Costa Verde Center Revitalization Project

Environmental Impact Report

SCH No. 2016071031; Project No. 477943

Appendix F

Geologic Reconnaissance Report

March 2020

GEOLOGIC RECONNAISSANCE REPORT

COSTA VERDE
CENTER REDEVELOPMENT
8650 GENESEE AVENUE
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

REGENCY CENTERS
SOLANA BEACH, CALIFORNIA

JULY 28, 2016
PROJECT NO. G1927-11-01

Project No. G1927-11-01
July 28, 2016

Regency Centers
420 Stevens Avenue, Suite 320
Solana Beach, California 92075

Attention: Mr. Gregg Sadowsky

Subject: GEOLOGIC RECONNAISSANCE REPORT
COSTA VERDE CENTER REDEVELOPMENT
8650 GENESEE AVENUE
SAN DIEGO, CALIFORNIA

Dear Mr. Sadowsky:

In accordance with your authorization of our Proposal No. LG-15417 dated November 9, 2015, we prepared this geologic reconnaissance report for the proposed Costa Verde Center redevelopment project. We understand that this report will be used to supplement the preparation of the EIR document for the project.

The accompanying report describes the general site soil, geologic conditions and limited geotechnical recommendations based on a desktop study. This report also includes field infiltration testing and storm water management recommendations.

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

Very truly yours,

GEOCON INCORPORATED

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GEOLOGIC RECONNAISSANCE REPORT

1. PURPOSE AND SCOPE

This report presents the results of a geologic reconnaissance for use in preparation of an EIR document. The Costa Verde Center is located at 8650 Genesee Avenue within the University Town Center (UTC) area of San Diego, California (see Vicinity Map, Figure 1). The purpose of this study is to review the referenced geotechnical documents (see List of References) and evaluate the existing geologic conditions and the geologic/geotechnical hazards that may affect re-development of the property. In addition, we performed field infiltration testing and prepared storm water management recommendations that also included preparation of Worksheet C.4-1.

The scope of our study consisted of performing site visits to observe the current site conditions, perform three field infiltration tests, and review of the site plans titled *Preliminary Concept Plan, Regency Centers: Costa Verde Center, Marketing Package*, prepared by Callison Architects, dated July 2016. In addition, we also reviewed the proposed grading and improvement plans prepared by Kimley-Horn, progress date July 25, 2016.

To aid in preparation of this report we reviewed:

1. *Geotechnical Engineering Investigation for Costa Verde, San Diego, California*, prepared by Geocon Incorporated dated April 4, 1986 (Project No. D-2631-J02).

2. *Final Report of Testing and Observation Services During Mass Grading Operations for Costa Verde, Lots 1, 2, 6 through 14, W.O. No. 850783, San Diego, California*, prepared by Geocon Incorporated dated July 17, 1987 (Project No. D-2631-W07).

3. *Final Report of Testing and Observation Services During Mass Grading Operations for Costa Verde, Lots 1 through 14, San Diego*, California, prepared by Geocon Incorporated dated November 19, 1987 (Project No. D-2631-W07).

4. *Soil and Geologic Reconnaissance, Planned 18-inch Sewer, Genesee Avenue and Rose Canyon, San Diego, California*, prepared by Geocon Incorporated dated July 12, 2012 (Project No. G1120-52-01).

5. *Update Geotechnical Report, Monte Verde, Genesee Avenue and La Jolla Village Drive, San Diego, California*, prepared by Geocon Incorporated dated June 4, 2014 (Project No. 05812-52-05).

6. *Geotechnical Engineering Investigation and Geologic Reconnaissance for La Jolla Towers, San Diego, California*, prepared by Geocon Incorporated dated May 28, 1992 (Project No. 04846-35-01).

The conclusions presented herein are based on a review of the geotechnical data for the property and on properties adjacent to this study and our experience with similar soil and geologic conditions in the surrounding area.

2. SITE AND PROJECT DESCRIPTION

The project site is located west of Genesee Avenue, north of Nobel Drive, east of Costa Verde Boulevard and Las Palmas Square Drive with Esplanade Court located within the northern portion of the site. Residential Towers are present to the west and the Monte Verde Towers project is currently in construction to the north. The site is currently occupied by a shopping center with multiple buildings, a two level parking structure on the northern portion of the site with one level partially subterranean and several large areas of on-grade parking. Existing buildings are one to two stories occupied by retail stores. The site generally gently slopes to the south with elevations ranging from about 340 feet above Mean Sea Level (MSL) to about 365 feet MSL at the south and north sides, respectively.

Geocon Incorporated provided the original geotechnical services during the investigation and mass grading operations for the Costa Verde Center in the 1980's as well as the adjacent residential towers to the west. We are also providing geotechnical engineering services during the construction of the Monte Verde Towers project to the north that consists of 4 levels of subterranean parking with excavations of roughly 45 to 50 feet. Geocon also performed the investigation and testing services for the 2015 Genesee Sewer Replacement project fronting the Costa Verde Center property. The previous geotechnical documents applicable to the subject site are referenced herein.

Based on our review of the concept plans prepared by Callison Architects, the planned redevelopment will include a new parking structure integrated with several buildings on the east side of the property that will connect to the elevated new Trolley Station within Genesee Avenue, a 200-room hotel on the northern portion of the property, several new retail buildings as well as modifications to existing retail buildings on the central portion of the property, and areas of on-grade parking. Amenities that will be included in the project include pedestrian friendly areas, patio decks, and a community room. The proposed parking structure will have one level subterranean and four levels above grade.

3. GEOLOGIC SETTING

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

The site is composed of fill soils placed in the 1980's overlying marine deposited Eocene-age Scripps Formation which is roughly 150 feet thick in the general area. Geomorphically the site is located on a former broad marine/non-marine terrace that generally sloped gently to the south toward the existing west flowing Rose Canyon drainage south of Nobel Drive.

4. SOIL AND GEOLOGIC CONDITIONS

Based on review of the referenced reports and our experience in the area with similar projects, the site is underlain by previously placed fill overlying the Scripps Formation. Figure 2 presents our Geologic Map, Figure 3 our geologic cross-sections, and Figure 4 the Regional Geologic Map, respectively. The locations of selected previously excavated deep exploratory borings on and adjacent to the site are presented on Figure 2 and the boring logs are included in Appendix A.

4.1 Previously Placed Fill (Qpf)

We expect localized areas of previously placed fill underlies a majority of the site associated with previous grading operations for the existing shopping center structures and improvements. We performed the testing and observation services performed during overall mass grading operations in the 1980's. We did not provide testing and observation services during subsequent fine grading operations for the building pads and utility trench backfill within the shopping center. Based on review of our previous mass grading reports and the existing finish grades, the majority of the site will have fill with a maximum thickness of approximately 10 to 15 feet (designated as Qpf_2 on Figure 2). A previous canyon drainage located on the south side of the site was filled with a maximum thickness of approximately 35 to 40 feet of compacted fill (designated as Qpf_1 on Figure 2) which included the placement of two canyon subdrains. The previously placed fill is generally composed of clayey or silty, fine to coarse sand and sandy clay. The fill soil will generally possesses a "very low" to "medium" expansion potential (expansion index of 90 or less) and likely possesses "Not Applicable" and "S0" to "Severe" and "S2" sulfate exposure to concrete improvements in contact with the native soils. We expect the upper portions of the previously placed fill impacted by improvements and irrigation practices will not be suitable to support the proposed re-development improvements and some remedial grading would be required.

4.2 Very Old Paralic Deposits (Qvop)

Middle to early Pleistocene-age Very Old Paralic Deposits were encountered at the site previous to mass grading operations in 1987. We encountered approximately 2 to 5 feet of Very Old Paralic Deposits (previously called the Lindavista Formation) underlying topsoil and overlying Scripps Formation during our previous geotechnical investigation performed in 1986. We expect a majority of the Very Old Paralic Deposits was removed during mass grading operations and may have been reused as fill. The Very Old Paralic Deposits generally consist of dense, reddish brown, silty, fine to medium sandstone and sandy siltstone with occasional traces of fine gravel. The Very Old Paralic Deposits likely possesses a “very low” to “low” expansion potential (expansion index of 50 or less) and likely possesses “Not Applicable” and “S0” sulfate exposure to concrete improvements in contact with this formation. The Very Old Paralic Deposits, if present, is considered suitable for additional fill or structural loads for the proposed re-development of the shopping center.

