UPDATE
GEOTECHNICAL INVESTIGATION

PLAZA LA MEDIA–NORTH
OTAY MESA ROAD AND
LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

WESTERN ALLIANCE BANK
REO/COMMERCIAL FACILITIES
% BANK OF NEVADA
LAS VEGAS, NEVADA

SEPTEMBER 11, 2017
PROJECT NO. 07056-32-04

Project No. 07056-32-04
September 11, 2017

Western Alliance Bank REO/Commercial Facilities
℅ Bank of Nevada
2700 West Sahara Avenue, 5th Floor
Las Vegas, Nevada 89102

Attention: Ms. Anne Marie Berg

Subject: UPDATE GEOTECHNICAL INVESTIGATION
PLAZA LA MEDIA–NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

Dear Ms. Berg:

In accordance with your authorization of our proposal No. LG-13395, dated February 26, 2016, we have prepared this update geotechnical investigation for the subject project. The accompanying report discusses soil and geologic conditions at the site and provides recommendations relative to the geotechnical engineering aspects for developing the project as presently proposed.

Provided that the recommendations of the report are followed, the site is considered suitable for construction of the planned development.

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

Very truly yours,

GEOCON INCORPORATED

Raul R. Garcia
GE 2842

REGISTERED PROFESSIONAL ENGINEER
RAUL R. GARCIA
No. 2842
GEOTECHNICAL
STATE OF CALIFORNIA

David B. Evans
CEG 1860

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

RRG:DBE:dmc:ejc

(2) Addressee
(4/del) Atlantis Group
Attention: Mr. Theodore R. L. Shaw
(2/del) Kettler Leweck Engineering
Attention: Mr. Steve Kettler

TABLE OF CONTENTS

1. PURPOSE AND SCOPE 1

2. SITE AND PROJECT DESCRIPTION 2

3. SOIL AND GEOLOGIC CONDITIONS 3
   - 3.1 Undocumented Fill (Qudf) 3
   - 3.2 Topsoil (unmapped) 3
   - 3.3 Old Paralic Deposits (Qvop) 3
   - 3.4 Otay Formation (To) 4

4. GROUNDWATER 4

5. GEOLOGIC STRUCTURE 4

6. GEOLOGIC HAZARDS 4
   - 6.1 Geologic Hazard Category 4
   - 6.2 Faulting and Seismicity 5
   - 6.3 Landslides 6
   - 6.4 Soil Liquefaction 6
   - 6.5 Tsunamis and Seiches 7
   - 6.6 Subsidence and Seismic Settlement 7
   - 6.7 Expansive Soil 7
   - 6.8 Ground Rupture 7

7. CONCLUSIONS AND RECOMMENDATIONS 8
   - 7.1 General 8
   - 7.2 Soil and Excavation Characteristics 9
   - 7.3 Temporary Excavations 11
   - 7.4 Grading 11
   - 7.5 Slope Stability 13
     - 7.5.1 Fill Slopes 13
     - 7.5.2 Cut Slopes 13
   - 7.6 Slope Maintenance 14
   - 7.7 Seismic Design Criteria 14
   - 7.8 Foundation Recommendations 16
   - 7.9 Concrete Slabs-on-Grade 16
   - 7.10 Lateral Loads for Retaining Walls 18
   - 7.11 Preliminary Pavement Recommendations 19
   - 7.12 Bio-Retention Basin and Bio-Swale Recommendations 21
   - 7.13 Drainage and Maintenance 22
   - 7.14 Grading and Foundation Plan Review 22

LIMITATIONS AND UNIFORMITY OF CONDITIONS

TABLE OF CONTENTS (Concluded)

FIGURES AND ILLUSTRATIONS
- Figure 1, Vicinity Map
- Figure 2, Geologic Map (Map Pocket)
- Figure 3, Geologic Cross Section A-A′
- Figure 4, Surficial Slope Stability Analysis-Fill Slopes
- Figure 5, Slope Stability Analysis-Fill Slopes
- Figure 6, Wall/Column Footing Dimension Detail
- Figure 7, Typical Retaining Wall Drain Detail
- Figure 8, Typical Bioswale Basin Detail

APPENDIX A
- FIELD INVESTIGATIONS
- Figures A-1 and A-3, Logs of Large-Diameter Exploratory Borings (Project No. D-4342-J01)
- Figures A-5 – A-9, Logs of Exploratory Trenches (Project No. D-4342-J01)

APPENDIX B
- LABORATORY TESTING
- Table B-I, Summary of Laboratory Maximum Dry Density and Optimum Moisture Content Test Results
- Table B-II, Summary of Laboratory Direct Shear Test Results
- Table B-III, Summary of Laboratory California Bearing Ratio Test Results
- Table B-IV, Summary of Laboratory Expansion Index Test Results
- Table B-V, Summary of Laboratory Atterberg Limits Test Results
- Table B-VI, Summary of Laboratory Potential of Hydrogen (pH), Water-Soluble Sulfates, and Water-Soluble Chlorides Test Results
- Figure B-1, Gradation Curve (File No. D-4342-J01)

APPENDIX C
- RECOMMENDED GRADING SPECIFICATIONS

UPDATE GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report summarizes the findings of our update geotechnical investigation of the proposed Plaza La Media-North project located southeast of Otay Mesa Road and La Media Road in the Otay Mesa area of San Diego, California (See Vicinity Map, Figure 1). The purpose of this study is to update previous geotechnical investigations performed by Geocon Incorporated and to evaluate whether the conclusions and recommendations presented in the referenced reports are relevant to the proposed development, and to provide additional recommendations, if necessary.

The scope of the study included a review of the following geotechnical reports previously prepared for the project and the current project plans:

1. *Soil and Geologic Investigation for Otay Mesa International Plaza Limited, San Diego, California,* dated April 26, 1989, revised October 13, 1989 (Project No. D-4342-J01).

2. *Updated Geotechnical Investigation [for] Judd and Dillard LLC (Otay Mesa International Plaza Limited), San Diego, California,* prepared by Geocon Incorporated, dated March 14, 2003 (Project No. 07056-22-01).

3. *Grading and Drainage Plans for Plaza la Media-North*, prepared by Kettler Leweck Engineering, received via email August 26, 2017.

The scope of this update geotechnical investigation also included a review of readily available geologic literature and in-house reports pertinent to the property. Reports and published literature reviewed for this investigation are summarized in the *List of References* at the end of this report.

The purpose of the referenced geotechnical investigations was to evaluate the surface and subsurface soil and geologic conditions at the site and, based on the conditions encountered, provide recommendations pertinent to the geotechnical engineering aspects of proposed site development. Previous subsurface exploration performed in the north section of the site included 2 large-diameter borings and 7 exploratory trenches used to estimate the thickness of the soil types (undocumented fill, topsoil, Very Old Paralic Deposits and Otay Formation), collect samples for laboratory testing, and to delineate the near-surface geologic units. Details of the previous field investigation and the boring and trench logs are presented in Appendix A.

Laboratory testing was performed on selected representative samples collected during the 1989 subsurface investigation. The purpose of the laboratory testing was to evaluate pertinent physical and chemical soil properties for engineering analysis to assist in providing recommendations for site grading and development. Details of the laboratory testing and a summary of the test results are presented in Appendix B.

The Geologic Map, Figure 2 (map pocket) depicts the configuration of the property, proposed grading, existing topography and geology, and the approximate locations of exploratory excavations. The Geologic Map is based on the referenced grading and drainage plans prepared by Kettler Leweck Engineering.

Conclusions and recommendations presented herein are based on an analysis of the data obtained from our recent geologic reconnaissance; our review of our previous studies; previous laboratory testing; and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The Plaza La Media-North consists of approximately 22 acres of undeveloped land located southeast of Otay Mesa Road and La Media Road in the Otay Mesa area of San Diego, California. The site is a semi-trapezoidal parcel and is delineated along the north property line with approximately 1280 feet of frontage with Otay Mesa road, to the east with 720 feet along proposed Avenida Costa Azul, to the west with 520 feet along La Media Road and to the south with 1400 feet adjacent and parallel to La Media Interstate 905 Offramp. The project limits are presented on the Geologic Map, Figure 2.

The site is relatively level with a northeast to southwesterly drainage gradient. Elevations vary from approximately 485 feet Mean Sea Level (MSL) in the northeast corner to approximately 478 feet MSL at the southwest corner. Vegetation typically consists of dense weeds and grasses.

Based on our review of the grading plans, we understand that proposed project will consist of developing a commercial retail center to receive 10 building pads with at grade parking areas, access driveways, associated improvements and six desilting basins. Widening of Otay Mesa and La Media Road and the construction of Avenida Costa Azul are contemplated as part of project development. We expect that the buildings will be one- to two-story structures with concrete slab-on-grade supported on conventional continuous and isolated spread footings.

Review of grading plans indicates that it is projected to import approximately 170,000 cubic yards of fill soil. In general, the grading will consist of importing fill to raise the grade approximately 6 to 10 feet above existing.

The locations and descriptions of the site and proposed development are based on a site reconnaissance. Review of the referenced plans, and our general understanding of the project as presently proposed. If project details vary significantly from those described, Geocon Incorporated should be retained to update and/or modify this report accordingly.

3. SOIL AND GEOLOGIC CONDITIONS

Three surficial soil deposits and one geologic formation exist at the site. Surficial soils consist of undocumented fill, topsoil, and Quaternary-age Very Old Paralic Deposits (formerly Lindavista Formation). The geologic unit is the Tertiary-age Otay Formation. Descriptions of the surficial soils and formational unit are provided in order of increasing age. The expected subsurface relationship between the surficial soils and geologic units is presented on the Geologic Map, Figure 2, and Geologic Cross-Section A-A', Figure 3.

3.1 Undocumented Fill (Qudf)

Undocumented fill was mapped along Otay Mesa and La Media Road, and was placed after our field investigation (1989). This fill is associated with the widening of Otay Mesa Road and La Media Road. An attempt to obtain an as-graded report for this embankment was unsuccessful. The fill is estimated to be approximately 3 to 5 feet thick, and consists of medium soft, dry to damp, sandy gravelly clay and loose clayey sand. The undocumented fill is unsuitable for support of settlement sensitive structures and/or improvements and will require complete removal and recompaction. The clayey soils are considered expansive; therefore, they should be placed in deeper parts of the fill areas and at least 5 feet below proposed rough grade.

3.2 Topsoil (unmapped)

Topsoil exists throughout the site with thicknesses of approximately 2 to 3 feet. The topsoil, as exposed in exploratory borings and trenches, consists of soft, dry to damp sandy clay. The topsoil is not suitable for support of structural fill or settlement sensitive structures and will require remedial grading in the form of complete removal and compaction. In addition, the topsoil is generally highly expansive and should be placed as compacted fill in deeper parts of the fill areas and at least 5 feet below proposed rough grade.

