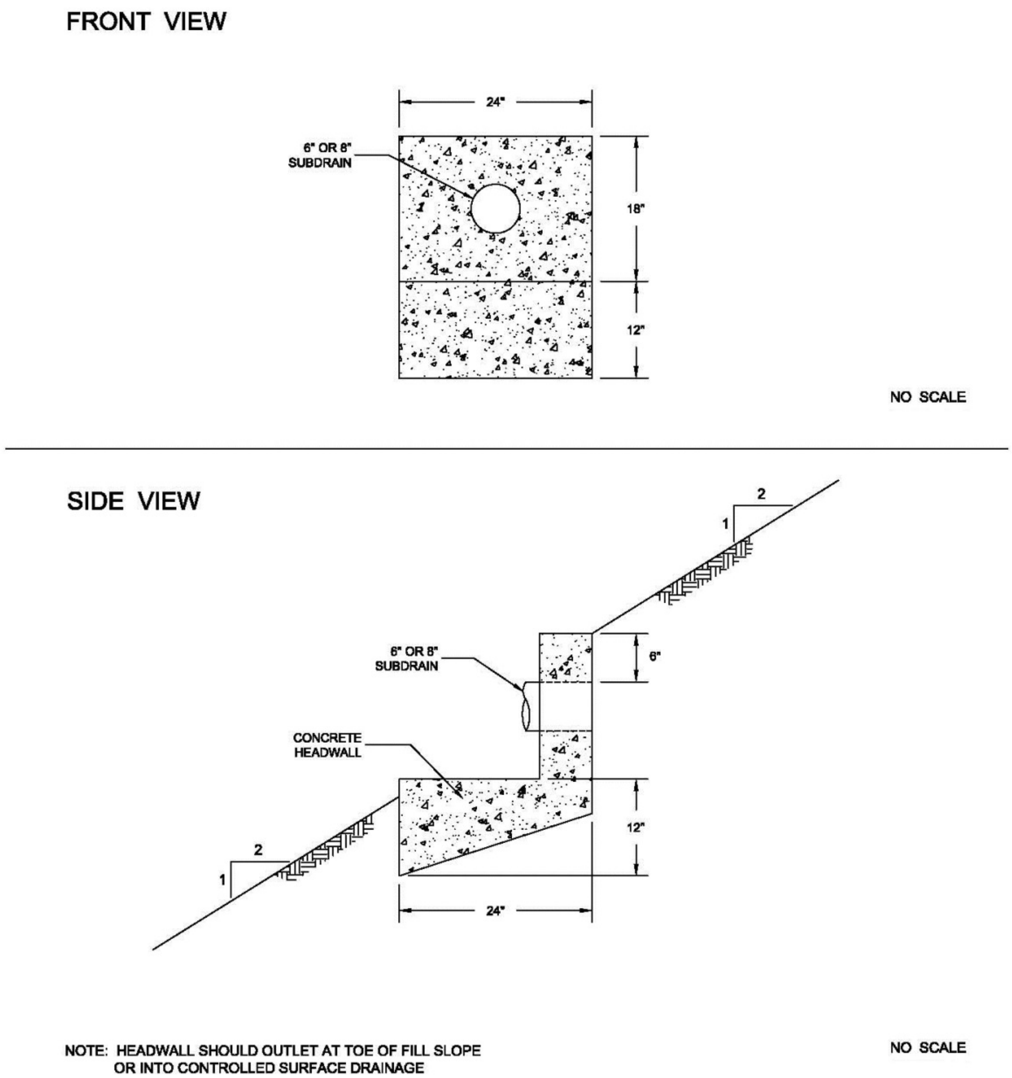


SIDE VIEW



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

TYPICAL HEADWALL DETAIL



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 1, 2016.

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

3. *ASCE 7-10, Minimum Design Loads for Buildings and Other Structures, Second Printing*, April 6, 2011.

4. Boore, D. M., and G. M Atkinson (2008), *Ground-Motion Prediction for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods Between 0.01 and 10.0 S*, Earthquake Spectra, Volume 24, Issue 1, pages 99-138, February 2008.

5. California Geological Survey, *Seismic Shaking Hazards in California,* Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). 10% probability of being exceeded in 50 years. http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html

6. Campbell, K. W., and 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 Robert R. Youngs, *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 for Earthquake Spectra, Spring 2008.

8. City of San Diego Seismic Safety Study, Geologic Hazards and Faults, 2008, Map Sheet 21.

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

10. Kennedy, M. P., and S. S. Tan, 2008, Geologic Map of the San Diego 30'x60' Quadrangle, California, USGS Regional Map Series Map No. 3, Scale 1:100,000.

11. Reed, L. D. (1990), *A New Upper Pleistocene Marine Sedimentary Unit, San Diego, California,* in Geotechnical Engineering Case Histories in San Diego County, San Diego Association of Geologists, p. 1-27.

12. Risk Engineering, *EZ-FRISK*, 2016.

13. SEAOC web application, OSHPD Seismic Design Maps, https://seismicmaps.org/.

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