4.3 Scripps Formation (Tsc)

Middle Eocene-age Scripps Formation underlies the previously placed fill and may exist at pad grade within the existing underground parking area. Materials encountered within this formation are variable and consist of hard and very dense, slightly and moderately cemented, light brown, olive brown and gray sandy siltstone, silty to clayey, fine sandstone and localized thick lenses of brown cobble conglomerate. Scripps Formation also typically contains localized areas of highly cemented concretionary beds. The Scripps Formation likely possesses a “very low” to “medium” expansion potential (expansion index of 90 or less) and likely possesses “Not Applicable” and “S0” to “Severe” and “S2” sulfate exposure to concrete improvements in contact with this formation. The Scripps Formation is considered suitable for additional fill or structural loads for the proposed re-development of the shopping center.

5. GROUNDWATER

We do not expect groundwater would significantly affect project development. We expect a permanent groundwater table exists in excess of 150 feet below the ground surface. It is not uncommon for seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered on site. During the rainy season, seepage conditions may develop that would require special consideration during improvement operations. Groundwater elevations are dependent on seasonal precipitation, irrigation and land use, among other factors, and vary as a result. Proper surface drainage will be critical to future performance of the project.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 30 defines the site with a Hazard Category 51: *Level mesas – Underlain by terrace deposits and bedrock – Nominal risk* and a Hazard Category 54: *Other Terrain – Steeply sloping terrain, unfavorable or fault controlled geologic structure, Moderate Risk*. A fault with a length of approximately 500 lineal feet within the Scripps Formation and categorized as potentially active, inactive, presumed inactive, or activity unknown is mapped approximately 100 feet southwest of the site.

6.2 Faulting and Seismicity

Based on a review of geologic literature and experience with the soil and geologic conditions in the general area, it is our opinion that known active, potentially active, or inactive faults are not located at the site. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. In addition to our background review, the site is not mapped in the vicinity of geologic hazards such as landslides, liquefaction areas, or faulting and is not located within the State of California Earthquake Fault Zone.

According to the computer program *EZ-FRISK* (Version 7.65), seven known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database to evaluate the fault parameters. The nearest known active fault is the Newport-Inglewood and Rose Canyon Faults, located approximately 3 miles west of the site and is the dominant source of potential ground motion. Earthquakes that might occur on these fault zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.47g, respectively. Table 6.2.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, 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	3	7.5	0.39	0.37	0.47
Rose Canyon	3	6.9	0.36	0.36	0.43
Coronado Bank	16	7.4	0.21	0.15	0.19
Palos Verdes Connected	16	7.7	0.23	0.17	0.22
Elsinore	35	7.9	0.16	0.10	0.13
Earthquake Valley	42	6.8	0.09	0.06	0.05
Palos Verdes	42	7.3	0.09	0.06	0.06

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the fault's slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS2008, and Chiou-Youngs (2007) NGA USGS2008 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	0.53	0.47	0.56
5% in a 50 Year Period	0.37	0.33	0.38
10% in a 50 Year Period	0.27	0.24	0.26

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 2013 California Building Code (CBC) guidelines currently adopted by the City of San Diego. We understand new 2016 CBC guidelines may go into effect in January 2017, which may require updated seismic design parameters.

6.3 Liquefaction and Seismically Induced Settlement

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soil is cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four of the previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. The potential for liquefaction and seismically induced settlement occurring within the site soil is considered to be negligible due to the very dense nature of the Scripps Formation and lack of groundwater within 50 feet of the ground surface.

6.4 Seiches and Tsunamis

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located in the vicinity of or downstream from such bodies of water. Therefore, the risk of seiches affecting the site is negligible.

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first-order driving force for locally generated tsunamis offshore from southern California is expected to be tectonic deformation from large earthquakes. The property is located at an elevation of about 350 feet above MSL and is about 3 miles from the Pacific Ocean; therefore, the risk of tsunamis affecting the site is negligible.

6.5 Landslides

Examination of aerial photographs in our files, review of published geologic maps for the site vicinity, and the relatively level topography, it is our opinion landslides are not present at the subject property.

6.6 Settlement Potential

The existing fill soil could experience settlement due to new compacted fill and building loading conditions. The magnitude of settlement is dependent on the amount of fill soil present below the improvement and the building loading from the proposed structure. The Scripps Formation will have much smaller settlement magnitudes from proposed building loads due to its very dense conditions. The risk of seismically induced settlement is considered very low due to the dense to very dense nature of the existing fill soil, Very Old Paralic Deposits (where present) and Scripps Formation.

6.7 Shrinkage/Subsidence Potential

Subsidence is a gradual settling or sudden sinking of the ground surface (i.e., loss of elevation). The principal causes of subsidence are aquifer-system compaction, drainage of organic soils, underground mining, and natural compaction. Shrinkage (also known as hydro-consolidation) is the reduction in volume in soil as the water content of the soil changes. The risk due to subsidence and hydro-consolidation affecting the project site is considered to be negligible.

6.8 Slope and Soil Instability

Existing fill slopes have been performing as intended and do not show slope instability or excessive soil erosion. Proper implementation of surface drainage and landscaping practices during future improvements will continue to create stable slopes and soil conditions for the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 From a geotechnical engineering standpoint, it is our opinion that soil or geologic conditions do not exist at the site that would prohibit the planned re-development project. A geotechnical investigation will be required by the City of San Diego to provide additional evaluation of the soil conditions, potential hazards on the property, and site specific recommendations for re-development once grading and structural plans are prepared.

7.1.2 Based on a review of the referenced documents and our experience in the area, we expect the site is generally underlain by previously placed fill overlying Scripps Formation. We expect the on-site soil can be used for properly compacted new fill from a geotechnical engineering standpoint.

7.1.3 We expect groundwater exists in excess of 150 feet below the existing grades or at an elevation below approximately 200 feet MSL. However, it is not uncommon for seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered on site.

7.1.4 We understand the current conceptual plans are preliminary. Therefore, we have prepared this report for use in preparation of an EIR document. We should prepare a geotechnical investigation level report for future improvements to the property once grading and structural plans are prepared.

7.1.5 We expect the existing structures at the site are supported on conventional shallow foundations with a concrete slab-on-grade. Based on limited, visual observations at the property, it appears the structures are behaving as designed from a geotechnical engineering standpoint.

7.1.6 We expect that most of the proposed new structures will be supported on conventional shallow foundations with a concrete slab-on-grade. However, some use of drilled piers may be needed based on lateral loading conditions to existing improvements and potential differential settlements due to differential fill thicknesses. In addition, review of lateral support elements for the adjacent Monte Verde development to the north should be performed to check for construction conflicts.

7.1.7 Adequate drainage provisions are imperative to the performance of the development. Site drainage should be maintained to direct surface runoff into controlled drainage devices.

Positive site drainage should be maintained away from structures and pavements and tops of slopes and directed to storm drain facilities.

7.2 Excavation and Soil Characteristics

7.2.1 Based on the results of expansion index laboratory testing performed during mass grading operations at the site and from adjacent sites, we expect the onsite soil can be considered to be "non-expansive" and "expansive" (expansion index less than 20 and greater than 20, respectively) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 7.2.1 presents soil classifications based on the expansion index. Based on the results of our previous laboratory testing, we expect the on-site materials possesses a "very low" to "medium" expansion potential (Expansion Index of 90 or less).