3.3 Old Paralic Deposits (Qvop)

Very Old Paralic Deposits (formerly Lindavista Formation) underlie the topsoil over the majority of the site. Very Old Paralic Deposits consist of two relatively distinct layers; an upper, highly expansive clay layer over a lower granular layer. The upper clay layer consists of approximately 3 to 10 feet of firm to very stiff clay. The lower granular layer consists of dense silty sand, sandy gravel and clayey sand. Results of our previous laboratory testing indicate that the lower granular soils have a *low* to *medium* expansion potential. Cobble content increases with depth within the sandier portions.

The Very Old Paralic Deposits should provide adequate support for the proposed import structural fill soil. Highly expansive Very Old Paralic Deposits, if exposed near rough grade, should be removed and placed as compacted in the deeper parts of the fill areas and at least 5 feet below rough grade.

3.4 Otay Formation (To)

The Otay Formation underlies the Very Old Paralic Deposits throughout the site. This geologic formation consists of dense to very dense, moist to very moist, fine- to medium-grained silty clayey sandstone to sandy clayey siltstone. The Otay Formation exhibits *low* to *medium* expansion characteristics and should provide adequate support for compacted fill and structural loads. However, the soil of this geologic formation is not expected to be encountered due to its depth below proposed grades.

4. GROUNDWATER

Groundwater or seepage was not encountered in the exploratory excavations conducted on the property during the 1989 field investigation. Perched groundwater conditions should be expected to occur seasonally and may affect site grading if grading operations are performed during or shortly after rainy season. Groundwater is not expected to impact the site; however, if grading operations are performed during the rainy season, saturated conditions and extensive moisture conditioning operations should be expected. Proper surface drainage of irrigation water and precipitation will be critical to future performance of project.

5. GEOLOGIC STRUCTURE

Bedding within the Very Old Paralic Deposits and Otay Formation ranges from massive to well-developed and bedding attitudes are typically horizontal. Geologic structure is not expected to present a constraint to the proposed project.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego *Seismic Safety Study, Geologic Hazards and Faults*, 2008 Edition, Map Sheets 3 and 7 define the site as Hazard Category 53: *Level or Sloping Terrain, unfavorable geologic structure, low to moderate risk*.

6.2 Faulting and Seismicity

Review of the referenced geologic reports and our knowledge of the general area indicate that the site is not underlain by active, potentially active, or inactive faulting. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The site is not located within State of California Earthquake Fault Zone.

A deterministic seismic hazard analysis was performed using the computer program *EZ-FRISK* (Risk Engineering, 2015), six 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. Based on this database, the nearest known active fault is the Newport-Inglewood/Rose Canyon Fault, located approximately 11 miles west of the site and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood/Rose Canyon Fault or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood/Rose Canyon Fault are 7.5 and 0.25g, respectively. Table 6.2.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the 6 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/Rose Canyon	11	7.5	0.25	0.20	0.25
Rose Canyon	11	6.9	0.21	0.18	0.20
Coronado Bank	18	7.4	0.20	0.14	0.17
Palos Verdes Connected	18	7.7	0.22	0.15	0.20
Elsinore	42	7.85	0.14	0.09	0.11
Earthquake Valley	46	6.8	0.08	0.06	0.05

A probalistic seismic hazard analysis was performed using the computer program *EZ-FRISK* (Risk Engineering, 2015). *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the faults slip rate. The program accounts for earthquake magnitude as a function of fault rupture length, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We 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	0.41	0.34	0.40
5% in a 50 Year Period	0.31	0.26	0.28
10% in a 50 Year Period	0.23	0.20	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 California Building Code (CBC) and other guidelines currently adopted by the City of San Diego.

6.3 Landslides

No landslides were encountered at the site or mapped in an area that could impact the property. Landslides are mapped outside and to the southwest of the site. The risk associated with landslide hazard is low for this project.

6.4 Soil Liquefaction

Soil liquefaction occurs within relatively loose, cohesionless sands located below the permanent table that are subjected to ground accelerations from earthquakes. Due to the anticipated depth to permanent groundwater ($\geq$50 feet) and the proposed compacted fill and dense nature of the Very Old Paralic Deposits and Otay Formation at the site, the risk associated with liquefaction hazard at the site is low.

6.5 Tsunamis and Seiches

The site is located approximately 10 miles east of the Pacific Ocean at an elevation of approximately 480 feet above Mean Sea Level (MSL). No large bodies of water are located upstream of the site. The risk associated with inundation hazard due to tsunamis or seiches is low.

6.6 Subsidence and Seismic Settlement

Based on the subsurface conditions encountered during our field investigation, we do not expect the site would be subject to hazards from ground subsidence or seismic settlement.

6.7 Expansive Soil

Based on our experience in the area and the laboratory testing performed, existing undocumented fill, topsoil and the upper clay of the Very Old Paralic Deposits exhibited a high to very high expansion potential (Expansion Index higher than 91). The underlying gravelly sand of the Very Old Paralic Deposits the Otay Formation exhibit low to medium expansion potential (Expansion Index between 21 and 90).

6.8 Ground Rupture

There is low risk for ground rupture within the site due to apparent lack of faulting within or adjacent to the site. As such, we do not expect that planned structures will experience fault ruptures.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 Based on our geologic reconnaissance, the site is in a similar condition to that encountered during our previous geotechnical investigations. It is the opinion of Geocon Incorporated that the conclusions and recommendations presented in this update report and in the previous geotechnical investigations are valid for the proposed site development.

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

7.1.3 Localized areas of undocumented fill with thickness on the order of 3 to 5 feet are located along Otay Mesa Road and La Media Road. Topsoil underlies the majority of the site to depths up to 3 feet. Highly expansive clays comprise the upper portions of Very Old Paralic Deposits, extending to depths ranging from approximately 3 to 10 feet. Granular, *low*- to *medium*-expansive Very Old Paralic Deposits underlie this clay layer. Otay Formation underlies the Very Plod Paralic Deposits.

7.1.4 The undocumented fill, topsoil, and isolated, soft clays of the Very Old Paralic Deposits (if encountered) are unsuitable in their present condition for support of structural fill or settlement sensitive structures and/or surface improvements. As such, removal and recompaction of these materials will be required. The majority of the Very Old Paralic Deposits and Otay Formation are suitable for the support of compacted fill and structural loads, however considering proposed grades, these soils will not influence significantly on proposed foundation systems.

7.1.5 Subsurface conditions observed may be extrapolated to reflect general soil and geologic conditions; however, variations in subsurface conditions between boring and trench locations should be expected.

7.1.6 Highly expansive soils will be encountered within the undocumented fill, topsoil and upper portion of the Very Old Paralic Deposits. Highly expansive soils should be placed in the deeper portions of the fill areas and at least 5 feet below proposed rough grade elevation. Granular low expansive soils should be placed in the upper 5 feet from proposed rough grade on the building pads and in the upper 3 feet from subgrade on paved areas.

7.1.7 Review of the grading plan indicates, that it is proposed to import approximately 170,000 cubic yards of fill to raise the grades from 6 to 10 feet across the site.

7.1.8 Following removal and recompaction as described herein, the site can receive the import fill soil until proposed grades are achieved.

7.1.9 The import fill should consist of granular soil with *low* to *medium* expansion potential. (expansion of less than 90).

7.1.10 No significant geologic hazards that would adversely affect the proposed project, other than seismic shaking and expansive soils, were observed or are known to exist on the site.

7.1.11 In general, undisturbed soils are expected to exhibit low erosion potential. However, fill areas or areas stripped of native vegetation will require special consideration to reduce the erosion potential. In this regard, desilting basins, improved surface drainage and early planting of erosion-resistant ground covers are recommended.

7.1.12 Surface settlement monuments or canyon subdrains will not be necessary for the project.

7.2 Soil and Excavation Characteristics

7.2.1 Excavations of the *in situ* soils should be suitable with moderate effort using heavy-duty grading equipment. Layers of cohesionless sand (if encountered within the Very Old Paralic Deposits) will require special attention with respect to the stability of excavations during trenching for utility lines. Planned excavations into the Very Old Paralic Deposits may be difficult due to localized cemented zones, cobbles, and boulders. The presence of cobbles and boulders could require special excavation methods. Cuts in excess of approximately 10 to 15 feet could generate oversize rocks.

7.2.2 Excavation and compaction difficulties may be experienced if grading operations are performed when clayey soils are very wet or very dry. Extensive moisture conditioning may be required if either case is encountered.

7.2.3 The soils encountered in the field investigation are considered to be expansive (expansion index [EI] greater than 20 as defined by 2016 California Building Code (CBC) Section 1803.5.3. Based on extensive studies performed in the area, the clayey sands and sandy gravels of the Very Old Paralic Deposits and the sandy soils of the Otay Formation possess *low* to *medium* expansion potential (Expansion Index <90). Existing undocumented fill, topsoil, clayey soil of the Very Old Paralic Deposits, and the clayey soil of the Otay Formation possess *high* expansion potential. (Expansion Index >91). Table 7.2.1 presents soil classifications based on the expansion index.

TABLE 7.2.1
SOIL 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	

7.2.4 We performed laboratory tests on three samples of the site materials to evaluate water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the near-surface on-site materials at the locations tested possess *Not Applicable* sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. Table 7.2.2 presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318. ACI guidelines should be followed when determining the type of concrete to be used. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 7.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Severity	Exposure Class	Water-Soluble Sulfate % by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	0.00-0.10	I or II	--	2,500
Moderate	S1	0.10-0.20	II	0.50	4,000
Severe	S2	0.20-2.00	V	0.45	4,500
Very Severe	S3	> 2.00	V + pozzolan or slag	0.45	4,500

7.2.5 We performed laboratory tests on samples to evaluate the corrosion potential to subsurface metal structures as part of our original geotechnical investigation. The laboratory test results are presented in Table B-VI. The laboratory tests were performed in accordance with California Test Method No. 643. Minimum resistivity test results indicated a moderate corrosion potential with respect to buried metal pipes.

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

7.3 Temporary Excavations

7.3.1 Temporary excavations should be constructed in conformance with OSHA requirements. The proposed compacted fill soil should be considered Type B soil in accordance with OSHA requirements. The Very Old Paralic Deposits and the Otay Formation should be considered Type A. In general, special shoring requirements will not be necessary if temporary excavations are less than 4 feet high. Temporary excavation depths greater than 4 feet should be laid back at an appropriate inclination or shored. The soils exposed in these excavations should not become saturated or allowed to dry. Surcharge loads should not be permitted within a distance equal to the depth 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.

7.4 Grading

7.4.1 All grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix C. Where the recommendations of this report conflict with those of Appendix C; this section of the report takes precedence. All grading should be observed by a representative of Geocon Incorporated to verify that the recommendations of this report have been followed.

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

7.4.3 The grading should be tested and observed by a representative of Geocon Incorporated.

7.4.4 Site preparation should begin with the removal of all deleterious material and vegetation. The depth of removal should be such that material exposed in areas to receive import fill or soils to be used as fill are relatively free of organic matter. Any existing underground improvements not projected to remain should be removed and the resulting depression (s) properly backfilled in accordance with the procedures described herein. Material generated during stripping and/or site demolition should be exported from the site.

7.4.5 Compressible surficial deposits (undocumented fill/topsoil or weathered Very Old Paralic Deposits) within areas of planned grading should be completely removed and recompacted prior to placement of additional fill. The actual extent of unsuitable soil removals should be evaluated in the field by the geotechnical engineer or engineering geologist. Overly wet surficial materials will require drying or mixing with drier soils to facilitate proper compaction. Representatives of Geocon Incorporated should evaluate removals of the compressible surficial deposits.

7.4.6 After removal of unsuitable soils and deleterious materials have been removed, areas planned to receive structural fill soils and/or settlement-sensitive improvements should be scarified to a depth of approximately 12 inches, moisture conditioned to 1 to 3 percent above optimum moisture content, and compacted to a minimum relative compaction of 90 percent (ASTM D 1557).

7.4.7 Following removals, the site should be brought to final subgrade elevations with imported structural fill compacted in layers. In general, soils native to the site are suitable for re-use as fill if free from vegetation, debris and other deleterious material. Highly expansive soils should be placed in deeper portions of the fill and at least 5 feet below proposed rough grade elevation. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill lifts of approximately 8 inches thick should be adequate for this project. All fill and backfill should be compacted to at least 90 percent of the maximum dry density at a moisture content ranging from 1 to 3 percent above optimum, as determined in accordance with ASTM D 1557. Fill soils placed at moisture contents outside this range of moisture content may be considered unacceptable at the discretion of the geotechnical engineer. The outer 15 feet of fill slopes should be composed of properly compacted granular soil.

7.4.8 The upper 5 feet of the building pads and 3 feet in pavement areas should be composed of properly compacted *low*-expansive soils. Fill soils with a *high*-expansion potential should be placed in the deeper fill areas and properly compacted. *Low*- to *medium*-expansive soils are defined as those soils that have Expansion Indices from varying 21 to less than 90 as defined in accordance with CBC Section 1805.5.3. Rocks greater than 12 inches in maximum dimension should be placed in accordance with Section 6 of Appendix C.

7.4.9 All import soil, should consist of granular materials with a *low*- to *medium*-expansion potential (EI less than 90). Prior to importing, representative samples of proposed borrow materials should be obtained and subjected to laboratory expansion testing to verify if the soil conforms to the recommended expansion criteria.

7.5 Slope Stability

7.5.1 Fill Slopes

7.5.1.1 Slope stability analyses using laboratory shear strength information and experience with similar soil conditions in nearby areas indicate that 2:1 (horizontal:vertical) fill slopes constructed of on-site granular materials should have calculated factors of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions for heights of 30 feet. Slope stability calculations for deep-seated and surficial stability conditions are presented on Figures 4 and 5. For the slope stability calculations, we used soil parameters obtained as part of the original geotechnical investigation and utilizing our experience with similar soil conditions on nearby projects.

7.5.1.2 Keying and benching operations during grading of the slopes should be performed in accordance with Appendix C.

7.5.1.3 The outer 15 feet of fill slopes should be composed of properly compacted granular fill to reduce the potential for surficial sloughing. In general, soils with an Expansion Index of less than 90 and at least 35 percent sand size particles should be acceptable as granular fill. Slopes should be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked at the completion of each slope such that the fill soils are uniformly compacted to at least 90 percent relative compaction to the face of the finished slope.

7.5.1.4 All slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, all slopes should be drained and properly maintained to reduce erosion. Slope planting should generally consist of drought-tolerant plants having a variable root depth. Slope watering should be kept to a minimum to just support the plant growth. A landscape architect should be contacted to provide recommendations for vegetation planned on slopes constructed with lime treated soils.

7.5.2 Cut Slopes

7.5.2.1 Cut slopes are not proposed as part of project development.

7.6 Slope Maintenance

7.6.1 Slopes steeper than 3:1 (horizontal:vertical) may, under conditions that are both difficult to prevent and predict, be susceptible to near-surface (surficial) slope instability. The instability is typically limited to the outer three feet of the slope and usually does not directly impact the improvements on pad areas above or below the slope. The occurrence of surficial instability is more prevalent on fill slopes and is generally preceded by a period of heavy rainfall, excessive irrigation or the migration of subsurface seepage. Disturbance and/or loosening of the surficial soils, as might result from root growth, soil expansion or excavation for irrigation lines and slope planting, may also be a significant contributing factor to surficial instability. We recommend that, to the maximum extent practical, (a) disturbed/loosened surficial soils be either removed or properly compacted, (b) irrigation systems be periodically inspected and maintained to eliminate leaks and excessive irrigation, and (c) surface drains on and adjacent to slopes be periodically maintained to preclude ponding or erosion. Although the incorporation of the above recommendations should reduce the potential for surficial slope instability, it will not eliminate the possibility, and it may be necessary to rebuild or repair a portion of the project's slopes in the future.

7.7 Seismic Design Criteria

7.7.1 We used the computer program *U.S. Seismic Design Maps* (USGS, 2014), to evaluate the seismic design criteria. Table 7.7.1 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. For preliminary purposes, the building structures and improvements should be designed using a Site Class D. Once final grading plans with specific building locations are available, Geocon Incorporated should be contacted to provide specific seismic design criteria. 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 7.7.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.7.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.818g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.313g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.173	Table 1613.3.3(1)
Site Coefficient, F_V	1.774	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.959g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.555g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.639g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.370g	Section 1613.3.4 (Eqn 16-40)

7.7.2 Table 7.7.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 7.7.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.319g	Figure 22-7
Site Coefficient, F_{PGA}	1.181	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.377g	Section 11.8.3 (Eqn 11.8-1)

7.7.3 Conformance to the criteria in Tables 7.7.1 and 7.7.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive.

7.8 Foundation Recommendations

7.8.1 Foundation recommendations presented herein are based on *low*- expansive within 5 feet of rough pad grade placed and compacted in accordance with the recommendations presented in this report.

7.8.2 Conventional continuous and/or isolated spread footings are suitable for support of the proposed building. Continuous footings should be at least 12 inches wide and 24 inches deep (below lowest adjacent grade). Isolated spread footings should be at least 2 feet wide and extend 24 inches below lowest adjacent grade. A typical wall/column footing dimension detail is presented in Figure 6.

7.8.3 Continuous footings should be reinforced with four, No. 4 steel, reinforcing bars, two placed near the top of the footing and two near the bottom. The project structural engineer should design reinforcement for spread footings.

7.8.4 Foundations proportioned as recommended may be designed for an allowable soil bearing pressure of 2,500 psf (dead plus live loads). This bearing pressure may be increased by 300 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

7.8.5 The allowable soil bearing recommendations presented above are for dead plus live loads only and may be increased by up to one third when considering transient loads such as those due to wind or seismic forces.

7.9 Concrete Slabs-on-Grade

7.9.1 Interior concrete slabs-on-grade should be at least 5 inches thick. Where heavy concentrated floor loads are anticipated, the slab thickness should be increased to 6 inches and should be underlain by 4 inches of Class 2 aggregate base material compacted to at least 95 percent relative compaction.

7.9.2 Minimum reinforcement of slabs-on-grade should consist of No. 3 reinforcing bars placed at 18 inches on center in both horizontal directions. The concrete slabs-on-grade should also be doweled into the foundation system to prevent vertical movement between the slabs, footings, and walls.

7.9.3 The concrete slab-on-grade recommendations are minimums based on soil support characteristics only. We recommend that the project structural engineer evaluate the structural requirements of the concrete slabs for supporting equipment and storage loads.

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

7.9.5 The project foundation engineer, architect, and/or developer should determine the thickness of bedding sand below the slab. Geocon should be contacted to provide recommendations if the bedding sand is thicker than 6 inches.

7.9.6 All exterior concrete flatwork not subject to vehicular traffic should be a minimum of 4 inches thick and conform to the following recommendations. Slab panels in excess of 8 feet square should be reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh to reduce the potential for cracking. In addition, all 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 soils for exterior slabs should be compacted in accordance with criteria presented in the grading section of this report. The subgrade soils should not be allowed to dry prior to placing concrete.

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

7.10 Lateral Loads for Retaining Walls

7.10.1 Retaining walls that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pcf. Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. Expansive soil should not be used as backfill material behind retaining walls. Soil placed for retaining wall backfill should have an Expansion Index less than 50. Existing soils exhibited a *low* to *high* expansion potential. Therefore, we expect import of *low*-expansive granular soil will be required for retaining wall backfill.

7.10.2 Where walls are restrained from movement at the top, an active soil pressure equivalent to the pressure exerted by a fluid density of 60 pcf should be used for horizontal backfill. 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 (unit weight 125 pcf).

7.10.3 Soil contemplated for use as retaining wall backfill should be identified in the field prior to backfilling. 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, onsite soil to be used as backfill will not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the onsite soil for use as wall backfill if standard wall designs will be used.

7.10.4 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the structures adjacent to the base of the wall. The above recommendations assume a properly compacted granular (EI of less than 50) free-draining backfill material with no hydrostatic forces or imposed surcharge load. A typical retaining wall drainage detail is presented on Figure 7, attached. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.10.5 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2013 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 16H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.377g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

7.10.6 To resist lateral loads, a passive pressure equivalent to the pressure exerted by a fluid density of 300 pcf should be used for design of footings or shear keys poured neat against properly compacted granular fill soils. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

7.10.7 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.4 should be used for design. To resist lateral loads, the passive resistance can be combined with friction.

7.10.8 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. In the event that walls higher than 8 feet are planned, Geocon Incorporated should be consulted for additional recommendations.

7.11 Preliminary Pavement Recommendations

7.11.1 The following recommendations are for preliminary purposes and are provided for private driveways and parking areas. The final pavement section design will depend upon soil conditions exposed at subgrade elevation and the results of additional Resistance Value (R-Value) laboratory tests. The following preliminary pavement section recommendations are based on an assumed R-Value of 10. Sections are presented for both flexible (asphalt concrete) and rigid (Portland cement concrete) pavement.