TABLE 7.2.1
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2013 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

7.2.2 We previously performed laboratory tests on samples of the site materials during mass grading to evaluate the percentage of water-soluble sulfate content. Based on the results from the laboratory water-soluble sulfate content tests previously performed, the on-site materials at the locations tested possess "not applicable" or "S0" to "Severe" or "S2" sulfate exposure to concrete improvements in contact with the project soils as defined by 2013 CBC Section 1904 and ACI 318-08 Sections 4.2 and 4.3. Additional laboratory testing should be performed subsequent to the remedial grading operations. Table 7.2.2 presents a summary of concrete requirements set forth by 2013 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 7.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Severity	Exposure Class	Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	$SO_4<0.10$	--	--	2,500
Moderate	S1	$0.10\leq SO_4<0.20$	II	0.50	4,000
Severe	S2	$0.20\leq SO_4\leq 2.00$	V	0.45	4,500
Very Severe	S3	$SO_4>2.00$	V+Pozzolan or Slag	0.45	4,500

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

7.2.4 Existing fill soil can be excavated with light to moderate effort using conventional heavy-duty grading and trenching equipment. The Scripps Formation will require heavy effort to excavate and may generate oversize rock within localized cemented zones. The oversize materials will likely require export if it cannot be broken down to suitable sizes and properly incorporated in new compacted fill areas. Cemented zones, gravel and cobble layers are not uncommon within the Scripps Formation and may require special excavation equipment such as rock breakers if encountered. This issue may be the focus of future studies. Blasting of the on-site materials will not be required during re-development of the shopping center.

7.3 Seismic Design Criteria

7.3.1 The underlying soil conditions should be evaluated during the future geotechnical investigation. The property will possess Site Class C or D in accordance with 2013 California Building Code (CBC; Based on the 2011 International Building Code [IBC] and ASCE 07-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The Site Class should be evaluated during the future geotechnical investigation based on final locations of buildings and improvements.

7.4 Proposed Foundation Systems

7.4.1 We expect the new buildings can be supported on conventional shallow foundations bearing in property compacted fill or the Scripps Formation. Proposed buildings may require deepened footings or drilled piers such that they do not surcharge adjacent existing or proposed buildings and retaining walls. Footings should be deepened such that they are extended below a 1:1 upward projection from adjacent building and retaining wall footings.

7.5 Site Drainage and Moisture Protection

7.5.1 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 2013 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

7.5.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. A perforated drainpipe of schedule 40 or better should be installed at the base of the wall below the floor slab and drained to an appropriate discharge area. Accordion-type pipe is not acceptable. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

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

7.5.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that 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.

7.6 Storm Water Management Background

7.6.1 We understand storm water management devices are being proposed in accordance with the *2016 City of San Diego Storm Water Standards* (SWS). If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeologic study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

7.6.2 The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table 7.6.1 presents the descriptions of the hydrologic soil groups. If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

TABLE 7.6.1
HYDROLOGIC SOIL GROUP DEFINITIONS

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

7.6.3 The United States Department of Agriculture (USDA), Natural Resources Conservation Services possesses general information regarding the existing soil conditions for areas within the United States. Table 7.6.2 presents the soil name based on the USDA website.

TABLE 7.6.2
USDA SOIL GENERAL INFORMATION

Map Unit Name	Map Unit Symbol	Hydrologic Soil Group	Approximate Percentage of Property
Chesterton Fine Sandy Loam, 2 to 5 Percent Slopes	CfB	D	86
Gaviota Fine Sandy Loam, 30 to 50 Percent Slopes	GaF	D	14

7.6.4 The USDA website also provides the Hydrologic Soil Group as presented in Table 7.6.2. Based on the USDA website, the soil at the site is defined as a Hydrologic Soil Group D. Table 7.6.2 presents the description of Hydrologic Soil Group. Based on the provided table, if a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in the natural condition are in group D are assigned to dual classes.

7.7 In-Situ Testing

7.7.1 The infiltration rate, percolation rates, and saturated hydraulic conductivity are different and have different meanings. Percolation rates tend to overestimate infiltration rates and saturated hydraulic conductivities by a factor of 10 or more. Table 7.7.1 describes the differences in the definitions.

TABLE 7.7.1
SOIL PERMEABILITY DEFINITIONS

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

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

7.7.3 We performed three Aardvark Permeameter tests at the property. The approximate locations of our infiltration tests are shown on Figure 2, Geologic Map. The test borings were 4 inches in diameter and were 4.0 and 6.0 feet deep. The results of the tests provide parameters regarding the saturated hydraulic conductivity and infiltration characteristics of the near surface geologic units. Table 7.7.2 presents the results of the estimated field saturated hydraulic conductivities obtained from the Aardvark Permeameter tests. The field sheets are included in Appendix B. We applied an appropriate factor of safety of 2 to the field results for use in preparation of Worksheet C.4-1. The results indicate an adjusted soil infiltration rate of 0.01 to 0.07 inches per hour or an average rate of 0.03 inches per hour applying a Factor of Safety of 2. Soil infiltration rates from in-situ tests can vary significantly from one location to another due to the heterogeneous characteristics inherent to most soil.

TABLE 7.7.2
FIELD PERMEAMETER INFILTRATION TEST RESULTS

Test No.	Geologic Unit	Test Depth and Elevation (feet, MSL)	Field-Saturated Hydraulic Conductivity, k_{sat} (inch/hour)	Worksheet[1] Saturated Hydraulic Conductivity, k_{sat} (inch/hour)
P-1	Tsc	(-5.2 feet) 355 feet MSL	0.14	0.07
P-3	Tsc	(-6.0 feet) 343 feet MSL	0.02	0.01
P-2	Tsc	(-4.0 feet) 345 Feet MSL	0.04	0.02

[1] Using a factor of safety of 2 for Worksheet C.4-1.

7.8 Storm Water Management Conclusions

7.8.1 The following presents a discussion of the soil types on site regarding storm water infiltration feasibility.

Compacted Fill – Compacted fill exists across the majority of the property to depths of up to about 10 to 15 feet. A canyon fill exists on the southern portion of the site with maximum fill depths of up to 35 to 40 feet. The compacted fill varies in soil type, density and some areas possess relatively high fines content (silt and clay). Water that is allowed to migrate within the compacted fill soil cannot be controlled due to lateral migration potential, would destabilize support for the existing improvements, and would shrink and swell. Therefore, full and partial infiltration should be considered infeasible within existing and proposed compacted fill.

Scripps Formation – The Scripps Formation exists below the compacted fill and consists of very dense and hard, moderately to well cemented silty to clayey sandstones, along with siltstones and claystones. This geologic unit can have a variable expansion potential of "very low" to "medium" (expansion index of 90 or less). Based on the low infiltration rates and the cemented and hard characteristics of this unit, full infiltration is considered infeasible within the Scripps Formation. Partial infiltration can be performed and side liners should be installed to prevent water from migrating within the existing fill materials.

7.8.2 We did not encounter groundwater during the previous grading or drilling operations on the property. The groundwater table will be in excess of 150 feet below existing grades. Therefore, infiltration associated with this risk is considered feasible.

7.8.3 Utilities are located on and adjacent to the property. Therefore, full infiltration near these utilities should be considered infeasible. Mitigation for utilities includes setting back the water management devices from the utility corridors and installing liners to prevent water migration into the utility backfill.

7.8.4 We are unaware of contaminated soil or groundwater on the property. Therefore, infiltration associated with this risk is considered feasible. We should be provided environmental reports if these have been prepared for the property.

7.8.5 Slopes are present within the southern and southeast portion of the site. Infiltration should not be considered within 50 feet of these slopes to reduce the potential for increased seepage forces and slope instability. Therefore, full and partial infiltration should be considered infeasible adjacent to slope areas.

7.8.6 We understand planters may be used as storm water management devices. The planters should be properly lined to prevent water migration into the adjacent improvements. Water storage devices can be installed to reduce the velocity and amount of water entering the storm drain system. The project civil engineer should provide the final design of the storm water management devices.

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

7.9 Storm Water Standard Worksheets

7.9.1 The SWS requests the geotechnical engineer complete the *Categorization of Infiltration Feasibility Condition* (Worksheet C.4-1 or I-8) worksheet information to help evaluate the potential for infiltration on the property. The attached Worksheet C.4-1 presents the completed information for the submittal process.

7.9.2 The regional storm water standards also have a worksheet (Worksheet D.5-1 or Form I-9) that helps the project civil engineer estimate the factor of safety based on several factors. Table 7.9.1 describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE 7.9.1
SUITABILITY ASSESSMENT RELATED CONSIDERATIONS
FOR INFILTRATION FACILITY SAFETY FACTORS

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

7.9.3 Based on our geotechnical investigation and the previous table, Table 7.9.2 presents the estimated factor values for the evaluation of the factor of safety. This table only presents the suitability assessment safety factor (Part A) of the worksheet. The project civil engineer should evaluate the safety factor for design (Part B) and use the combined safety factor for the design infiltration rate.