7.11.2 The pavement sections for public streets will be determined by the City of San Diego Engineering Department. The final pavement sections of public streets will be dependent on the traffic index designated by the City of San Diego Engineering Department and the R-Value laboratory test results of the exposed subgrade soils.

TABLE 7.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS – IMPORTED LOW- TO MEDIUM-EXPANSIVE SUBGRADE SOIL

Location	Assumed Traffic Index (TI)	Assumed R-Value	Asphalt Concrete Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
Parking stalls for automobiles and light-duty vehicles	4.5	10	3	7
Driveways for automobiles and light-duty vehicles	5.5	10	4	9
Driveways and parking areas for heavy-duty trucks and fire lanes	7.0	10	4	14.5

TABLE 7.11.2
PRELIMINARY RIGID PAVEMENT SECTIONS – IMPORTED LOW- TO MEDIUM-EXPANSIVE SUBGRADE SOIL

Location	Average Daily[1] Truck Traffic (ADTT assumed)	Assumed R-Value	Portland Cement Concrete[2] (inches)	Class 2 Aggregate Base Thickness (inches)
Parking stalls[3] for automobiles and light-duty vehicles	25-100	10	5	4
Driveways[3] for automobiles and light-duty vehicles	300-500	10	6*	4
Driveways and parking areas for heavy-duty trucks and fire lanes	100-500	10	7**	6

*Slabs should be reinforced with No. 3 steel reinforcing bars placed at 24 inches on centers.
**Slabs should be reinforced with No. 4 steel reinforcing bars placed at 24 inches on centers.

7.11.3 The subgrade soils should be compacted to a minimum relative compaction of 95 percent at near the optimum moisture content. The depth of subgrade compaction should be approximately 12 inches.

7.11.4 Class 2 aggregate base should conform to Section 26-1.-02B of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* and should be compacted to a minimum of 95 percent of the maximum dry density at near optimum moisture content. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Green Book)*.

7.11.5 Where trash bin enclosures are planned within asphalt paved areas, we recommend that the pavement sections be equivalent to the heavy-duty truck categories presented in the respective tables. The concrete should extend into the roadway sufficiently so that all wheels of the trash truck are on the concrete when loading.

7.11.6 Rigid Portland cement concrete sections were evaluated using methods suggested by the American Concrete Institute *Guide for Design and Construction of Concrete Parking Lots (ACI330R-08).*

7.11.7 Construction joints should be provided at a maximum spacing of 12 feet each way to control shrinkage. Installation of these types of joints should be made immediately after concrete finishing.

7.11.8 Construction jointing, doweling, and reinforcing should be provided in accordance with recommendations of the American Concrete Institute.

7.11.9 The performance of asphalt concrete pavements and Portland cement concrete pavements is highly dependent upon providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement will likely result in pavement distress and subgrade failure. If planter islands are proposed, the perimeter curb should extend at least 12 inches below proposed subgrade elevations. In addition, the surface drainage within the planter should be such that ponding will not occur.

7.11.10 Our experience indicates that even with these provisions, a groundwater condition can develop as a result of increased irrigation, landscaping and surface runoff.

7.12 Bio-Retention Basin and Bio-Swale Recommendations

7.12.1 The site will be underlain by import fill soils and clayey soil and the Very Old Paralic Deposits that are generally composed of clay and very clayey sand with gravel. Based on our experience with the onsite soils and infiltration testing in nearby projects, the onsite soil has very low permeability and generally very low infiltration characteristics. It is our opinion the existing soil is unsuitable for infiltration of storm water runoff. A separate Storm Water Management report was prepared by Geocon Incorporated dated June 23, 2016.

7.12.2 Any bio-retention basins, bioswales, and bio-remediation areas should be designed by the project civil engineer and reviewed by Geocon Incorporated. Typically, bioswales consist of a surface layer of vegetation underlain by clean sand. A subdrain should be provided beneath the sand layer. Water should not be allowed to infiltrate adjacent to the planned improvements. We recommend that retention basins, be properly lined to prevent water infiltration into the underlying soil. Prior to discharging into the storm drain pipe or other approved outlet structure, a seepage cutoff wall should be constructed at the interface between the subdrain and storm drainpipe. The concrete cut-off wall should extend at least 6 inches beyond the perimeter of the gravel-packed subdrain system. Figure 8 presents a typical bioswale detail.

7.12.3 The landscape architect should be consulted to provide the appropriate plant recommendations if a vegetated swale is to be implemented. If drought resistant plants are not used, irrigation may be required.

7.13 Drainage and Maintenance

7.13.1 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 1803.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into storm drains and conduits that carry runoff away from the proposed structure.

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

7.13.3 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. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

7.14 Grading and Foundation Plan Review

7.14.1 Geocon Incorporated should review the grading and foundation plans prior to finalization to verify their compliance with the recommendations of this report and determine the need for additional comments, recommendations, and/or analysis.

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.

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

VICINITY MAP

GEOCON INCORPORATED

GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974

PHONE 858 558-6900 - FAX 858 558-6159

PLAZA LA MEDIA - NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

RM / CW		DSK/GTYPD	DATE 09 - 11 - 2017	PROJECT NO. 07056 - 32 - 04	FIG. 1

Plotted:09/11/2017 8:14AM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\07056-32-04 (Plaza La Media)\DETAILS\07056-32-04 Vic Map.dwg

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL
Qt ....VERY OLD PARALIC DEPOSITS (Dotted Where Bureid)
To ....OTAY FORMATION (Dotted Where Buried)
B-3 ....APPROX. LOCATION OF LARGE DIAMETER BORING (Geocon Inc., Project No. D-4342-J01)
T-7 ....APPROX. LOCATION OF EXPLORATORY TRENCH (Geocon Inc., Project No. D-4342-J01)
........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
A A' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION

GEOLOGIC MAP

PLAZA LA MEDIA - NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

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

SCALE 1" = 40'	DATE 09 - 11 - 2017	
PROJECT NO. 07056 - 32 - 04		FIGURE 2
SHEET 1 OF 1		

Plotted:09/11/2017 12:28PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\07056-32-04 (Plaza La Media)\SHEETS\07056-32-04 SitePlan (UPD 2017-08-23).dwg

GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 40' (Vert. = Horiz.)

GEOCON LEGEND

- **Qudf** ........UNDOCUMENTED FILL
- **Qt** ....VERY OLD PARALIC DEPOSITS
- **To** ....OTAY FORMATION
- B-3 ....APPROX. LOCATION OF LARGE DIAMETER BORING
- T-3 ....APPROX. LOCATION OF EXPLORATORY TRENCH
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

GEOLOGIC CROSS - SECTION

PLAZA LA MEDIA - NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

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

SCALE 1" = 40'	DATE 09 - 11 - 2017	
PROJECT NO. 07056 - 32 - 04		FIGURE 3
SHEET 1 OF 1		

Plotted:09/11/2017 12:08PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\07056-32-04 (Plaza La Media)\SHEETS\07056-32-04 Cross-Section.dwg

ASSUMED CONDITIONS :

SLOPE HEIGHT	H = Infinite
DEPTH OF SATURATION	Z = 3 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)
SLOPE ANGLE	i = 26.6 degrees
UNIT WEIGHT OF WATER	γ_w = 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t = 122.0 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ = 26 degrees
APPARENT COHESION	C = 270 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} = 2.9$$

REFERENCES :

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

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

SURFICIAL SLOPE STABILITY ANALYSIS - FILL SLOPES

GEOCON INCORPORATED

GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974

PHONE 858 558-6900 - FAX 858 558-6159

PLAZA LA MEDIA - NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

RM / CW		DSK/GTYPD	DATE 09 - 11 - 2017	PROJECT NO. 07056 - 32 - 04	FIG. 4

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 30 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 122.0 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 26 degrees
APPARENT COHESION	C	= 270 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi} = \dfrac{\gamma_t H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
$FS = \dfrac{N_{cf} C}{\gamma_t H}$	EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi} = 6.6$	CALCULATED USING EQ. (3-3)
$N_{cf} = 24$	DETERMINED USING FIGURE 10, REFERENCE 2
$FS = 1.8$	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

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

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

SLOPE STABILITY ANALYSIS - FILL SLOPES

GEOCON INCORPORATED

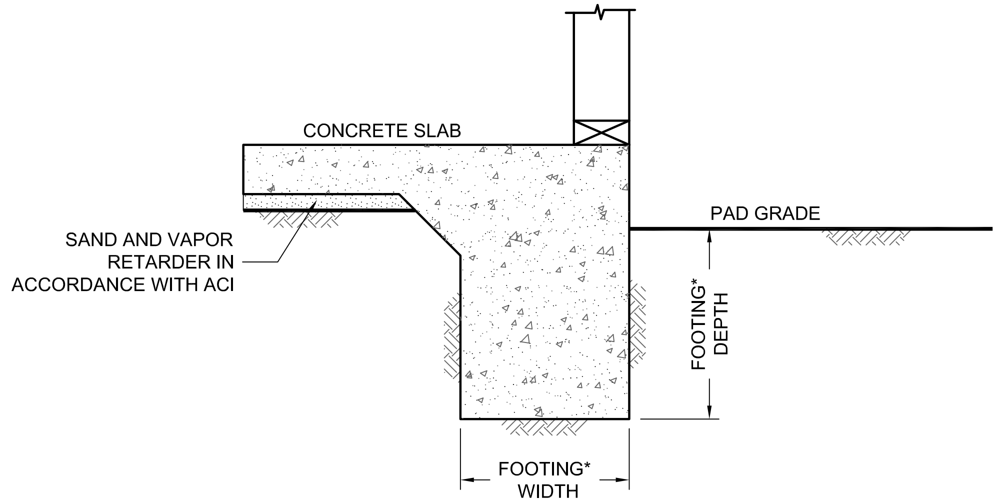
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

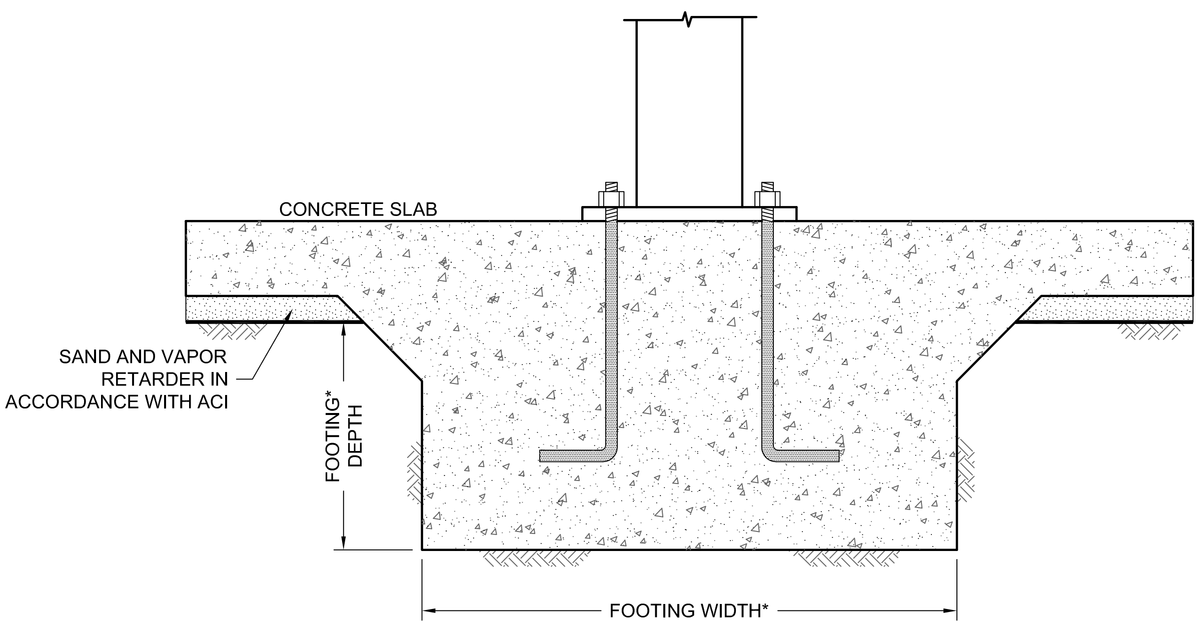
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974

PHONE 858 558-6900 - FAX 858 558-6159

PLAZA LA MEDIA - NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

RM / CW		DSK/GTYPD	DATE 09 - 11 - 2017	PROJECT NO. 07056 - 32 - 04	FIG. 5





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

NO SCALE

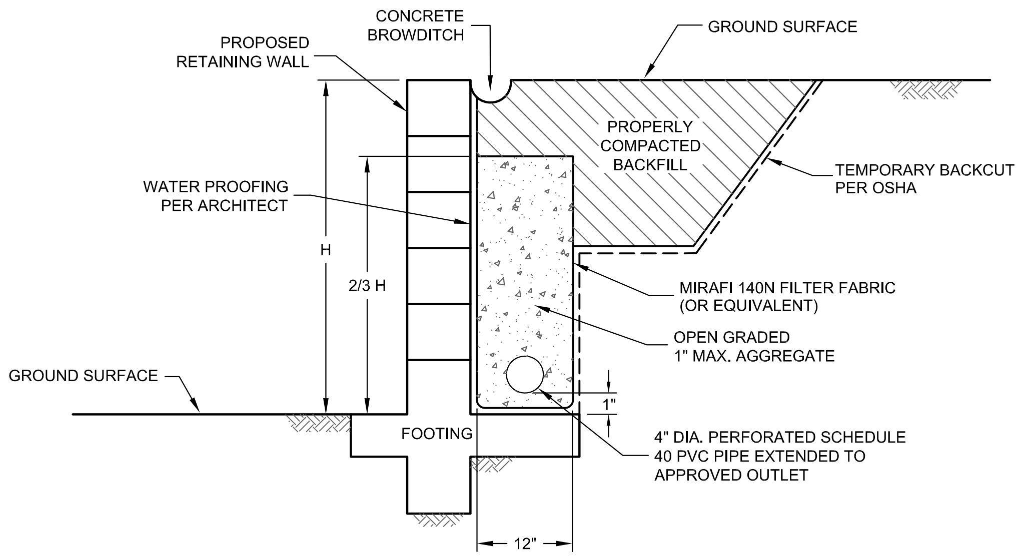
WALL / COLUMN FOOTING DIMENSION DETAIL

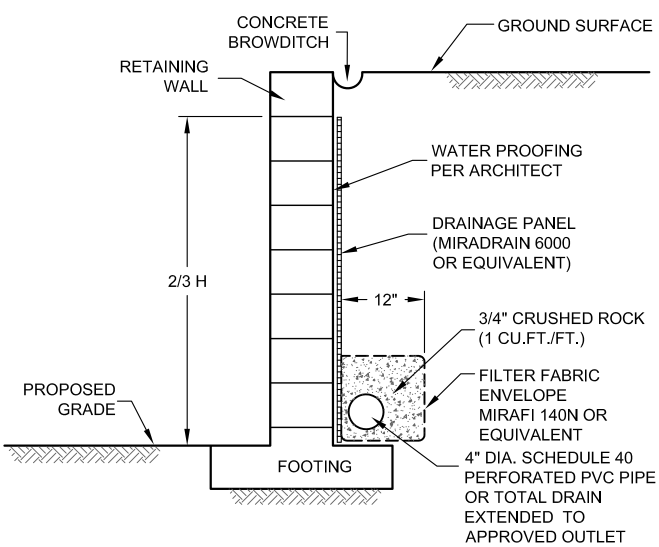
GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

PLAZA LA MEDIA - NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

RM / CW		DSK/GTYPD	DATE 09 - 11 - 2017	PROJECT NO. 07056 - 32 - 04	FIG. 6

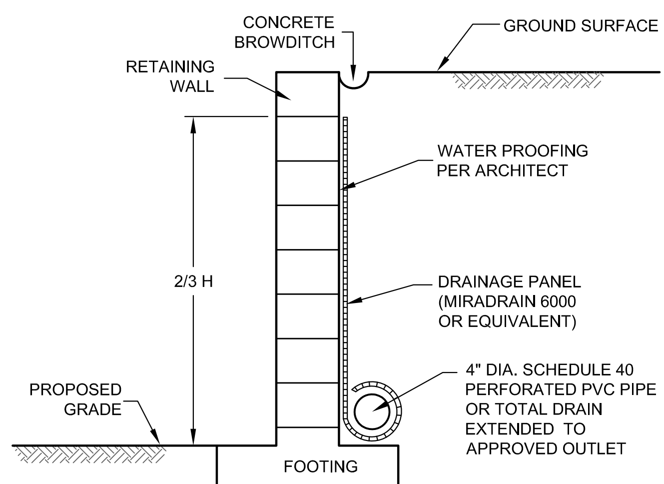
Plotted:09/11/2017 12:39PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\07056-32-04 (Plaza La Media)\DETAILS\Wall-Column Footing Dimension Detail (COLFOOT2).dwg





NOTE :

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



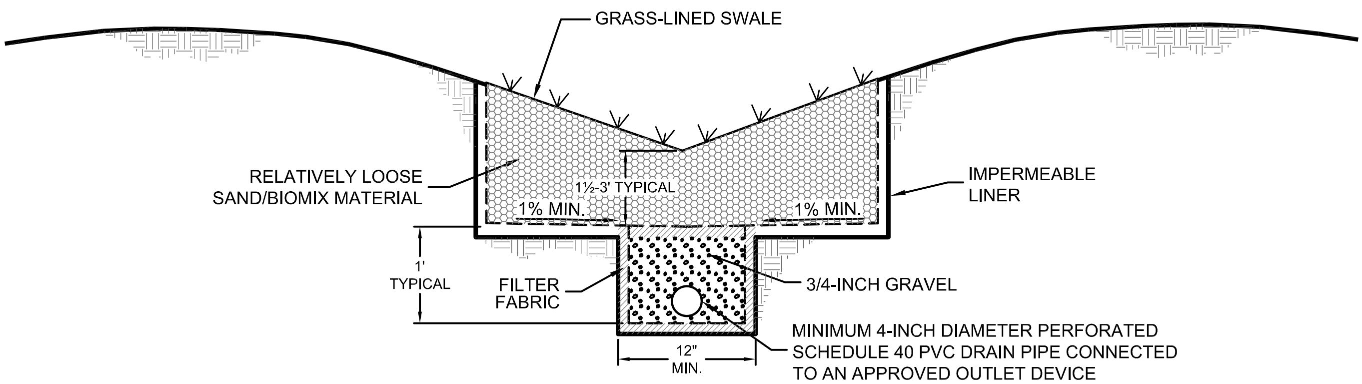
NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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

PLAZA LA MEDIA - NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

RM / CW		DSK/GTYPD	DATE 09 - 11 - 2017	PROJECT NO. 07056 - 32 - 04	FIG. 7



NO SCALE

TYPICAL BIORETENTION BASIN DETAIL

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

PLAZA LA MEDIA - NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

RM / CW		DSK/GTYPD	DATE 09 - 11 - 2017	PROJECT NO. 07056 - 32 - 04	FIG. 8

Plotted:09/11/2017 12:36PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\07056-32-04 (Plaza La Media)\DETAILS\VSBD_2 (Bioswale).dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed between March 20 and March 29, 1989, and consisted of a site reconnaissance by an engineering geologist and the excavation of 2 large diameter borings and 7 backhoe trenches. Borings extended to depths ranging from 20 to 31 feet below the existing ground surface. The large-diameter borings were drilled using an E-100 drill rig equipped with a 30-inch-diameter bucket. Trenches were excavated to depths varying from 10 feet to 18 feet below the existing ground surface using a John Deere 555 tractor-mounted backhoe equipped with a 24-inch-wide bucket. Relatively undisturbed drive samples and disturbed bulk samples were obtained at selected locations within the exploratory excavations.

The soils encountered in the exploratory borings and trenches were visually examined, classified, and logged. Logs of the large diameter borings and trenches are presented on Figures A-1 through A-9. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate location of the exploratory excavations is depicted on the Geologic Map, Figure 2 (map pocket).

File No. D-4342-J01
April 26, 1989
Revised October 13, 1989

BORING B-1
ELEVATION 485 MSL DATE DRILLED 3/28/89
EQUIPMENT E-100

DEPTH IN FEET	SAMPLE NO	SOIL CLASS (U.S.C.S.)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
0 – 2		CL	TOPSOIL Soft, moist, dark brown, fine to medium, Sandy CLAY			
2 – 4.5		CL	TERRACE DEPOSITS Stiff, slightly wet, brown, fine Sandy CLAY			
6	B1-1	CL	Hard, moist, dark orange, silty CLAYSTONE	3	89.0	26.7
8		SC	Dense, moist, orange, slightly Clayey, fine to medium SANDSTONE			
10 – 12	B1-2	CL	Hard, moist, orange, fine Sandy CLAYSTONE layer of Gravel 6" thick	3	101.5	22.1
12 – 16	B1-3	SM	OTAY FORMATION Dense, moist, light gray-pink, Silty, very fine SANDSTONE, micaceous with alternating layers of very hard, light pink, Silty CLAYSTONE	6		
20	B1-4	SM		3	107.4	19.8
26		SM	becomes dark greenish-gray			
			Break in Log			
31	B1-5	SM	BORING TERMINATED AT 31.0 FEET	6	103.5	18.6

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

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.