TABLE 7.9.2
FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	3	0.75
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/ Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \sum p$			2.00

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

7.10 Geotechnical Investigation

7.10.1 A geotechnical investigation will be required by the City of San Diego to provide additional evaluation of the soil conditions, potential hazards on the property, and site specific recommendations for re-development once grading and structural plans are prepared. The field investigation would consist of evaluating proposed building and parking structure locations to perform the proposed field drilling program and sampling of the existing soil conditions.

7.10.2 Laboratory tests should be performed on selected soil samples to evaluate maximum dry density and optimum moisture content, shear strength, expansion characteristics, water-soluble sulfate content, pH, resistivity, chloride-ion content, consolidation, resistance value (R-Value), plasticity index, in-situ dry density and moisture content and gradation of the soil encountered.

7.10.3 The geotechnical investigation report should present the findings, conclusions, and recommendations regarding the geotechnical aspects of structures as proposed in the future. Foundation and concrete slab on-grade design criteria, current California Building Code seismic design parameters, temporary shoring recommendations, excavation characteristics, geologic hazard analyses, and remedial grading measures at the site would be included in the report.

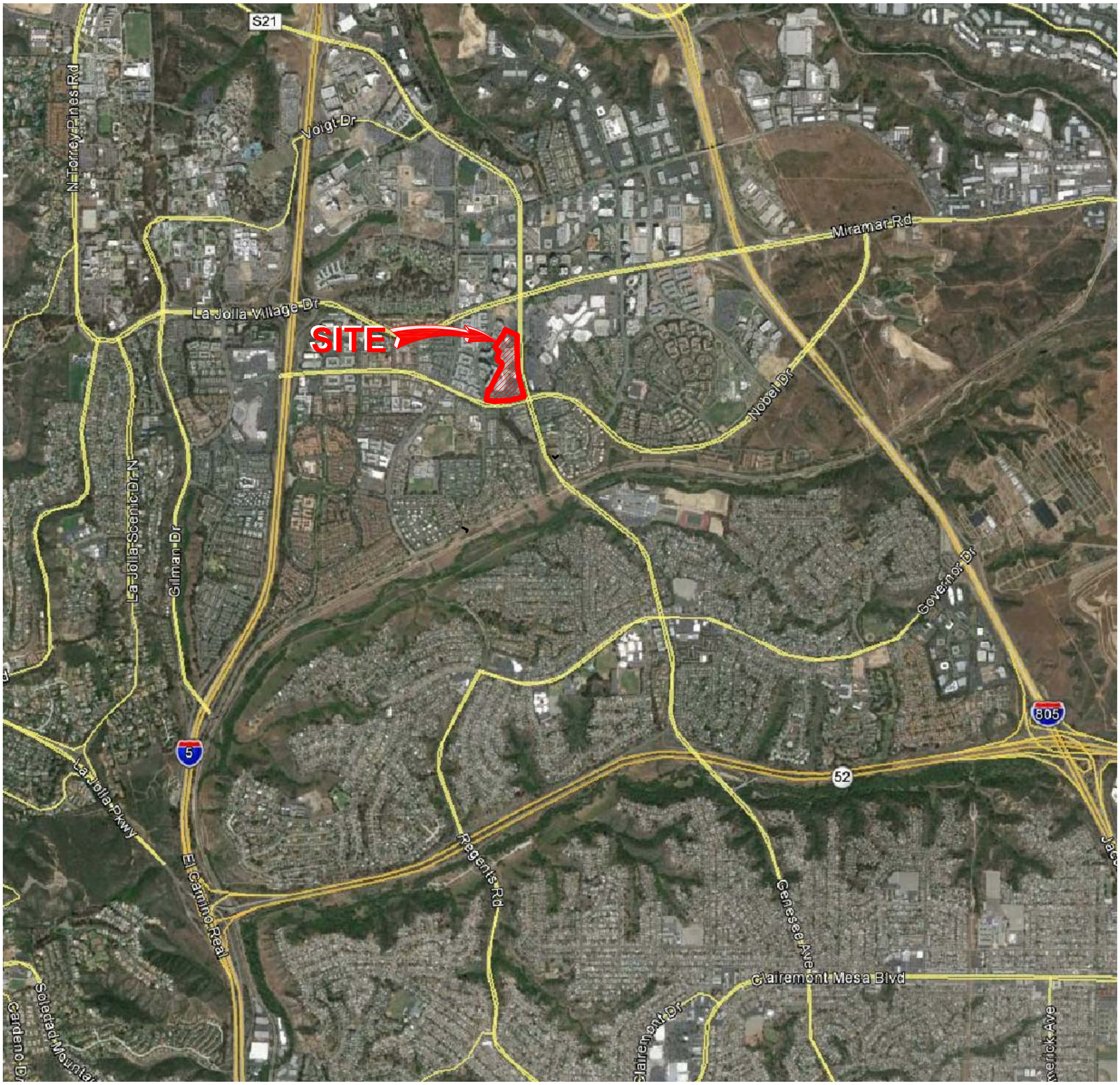
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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



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N

NO SCALE

VICINITY MAP

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GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

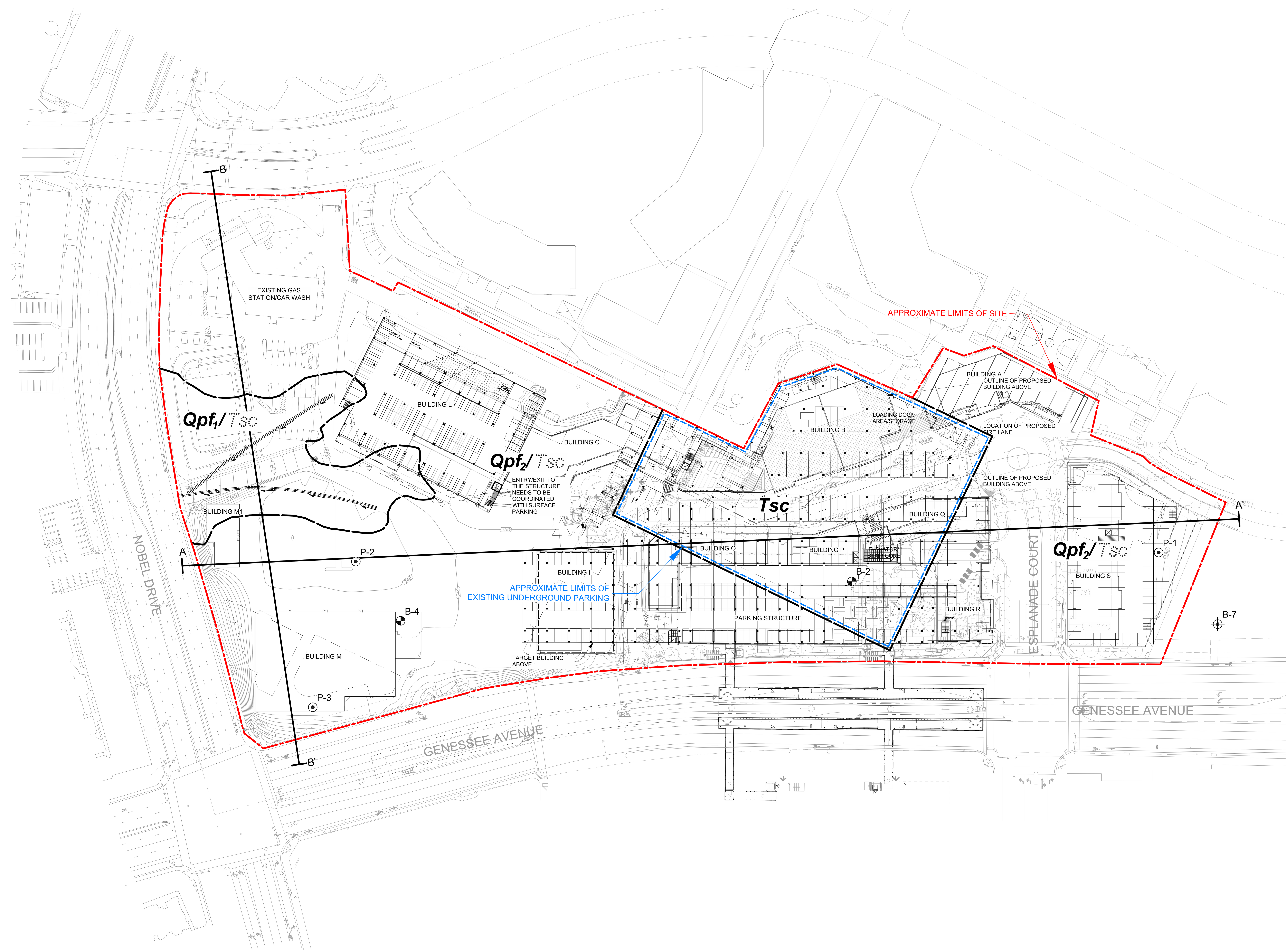
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974

PHONE 858 558-6900 - FAX 858 558-6159

COSTA VERDE CENTER REDEVELOPMENT
8650 GENESEE AVENUE
SAN DIEGO, CALIFORNIA

LR / CW		DSK/GTYPD	DATE 07 - 28 - 2016	PROJECT NO. G1927 - 11 - 01	FIG. 1

Plotted:07/27/2016 10:23AM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G1927-11-01 Costa Verde Center Redevelopment\DETAILS\G1927-11-01_Vicinity Map.dwg



0' 60' 120' 180' 240'

SCALE 1"=60' (On 36x24)

GEOCON LEGEND

- Qpf_1........PREVIOUSLY PLACED CANYON FILL (1986)
- Qpf_2........PREVIOUSLY PLACED FILL (Circa, 1986-1988)
- Tsc........SCRIPPS FORMATION (Dotted Where Buried)
- B-4........APPROX. LOCATION OF PREVIOUS BORING (Geocon, 1986)
- B-7........APPROX. LOCATION OF PREVIOUS BORING (Geocon, 1999)
- P-3........APPROX. LOCATION OF INFILTRATION TEST
- B B'........GEOLOGIC CROSS - SECTION
- ....APPROX. LOCATION OF CANYON SUBDRAINS
- ....APPROX. LOCATION OF GEOLOGIC CONTACT

GEOLOGIC MAP

COSTA VERDE CENTER REDEVELOPMENT
8650 GENESEE AVENUE
SAN DIEGO, CALIFORNIA

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

SCALE 1" = 60'	DATE 07 - 28 - 2016
PROJECT NO. G1927 - 11 - 01	FIGURE 2
SHEET 1 OF 1	

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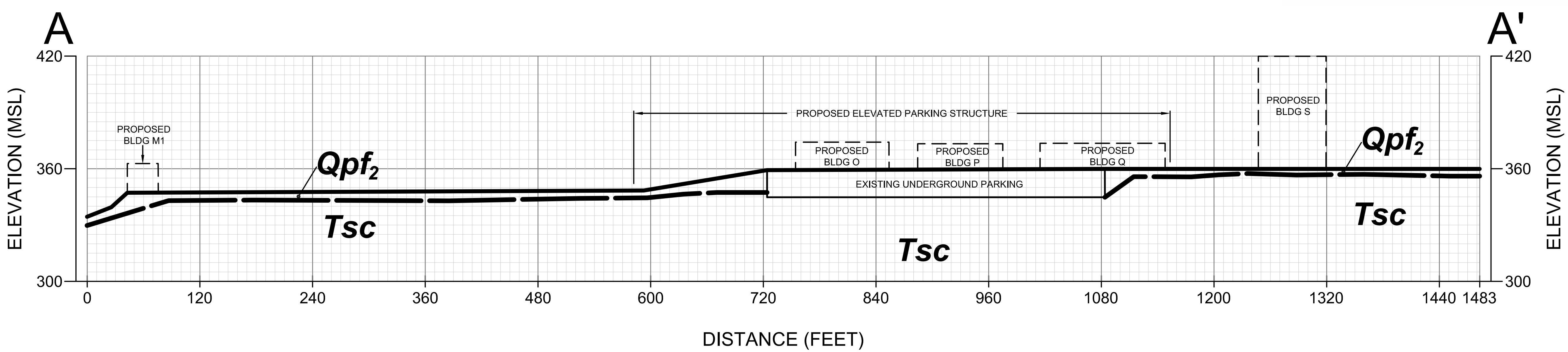
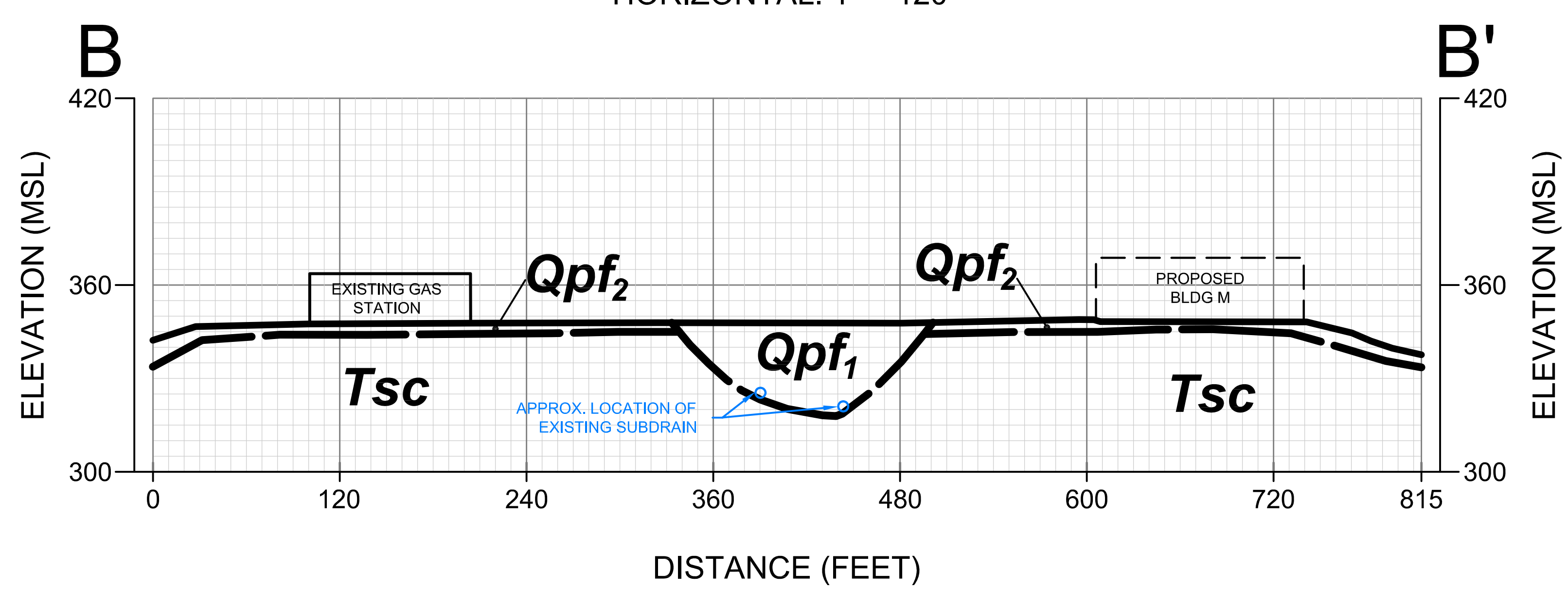
COSTA VERDE CENTER REDEVELOPMENT
8650 GENESSEE AVENUE
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION A-A'

SCALE
VERTICAL: 1" = 60'
HORIZONTAL: 1" = 120'



GEOLOGIC CROSS-SECTION B-B'

SCALE
VERTICAL: 1" = 60'
HORIZONTAL: 1" = 120'

GEOCON LEGEND

Qpf1........PREVIOUS PLACED FILL (1986)
Qpf2........PREVIOUS PLACED FILL (CIRCA, 1986-1988)
Tsc........SCRIPPS FORMATION
........APPROX. LOCATION OF GEOLOGIC CONTACT