File No. D-4342-J01
April 26, 1989
Revised October 13, 1989

BORING B-3

ELEVATION 482 MSL DATE DRILLED 3/28/89

EQUIPMENT E-100

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT	DRY DENSITY PCF	MOISTURE CONTENT %
0–3				CL	TOPSOIL Soft, moist, dark brown, fine to medium Sandy CLAY			
3–5.5				CL	TERRACE DEPOSITS Firm, wet, brown, fine Sandy CLAY			
5	B3-1						BULK	SAMPLE
5.5	B3-2					3	120.6	13.0
5.5–20				GC	Dense, moist, dark yellow-brown, Clayey fine to coarse Sandy GRAVEL and Cobbles to 10"			
16					becomes very dense, GRAVEL and Cobbles to 18"			
20					BORING TERMINATED AT 20.0 FEET			

Figure A-3, Log of Test Boring B-3

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

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

File No. D-4342-J01
April 26, 1989
Revised October 13, 1989

TRENCH T-1

ELEVATION 485 MSL DATE DRILLED 3/20/89

EQUIPMENT JD 555

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S.)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
0								
2	T1-1			CL	TOPSOIL Soft, dry, dark brown, fine to medium Sandy CLAY		106.4	13.3
4	T1-2			CL	TERRACE DEPOSITS Stiff, wet, brown, fine Sandy CLAY		82.0	27.6
6 – 8					becomes dark reddish Sandy CLAY			
10 – 12				GW	Dense, moist, dark reddish-orange, Silty, fine to very coarse Sandy GRAVEL and Cobbles to 10"			
14					TRENCH TERMINATED AT 13.0 FEET			

Figure A 5, Log of Test Trench T 1

SAMPLE SYMBOLS			
	☐ — SAMPLING UNSUCCESSFUL	— TAN ARD PENETRATION TEST	— DRIVE SAMPLE (UNDISTURBED)
	⊠ — DISTURBED OR BAG SAMPLE	— H NK SAMPLE	— WATER TABLE OR SEEPAGE

NOTE THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE F FIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED IT IS NOT WARRANTED TO BE REPRESENTATIVE OF UB RF N ITIONS AT OTHER LOCATIONS AND TIMES

File No. D-4342-J01
April 26, 1989
Revised October 13, 1989

TRENCH T-2

ELEVATION 483 MSL DATE DRILLED 3/20/89

EQUIPMENT JD 555

DEPTH IN FEET	SAMPLE NO	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT	DRY DENSITY P.C.F	MOISTURE CONTENT %
0	T201			CL	TOPSOIL Soft, dry, dark brown, fine Sandy CLAY		BULK SAMPLE	
2	T2-2				becomes stiff and moist			
4				CL	TERRACE DEPOSITS Stiff, moist, dark orange, fine to coarse, Sandy CLAY			
6	T2-3						117.9	11.9
8				SM	Hard, moist, dark orange, Silty, fine to coarse SANDSTONE			
10	T2-4						122.6	10.4
12				GW	Dense, moist, dark orange, Silty, fine to very coarse Sandy GRAVEL and Cobbles to 18"			
14								
16								
18					TRENCH TERMINATED AT 17.5 FEET			

Figure A-6, Log of Test Trench T-2

SAMPLE SYMBOLS

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

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

File No. D-4342-J01
April 26, 1989
Revised October 13, 1989

TRENCH T-3

ELEVATION 486 MSL DATE DRILLED 3/20/89

EQUIPMENT JD 555

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S.)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT %
0–3	T3-1			CL	TOPSOIL Soft, dry, dark brown, fine Sandy CLAY		109.5	8.5
3–10	T3-2 T3-3			CL	TERRACE DEPOSITS Stiff, moist, dark gray, fine Sandy CLAY some GRAVEL becomes dark reddish-orange		BULK SAMPLE 103.6	 20.8
				GP	Dense, moist, dark reddish-brown, Silty, fine to very coarse Sandy GRAVEL and Cobbles to 10"			
					TRENCH TERMINATED AT 10.0 FEET			

TRENCH T-4 Elevation 481 MSL

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S.)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT %
0–3	T4-1			CL	TOPSOIL Firm, dry, dark brown, fine Sandy CLAY		107.2	14.0
3–11	T4-2			CL	TERRACE DEPOSITS Stiff, moist, dark orange-brown, fine Sandy CLAY some GRAVEL		99.7	21.1
				GM	Dense, moist, orange, Silty, fine to very coarse Sandy GRAVEL and Cobbles to 10"			
12					TRENCH TERMINATED AT 11.0 FEET			

Figure A-7, Log of Test Trenches T-3 and T-4

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

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

File No. D-4342-J01
April 26, 1989
Revised October 13, 1989

TRENCH T-5

ELEVATION 483 MSL DATE DRILLED 3/20/89

EQUIPMENT JD 555

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	T5-1			CL	TOPSOIL Soft, damp, dark brown, fine Sandy CLAY			
4	T5-2			CL	TERRACE DEPOSITS Stiff, slightly wet, dark gray, fine Sandy CLAY		95.6	19.9
6								
8								
10	T5-3						93.9	27.2
12				GM	Dense, moist, dark orange-red, Silty, fine to very coarse Sandy GRAVEL and Cobbles to 6"			
14					TRENCH TERMINATED AT 12.5 FEET			

TRENCH T-6 Elevation 485 MSL

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	T6-1			CL	TOPSOIL Soft, dry, dark brown, fine to medium, Sandy CLAY		120.0	7.9
4	T6-2			CL	TERRACE DEPOSITS Stiff, slightly wet, dark reddish-brown, fine Sandy CLAY		95.5	23.5
6								
8					Dense, moist, orange, Clayey, fine to medium SANDSTONE			
10	T6-3			SC	TRENCH TERMINATED AT 11.0 FEET		108.3	14.0

Figure A-8, Log of Test Trenches T-5 and T-6

SAMPLE SYMBOLS

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

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

File No. D-4342-J01
April 26, 1989
Revised October 13, 1989

TRENCH T-7

ELEVATION 484 MSL DATE DRILLED 3/21/89

EQUIPMENT JD 555

DEPTH IN FEET	SAMPLE NO	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT	DRY DENSITY P.C.F	MOISTURE CONTENT %
0								
2	T7-1			CL	TOPSOIL Firm, damp, dark brown, fine Sandy CLAY		110.3	13.0
4				CL	TERRACE DEPOSITS Stiff, moist, dark reddish-brown, fine Sandy CLAY		104.9	13.8
6	T7-2							
	T7-3				becomes dark orange		BULK	SAMPLE
8								
10	T7-4			SM	Dense, moist, dark orange, Silty, fine to coarse SANDSTONE		108.3	14.9
12								
14	T7-5						104.6	15.9
16	T7-6			SM	Dense, moist, light brown, slightly Silty, very fine to medium SANDSTONE, micaceous		BULK	SAMPLE
18					TRENCH TERMINATED AT 18.0 FEET			

Figure A-9, Log of Test Trench T-7

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

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

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their maximum dry density and optimum moisture content, expansion index, and shear strength characteristics. Selected soils samples were also tested to evaluate plasticity, water-soluble sulfate, water-soluble chloride, pH, and minimum resistivity characteristics.

The results of our laboratory tests are presented as follows on Tables B-I through B-VI. The in-place dry density and moisture content results are indicated on the exploratory boring and trench logs.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T2-1	Dark brown, Sandy CLAY	124.4	11.3
T3-2	Dark gray, Sandy CLAY	119.0	13.3
T8-4	Dark red, Silty, fine to medium SAND	121.0	12.3

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

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
T2-1*	112.3	10.9	260	21
T3-2*	107.4	12.9	370	8
B8-4*	109.7	11.5	270	26
B1-2	101.5	22.1	400	25
B2-2	93.8	27.0	1950	22

*Soil samples remolded approximately to 90 percent relative density at near optimum moisture content.

TABLE B-III
SUMMARY OF LABORATORY CALIFORNIA BEARING RATIO TEST RESULTS

Description	Sample No. T2-1	Sample No. T7-3
% + #4 Screen	98.6	98.2
% - #4 Screen	1.4	1.8
Sand Equivalent	---	---
CBR Value @ :		
0.1" penetration	2.7	2.7
0.2" penetration	3.2	3.5
0.3" penetration	3.4	4.1
0.4" penetration	3.5	4.2
0.5" penetration	3.5	4.3
% Moisture before soaking	10.4	12.3
% Moisture after soaking	21.9	25.3
Compacted dry weight, pcf	114.4	108.6
96-hour expansion, %	3.9	9.1

TABLE B-IV
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index	Potential Expansion	Type of Soil
	Before Test (%)	After Test (%)				
T2-1	10.1	30.0	103.0	105	High	Topsoil
T3-2	11.7	30.3	102.8	82	Medium	Terrace Deposits (clays)
T8-4	9.9	25.7	109.2	60	Medium	Terrace Deposits (sands)
T11-5	11.5	26.3	103.5	85	Medium	Terrace Deposits