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GEOLOGIC CROSS - SECTION
PROJECT NO. G1927 - 11 - 01
FIGURE 3
DATE 07 - 28 - 2016

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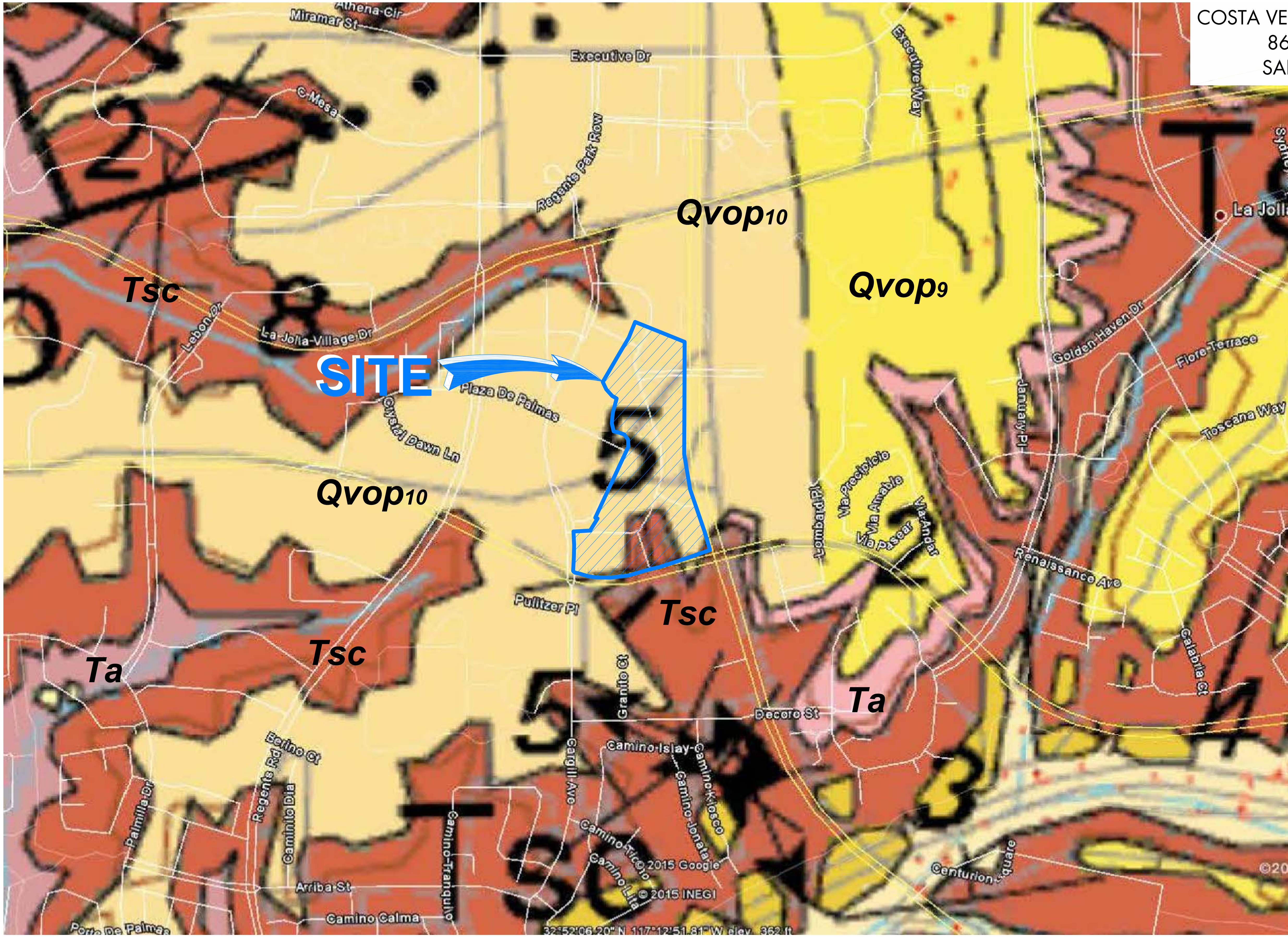

COSTA VERDE CENTER REDEVELOPMENT
8650 GENESEE AVENUE
SAN DIEGO, CALIFORNIA

0' 600' 1200'
SCALE 1"=600'

GEOCON LEGEND

Qvop10	........VERY OLD PARALIC DEPOSITS
Qvop9	........VERY OLD PARALIC DEPOSITS
Tsc	........SCRIPPS FORMATION
Ta	........ARDATH SHALE

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REGIONAL GEOLOGIC MAP

PROJECT NO. G1927 - 11 - 01
FIGURE 4
DATE 07 - 28 - 2016

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APPENDIX A

APPENDIX A

PREVIOUS GEOTECHNICAL BORINGS
(GEOCON, 1986 AND 1999)

FOR

COSTA VERDE CENTER REDEVELOPMENT
8650 GENESEE AVENUE
SAN DIEGO, CALIFORNIA

PROJECT NO. G1927-11-01

File No. D-2631-J02
April 4, 1986

BORING 2

ELEVATION 359± DATE DRILLED 3/19/86

EQUIPMENT Watson 2000

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (U.S.C.S)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
0–1.5		CL	TOPSOILS Soft/loose, moist to very moist, red-brown to gray-brown, fine Sandy Silty CLAY			
1.5–10.5	2-1 (at 4–6)	SM	SCRIPPS FORMATION Dense, slightly moist to moist, orange-brown, fine to medium, Silty SAND		BULK	SAMPLE
10	2-2			8	107.9	16.9
10.5–15		ML	Very dense, slightly moist, gray, fine, Sandy SILTSTONE, little to some clay – becomes very moist (seep)			
15–20		SM	Very dense, slightly moist, gray-orange-brown strata, fine to medium, Sandy SILT			
20	2-3	SM/ML	Very dense, slightly moist, fine to medium, gray-brown, Silty SANDSTONE and Clayey SILTSTONE	25	117.1	10.9
			Break in log			
30	2-4			14/ 6"	112.8	8.1

Figure A-3, Log of Test Boring 2

Continued next page

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-2631-J02
April 4, 1986

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S)	BORING 2 CONTINUED ELEVATION ____ DATE DRILLED ____ EQUIPMENT ____	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
					MATERIAL DESCRIPTION			
36								
					possible CLAY seam			
38								
40								
	2-5					10/ 6"	109.7	7.1
42								
44								
46					possible CLAY seam			
48								
50								
	2-6					12/ 6"	103.1	11.8
52					BORING TERMINATED AT 51.5 FEET			

Figure A-4, Log of Test Boring 2 Continued

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-2631-J02
April 4, 1986

BORING 4

ELEVATION 353± DATE DRILLED 3/20/86

EQUIPMENT Watson 2000

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					TOPSOILS			
2				GM/CL				
					LINDAVISTA FORMATION Soft to medium, wet, red-brown, fine Sandy Silty CLAY, trace of coarse sand, fine to medium gravel and cobble			
4								
6				ML	SCRIPPS FORMATION Dense, dry to slightly moist, orange-brown to gray-brown, fine, Sandy SILT with interbedded SILTSTONES and CLAYSTONES			
8								
					grading slightly moist to moist			
10	4-1					10/ 6"	92.1	28.0
12								
14				ML	very stiff, slightly moist, gray, fine Sandy SILTSTONE layer			
16								
18								
20	4-2					12/ 10"	104.3	18.1
22								
24								
	4-3						BULK	SAMPLE
26								
28								
30								

Figure A-7, Log of Test Boring 4

Continued next page

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-2631-J02
April 4, 1986

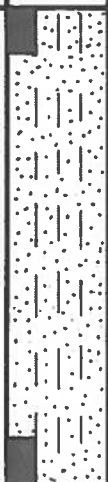
DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S)	BORING 4 CONTINUED ELEVATION ______ DATE DRILLED ______ EQUIPMENT ______	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
					MATERIAL DESCRIPTION			
30	4-4			SM	grading occasional seams of orange-brown-gray, fine, Silty SANDSTONE	10	109.9	17.3
32								
34								
36								
38								
40	4-5					4	98.9	18.3
					BORING TERMINATED AT 40.0 FEET			

Figure A-8, Log of Test Boring 4 Continued

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.