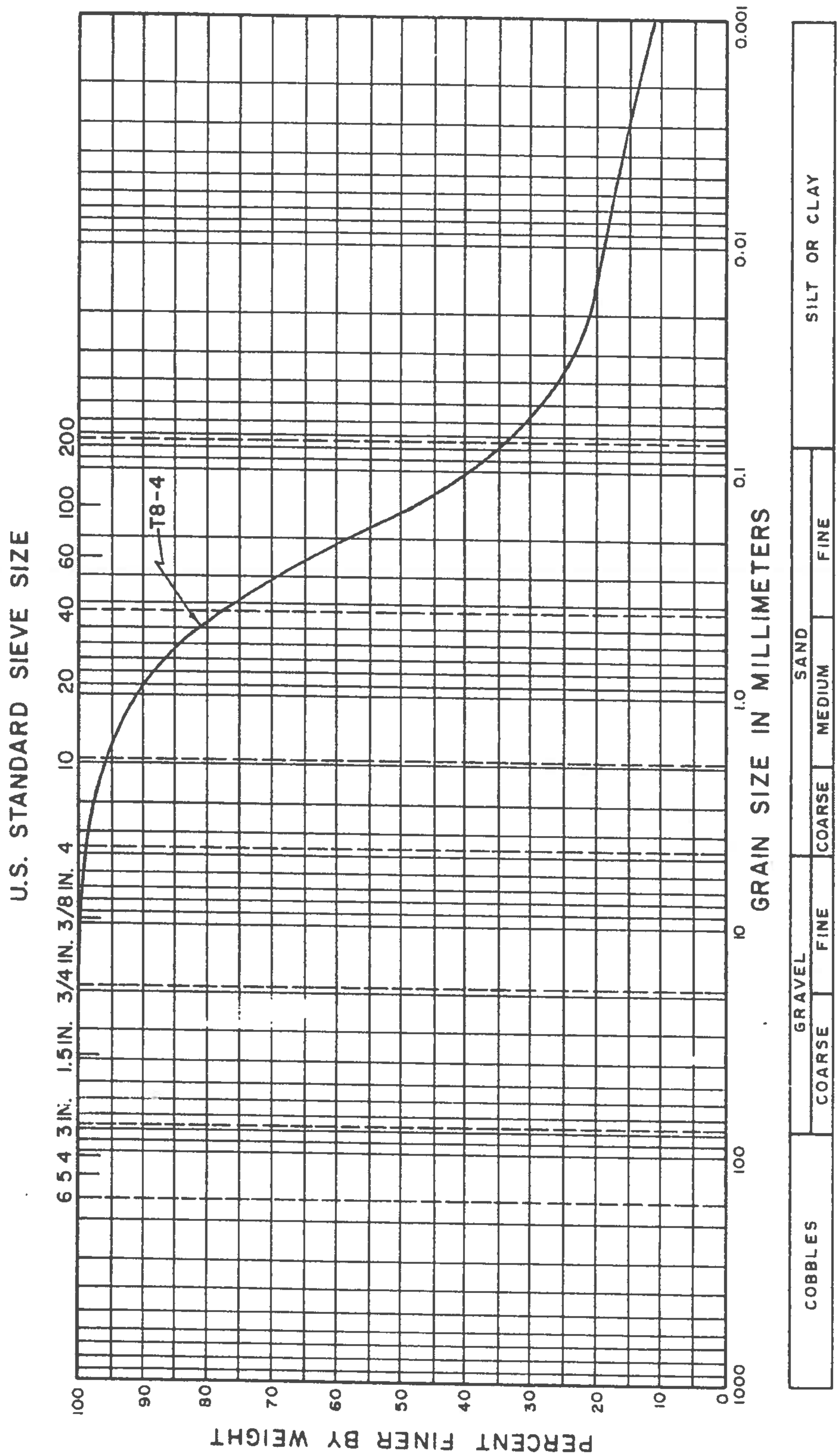
TABLE B-V
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	Category
T2-1	35	13	22	CL
T3-2	44	14	30	CL
T8-4	30	18	12	CL
T11-5	30	18	12	CL

TABLE VI
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (PH), WATER-SOLUBLE SULFATES, AND WATER-SOLUBLE CHLORIDES TEST RESULTS

Sample No.	Resistivity (ohm-cm)	pH	Water-Soluble Sulfates (%)	Water-Soluble Chlorides (%)
T2-1	1260	7.4	0.004	0.002
T3-2	390	7.6	0.031	0.006
T8-4	620	7.5	0.020	0.004

File No. D-4342-J01
April 26, 1989



SAMPLE NO.	DEPTH	CLASSIFICATION		NAT. WC	LL	PL	PI
T8-4	9-11 Feet	SP-SM	Poorly Graded, Silty SAND				

GRADATION CURVE

R-105

Figure B-1

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

PLAZA LA MEDIA–NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

PROJECT NO. 07056-32-04

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

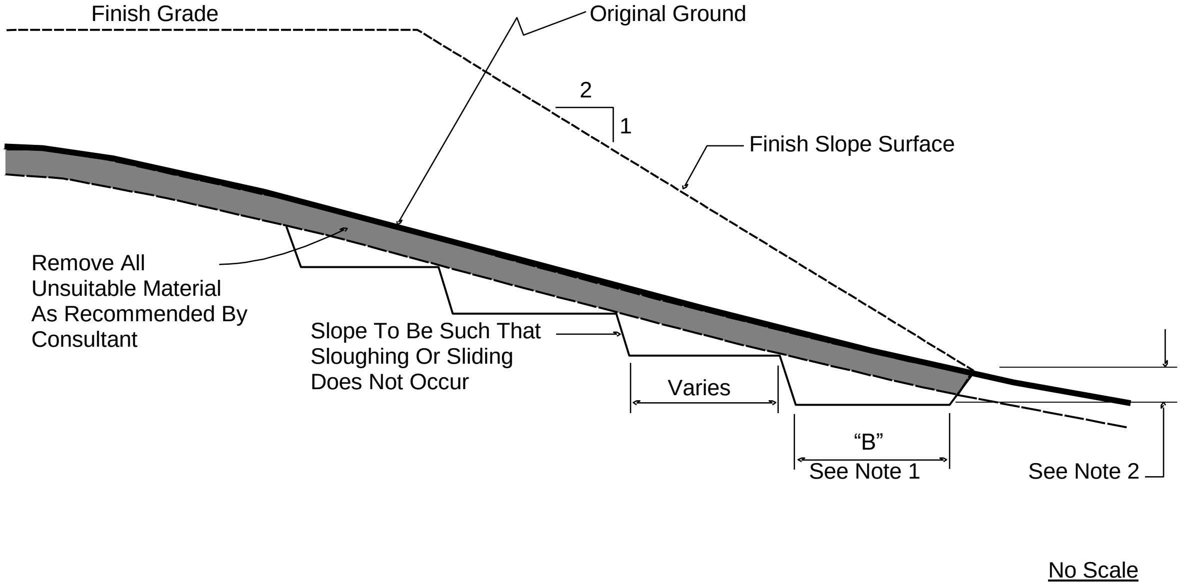
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

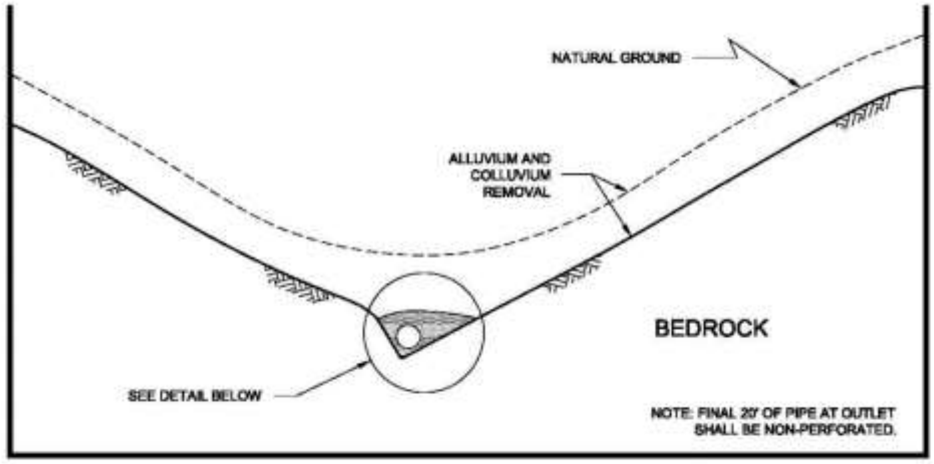
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

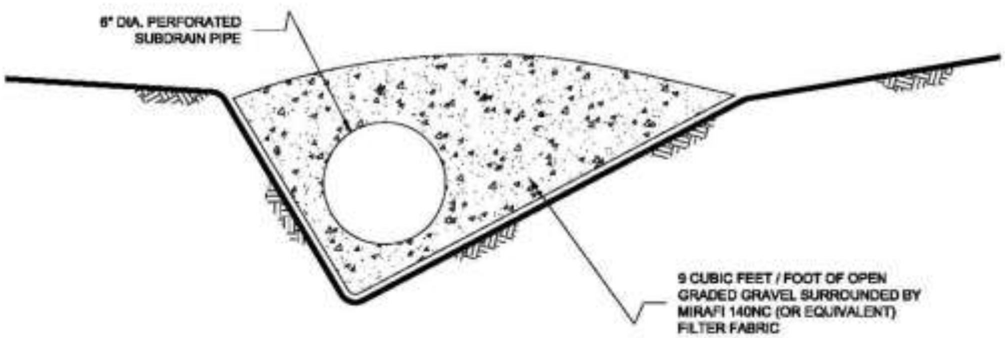
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

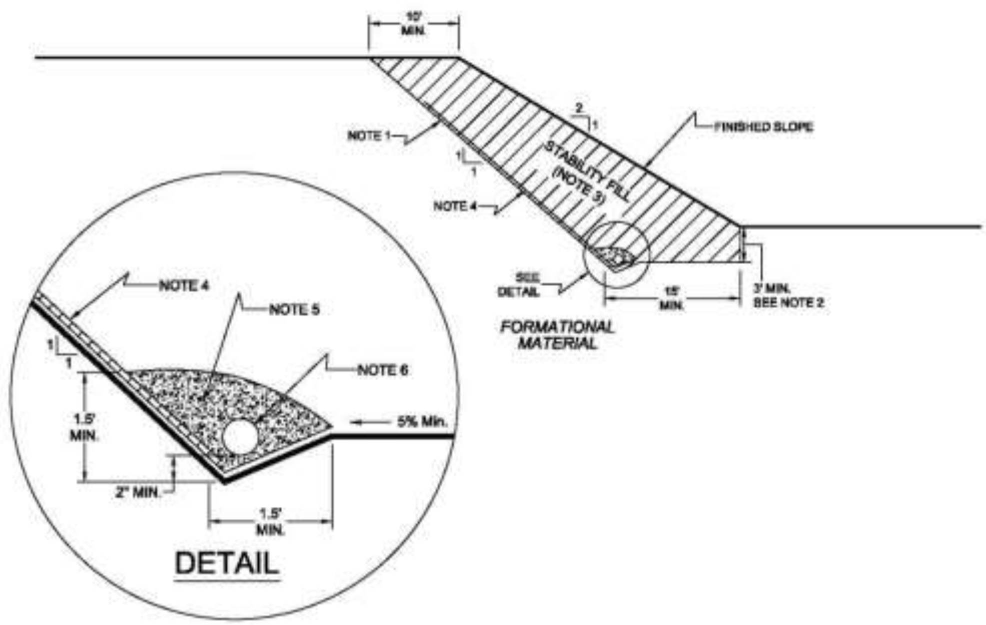
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