BORING B 7

ELEV. (MSL.) 357 DATE COMPLETED 01-27-1999

EQUIPMENT CME 55

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			ASPHALT BASE MATERIAL			
2	B 7-1	SM	**UNDOCUMENTED FILL (Qudf)** Dense to loose, moist, red-brown to brown, SIlty, fine- to medium SAND, some to trace clay, trace gravel	32	116.7	10.4
4	B 7-2					
	B 7-3			14		
6						
8	B 7-4	SC	Loose, very moist to wet, brown, very Clayey, fine- to medium SAND, trace gravel			
10	B 7-5	ML-SP	**SCRIPPS FORMATION (Tsc)** Very dense, very moist to wet, brown SILT and very fine- to fine SAND	76/10"	118.3	13.5
12						
14			Very dense,m very moist to wet, light brown to red-brown, Silty, very fine- to medium SAND			
16	B 7-6			92/9"	114.2	14.8
18	B 7-7	SM				
20	B 7-8			50/6"	107.7	12.1
22						
24						
26						
28		ML-CH	Very dense, very moist, light brown to red-brown, very fine Sandy SILTSTONE, some gray, silty claystone -Difficult drilling from 29 feet			
30	B 7-9			75/9"	107.8	20.6
32						
34		ML-CH				

Figure A-7,
Log of Boring B 7, Page 1 of 2

05812-52-05 GPJ

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

PROJECT NO. 05812-52-05

BORING B 7

ELEV. (MSL.) 357 DATE COMPLETED 01-27-1999

EQUIPMENT CME 55

DEPTH IN FEET	SAMPLE NO	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
36	B 7-10							
40	B 7-11					81/8"	106.8	19.8
					Very dense, damp, yellow-brown to red-brown, Silty, very fine- to medium SAND -Difficult drilling, water added at 41 feet			
50	B 7-12			SM		50/4"	97.6	5.0
					-Difficult drilling, water added at 51 feet			
60	B 7-13					50/6"	98.9	10.9
					BORING TERMINATED AT 60.5 FEET No Groundwater Encountered			

Figure A-7,
Log of Boring B 7, Page 2 of 2

05812-52-05 GPJ

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

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

APPENDIX B

APPENDIX B

STORM WATER MANAGEMENT WORKSHEETS

FOR

COSTA VERDE CENTER REDEVELOPMENT
8650 GENESEE AVENUE
SAN DIEGO, CALIFORNIA

PROJECT NO. G1927-11-01

Categorization of Infiltration Feasibility Condition			Worksheet C.4-1
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: We obtained the following infiltration rates based on field testing: P-1: 0.14 inches/hour (0.07 with FOS=2) P-2: 0.02 inches/hour (0.01 with FOS=2) P-3: 0.04 inches/hour (0.02 with FOS=2) Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
Provide basis: The project geotechnical report presents compacted fill and the Scripps Formation underlie the property. Water that would be allowed to infiltrate would migrate laterally outside of the property limits to the existing right-of-ways and toward the adjacent downtown properties. Based on the comprehensive geotechnical evaluation and the very low infiltration rates obtained, full infiltration is not feasible due to the dense to very dense and cemented nature of the underlying materials and the potential for distress to adjacent properties. Setbacks from slopes and liners on the sidewalls of the basins will be required to prevent daylight seepage/slope instability and lateral water migration. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

Based on the geotechnical report, groundwater is at least 150 feet below existing grades. Therefore, infiltration (if possible) would be feasible.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

We do not expect infiltration will cause water balance issues such as seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">If all answers to rows 1 - 4 are “Yes” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration

If any answer from row 1-4 is “No”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2</td><td>Not Full Infiltration</td></tr>
</table>

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

Worksheet C.4-1 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

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

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

Provide basis:

We obtained the following infiltration rates based on field testing:

P-1: 0.14 inches/hour (0.07 with FOS=2)
P-2: 0.02 inches/hour (0.01 with FOS=2)
P-3: 0.04 inches/hour (0.02 with FOS=2)

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

Criteria	Screening Question	Yes	No
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	

Provide basis:

The project geotechnical report presents compacted fill and the Scripps Formation underlie the property. Water that would be allowed to infiltrate would migrate laterally outside of the property limits to the existing right-of-ways and toward the adjacent downtown properties. Based on the comprehensive geotechnical evaluation and the very low infiltration rates obtained, partial infiltration within the formational materials can be performed.

Setbacks from slopes and liners on the sidewalls of the basins will be required to prevent daylight seepage/slope instability and lateral water migration.

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

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: Based on the geotechnical report, groundwater is at least 150 feet below existing grades. Therefore, partial infiltration would be feasible. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not provide a study regarding water rights. However, these rights are not typical in the San Diego area. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	Partial Infiltration	

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

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	Costa Verde	Date:	7/22/2016
Project Number:	G1927-11-01	By:	JML
Borehole Location:	P-1	Ref. EL (feet, MSL):	
		Bottom EL (feet, MSL):	

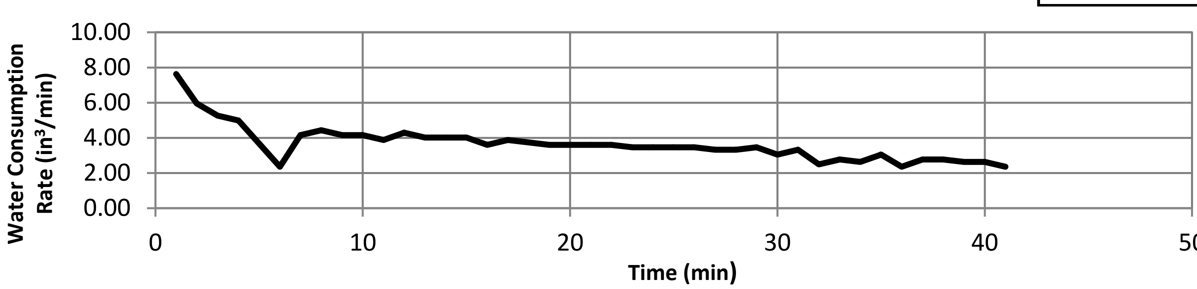
Borehole Diameter (inches):	4.00
Borehole Depth, **H** (feet):	5.17
Distance Between Reservoir & Top of Borehole (feet):	2.33
Depth to Water Table, **s** (feet):	200
Height APM Raised from Bottom (inches):	2.00

Wetted Area, **A** (in^2): 238.76

Distance Between Resevoir and APM, **D** (feet):	6.73
Head Height, **h** (inches):	18.00
Distance Between Constant Head and Water Table, **L** (inches):	2356

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0.00			20.600			
2	1.00	1.00		20.325	0.28	0.28	7.62
3	2.00	1.00		20.110	0.22	0.49	5.96
4	3.00	1.00		19.920	0.19	0.68	5.27
5	4.00	1.00		19.740	0.18	0.86	4.99
6	6.00	2.00		19.570	0.17	1.03	2.36
7	7.00	1.00		19.420	0.15	1.18	4.16
8	8.00	1.00		19.260	0.16	1.34	4.44
9	9.00	1.00		19.110	0.15	1.49	4.16
10	10.00	1.00		18.960	0.15	1.64	4.16
11	11.00	1.00		18.820	0.14	1.78	3.88
12	12.00	1.00		18.665	0.16	1.94	4.30
13	13.00	1.00		18.520	0.15	2.08	4.02
14	14.00	1.00		18.375	0.15	2.23	4.02
15	15.00	1.00		18.230	0.15	2.37	4.02
16	16.00	1.00		18.100	0.13	2.50	3.60
17	17.00	1.00		17.960	0.14	2.64	3.88
18	18.00	1.00		17.825	0.14	2.78	3.74
19	19.00	1.00		17.695	0.13	2.91	3.60
20	20.00	1.00		17.565	0.13	3.04	3.60
21	21.00	1.00		17.435	0.13	3.17	3.60
22	22.00	1.00		17.305	0.13	3.30	3.60
23	23.00	1.00		17.180	0.13	3.42	3.46
24	24.00	1.00		17.055	0.13	3.55	3.46
25	25.00	1.00		16.930	0.13	3.67	3.46
26	26.00	1.00		16.805	0.13	3.80	3.46
27	27.00	1.00		16.685	0.12	3.92	3.33
28	28.00	1.00		16.565	0.12	4.04	3.33
29	29.00	1.00		16.440	0.13	4.16	3.46
30	30.00	1.00		16.330	0.11	4.27	3.05
31	31.00	1.00		16.210	0.12	4.39	3.33
32	32.00	1.00		16.120	0.09	4.48	2.49
33	33.00	1.00		16.020	0.10	4.58	2.77
34	34.00	1.00		15.925	0.09	4.68	2.63
35	35.00	1.00		15.815	0.11	4.79	3.05
36	36.00	1.00		15.730	0.08	4.87	2.36
37	37.00	1.00		15.630	0.10	4.97	2.77
38	38.00	1.00		15.530	0.10	5.07	2.77
39	39.00	1.00		15.435	0.09	5.17	2.63
40	40.00	1.00		15.340	0.10	5.26	2.63
41	41.00	1.00		15.255	0.08	5.35	2.36