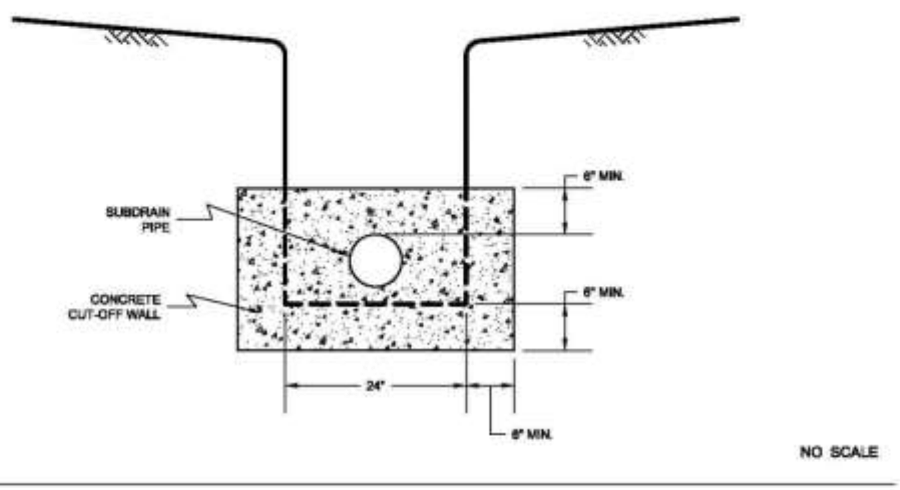
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

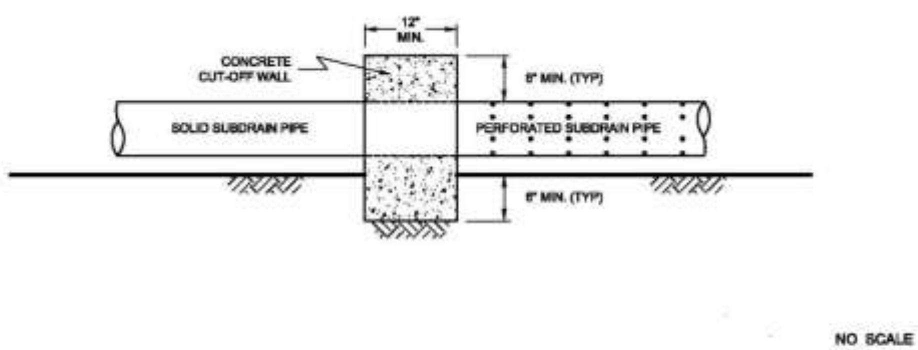
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

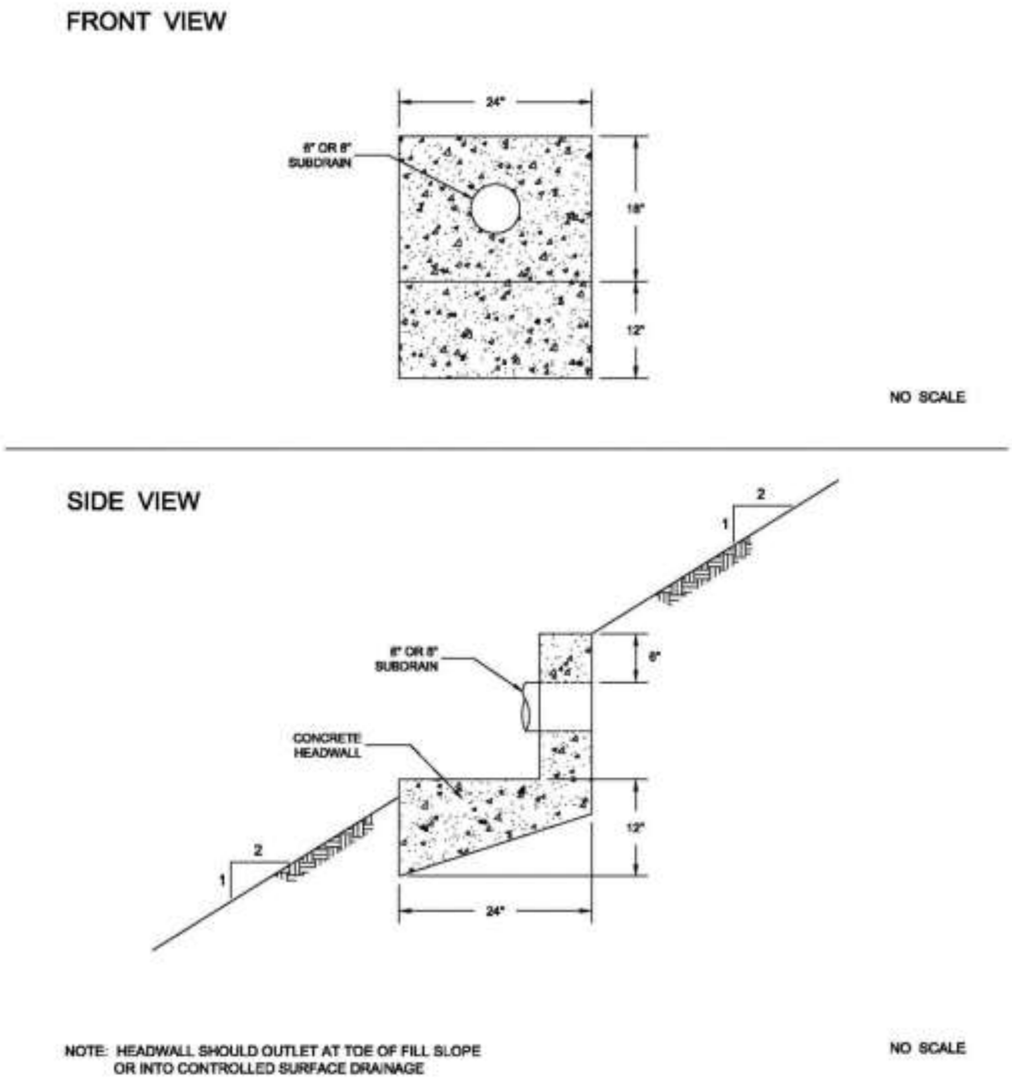


SIDE VIEW



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

Blake, T. F., *EQFAULT, A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults*, User's Manual, 1989a, p. 79, updated 2000.

--------, *EQFAULT, A Computer Program for the Estimation of Peak Horizontal Acceleration from Southern California Historical Earthquake Catalogs,* User's Manual, 1989b, p. 94 (updated, 1997).

City of San Diego Seismic Safety Study, Geologic Hazards and Faults, prepared by the City of San Diego Development Services Department, 1995 edition.

Geocon Incorporated, 1997, *Update Geotechnical Investigation [for] Sun Road Otay Center (Otay Mesa III Limited), T.M. 91-0394, San Diego, California,* dated September 17.

--------, *Soil and Geologic Investigation [for] Otay Mesa International Plaza Limited, San Diego, California,* prepared by Geocon Incorporated, Revised date October 13, 1989 (Project No. D-4342-J01).

--------, *Updated Geotechnical Investigation [for] Judd and Dillard LLC (Otay Mesa International Plaza Limited), San Diego, California,* dated March 14, 2003 (Project No. 07056-22-01).

--------, 1989a, *Soil and Geologic Investigation [for] Otay Mesa III Limited, San Diego, California* dated April 26, 1989 revised October 13 (Project No. D-4341-J01).

--------, 1989b, *Soil and Geologic Investigation for San Diego Mesa Center, Tract 86-1006, San Diego, California,* dated October 19 (Project No. D-4435-J01).

Jennings, C. W., 1994, *Fault Map of California with locations of Volcanoes, Thermal springs and Thermal Walls*, California Division of Mines and Geology, California Geologic Data Map Series Map No. 6.

Kennedy, Michael P., and Siang S. Tan, *Geology of National City, Imperial Beach, and Otay Mesa Quadrangles, Southern San Diego Metropolitan Area, California,* California Division of Mines and Geology, map sheet 29, 1997.

Kettler Leweck Engineering, *Grading and Drainage Plans for Plaza la Media-North,* received via e-mail August 21, 2017.

Landslide Hazards in the Southern Part of the San Diego Metropolitan Area, San Diego County, California, Division of Mines and Geology Open-File Report 95-03, Department of Conservation, Division of Mines and Geology, 1995

Sadigh, *et al.*, 1997, *Attenuation relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data*, Seismological Research Letters, Vol. 68, No. 1, January/February, pp. 180-189.

Project No. 07056-42-06
April 15, 2020

Western Alliance Bank REO/Commercial Facilities
℅ Bank of Nevada
2700 West Sahara Avenue, 5th Floor
Las Vegas, Nevada 89102

Attention: Ms. Anne Marie Berg

Subject: GEOTECHNICAL ENGINEERING CONSULTATION
PLAZA LA MEDIA NORTH
OTAY MESA ROAD AND LA MEDIA ROAD
SAN DIEGO, CALIFORNIA

References:

1. *Soil and Geologic Investigation for Otay Mesa International Plaza Limited, San Diego, California*, prepared by Geocon Incorporated, dated April 26, 1989 (Project No. D-4342-JO1).

2. *Update Geotechnical Investigation for Judd and Dillard, LLC (Otay Mesa International Plaza Limited), San Diego, California*, prepared by Geocon Incorporated, dated March 14, 2003 (Project No. 07056-22-01).

3. *Update Geotechnical Investigation for Plaza La Media-North, Otay Mesa Road and La Media Road, San Diego, California*, prepared by Geocon Incorporated, dated September 11, 2017 (Project No. 07056-32-04).

4. *Storm Water Management Recommendations for Plaza La Media-North*, prepared by Geocon Incorporated, revised date January 15, 2018 (Project No. 07056-32-04).

5. *Site Plan for Majestic La Media North, Sheet A1, prepared by Kimley Horn and Associates, received via email February 21,* 2020.

6. *SWQMP Addendum for PDP SWQMP Plaza La Media-North*, prepared by Chang Consultants, dated February 28, 2019.

Dear Ms. Berg:

In accordance with the request of Mr. Bryan Nord with Kimley Horn and Associates, we have prepared this letter regarding the land use change from commercial to industrial.

Based on the review of the above mentioned reports and site plan, it is our opinion that the recommendations presented in our reports remain applicable for the intended industrial land use.

Should you have any question regarding this letter, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON INCORPORATED

Raúl R. Garcia
GE 2842

RRG:arm

(e-mail) Addressee
(e-mail) Atlantis Group
Attention: Mr. Theodore R. L. Shaw
(e-mail) Bank of Nevada
Attention: Ms. Geysy Fernandez
(e-mail) Kimley Horn and Associates, Incorporated
Attention: Mr. Bryan Nord and Mr. Michael Knapton