Steady Flow Rate, Q (in^3/min): 2.36



Field-Saturated Hydraulic Conductivity

Case 1: L/h > 3 K_{sat} = 0.002 in/min 0.14 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	Costa Verde	Date:	1/22/2016
Project Number:	G1927-11-01	By:	JML
Borehole Location:	P-2	Ref. EL (feet, MSL):	
		Bottom EL (feet, MSL):	

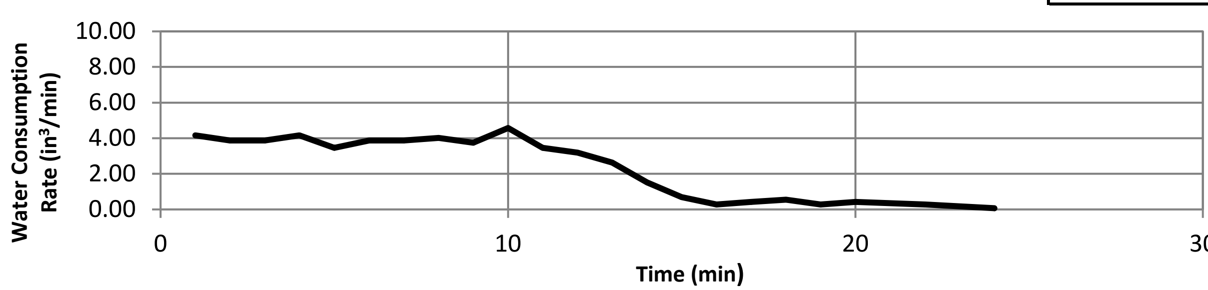
Borehole Diameter (inches):	4.00
Borehole Depth, **H** (feet):	6.00
Distance Between Reservoir & Top of Borehole (feet):	2.50
Depth to Water Table, **s** (feet):	200
Height APM Raised from Bottom (inches):	2.00

Wetted Area, **A** (in^2): 81.68

Distance Between Resevoir and APM, **D** (feet):	7.73
Head Height, **h** (inches):	5.50
Distance Between Constant Head and Water Table, **L** (inches):	2334

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0.00			22.700			
2	1.00	1.00		22.550	0.15	0.15	4.16
3	2.00	1.00		22.410	0.14	0.29	3.88
4	3.00	1.00		22.270	0.14	0.43	3.88
5	4.00	1.00		22.120	0.15	0.58	4.16
6	5.00	1.00		21.995	0.13	0.70	3.46
7	6.00	1.00		21.855	0.14	0.84	3.88
8	7.00	1.00		21.715	0.14	0.98	3.88
9	8.00	1.00		21.570	0.15	1.13	4.02
10	9.00	1.00		21.435	0.14	1.27	3.74
11	10.00	1.00		21.270	0.16	1.43	4.57
12	11.00	1.00		21.145	0.13	1.56	3.46
13	12.00	1.00		21.030	0.11	1.67	3.19
14	13.00	1.00		20.935	0.10	1.77	2.63
15	14.00	1.00		20.880	0.05	1.82	1.52
16	15.00	1.00		20.855	0.02	1.85	0.69
17	16.00	1.00		20.845	0.01	1.86	0.28
18	17.00	1.00		20.830	0.02	1.87	0.42
19	18.00	1.00		20.810	0.02	1.89	0.55
20	19.00	1.00		20.800	0.01	1.90	0.28
21	20.00	1.00		20.785	0.02	1.92	0.42
22	22.00	2.00		20.765	0.02	1.94	0.28
23	24.00	2.00		20.760	0.00	1.94	0.07
24							
25							
26							

Steady Flow Rate, Q (in^3/min): 0.07



Field-Saturated Hydraulic Conductivity

Case 1: L/h > 3 K_{sat} = 0.0004 in/min 0.02 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name: Costa Verde
Date: 7/22/2016
Project Number: G1927-11-02
By: JML
Borehole Location: P-3
Ref. EL (feet, MSL):
Bottom EL (feet, MSL):

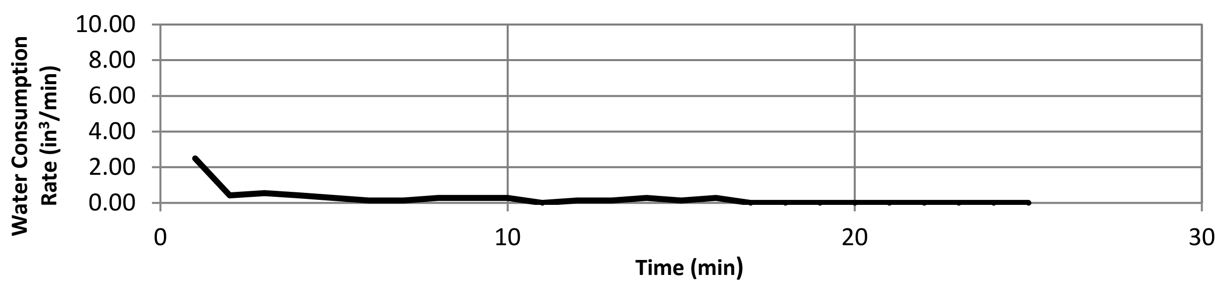
Borehole Diameter (inches): 4.00
Borehole Depth, **H** (feet): 4.00
Distance Between Reservoir & Top of Borehole (feet): 2.50
Depth to Water Table, **s** (feet): 200
Height APM Raised from Bottom (inches): 2.00

Wetted Area, **A** (in^2): 87.96

Distance Between Resevoir and APM, **D** (feet): 5.73
Head Height, **h** (inches): 6.00
Distance Between Constant Head and Water Table, **L** (inches): 2358

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0.00			20.100			
2	1.00	1.00		20.010	0.09	0.09	2.49
3	2.00	1.00		19.995	0.02	0.11	0.42
4	3.00	1.00		19.975	0.02	0.13	0.55
5	4.00	1.00		19.960	0.02	0.14	0.42
6	6.00	2.00		19.950	0.01	0.15	0.14
7	7.00	1.00		19.945	0.00	0.16	0.14
8	8.00	1.00		19.935	0.01	0.17	0.28
9	9.00	1.00		19.925	0.01	0.18	0.28
10	10.00	1.00		19.915	0.01	0.19	0.28
11	11.00	1.00		19.915	0.00	0.19	0.00
12	12.00	1.00		19.910	0.00	0.19	0.14
13	13.00	1.00		19.905	0.00	0.20	0.14
14	14.00	1.00		19.895	0.01	0.21	0.28
15	15.00	1.00		19.890	0.00	0.21	0.14
16	16.00	1.00		19.880	0.01	0.22	0.28
17	17.00	1.00		19.880	0.00	0.22	0.00
18	18.00	1.00					
19	19.00	1.00					
20	20.00	1.00					
21	21.00	1.00					
22	22.00	1.00					
23	23.00	1.00					
24	24.00	1.00					
25	25.00	1.00					

Steady Flow Rate, Q (in^3/min): 0.14



Field-Saturated Hydraulic Conductivity

Case 1: L/h > 3 ***K_{sat} =*** 0.001 in/min 0.04 in/hr

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