CLIMATE ACTION PLAN
CONSISTENCY CHECKLIST INTRODUCTION

In December 2015, the City adopted a Climate Action Plan (CAP) that outlines the actions that City will undertake to achieve its proportional share of State greenhouse gas (GHG) emission reductions. The purpose of the Climate Action Plan Consistency Checklist (Checklist) is to, in conjunction with the CAP, provide a streamlined review process for proposed new development projects that are subject to discretionary review and trigger environmental review pursuant to the California Environmental Quality Act (CEQA).[1]

Analysis of GHG emissions and potential climate change impacts from new development is required under CEQA. The CAP is a plan for the reduction of GHG emissions in accordance with CEQA Guidelines Section 15183.5. Pursuant to CEQA Guidelines Sections 15064(h)(3), 15130(d), and 15183(b), a project's incremental contribution to a cumulative GHG emissions effect may be determined not to be cumulatively considerable if it complies with the requirements of the CAP.

This Checklist is part of the CAP and contains measures that are required to be implemented on a project-by-project basis to ensure that the specified emissions targets identified in the CAP are achieved. Implementation of these measures would ensure that new development is consistent with the CAP's assumptions for relevant CAP strategies toward achieving the identified GHG reduction targets. Projects that are consistent with the CAP as determined through the use of this Checklist may rely on the CAP for the cumulative impacts analysis of GHG emissions. Projects that are not consistent with the CAP must prepare a comprehensive project-specific analysis of GHG emissions, including quantification of existing and projected GHG emissions and incorporation of the measures in this Checklist to the extent feasible. Cumulative GHG impacts would be significant for any project that is not consistent with the CAP.

The Checklist may be updated to incorporate new GHG reduction techniques or to comply with later amendments to the CAP or local, State, or federal law.

[1] Certain projects seeking ministerial approval may be required to complete the Checklist. For example, projects in a Community Plan Implementation Overlay Zone may be required to use the Checklist to qualify for ministerial level review. See Supplemental Development Regulations in the project's community plan to determine applicability.

This page intentionally left blank

CAP CONSISTENCY CHECKLIST SUBMITTAL APPLICATION

- The Checklist is required only for projects subject to CEQA review.[2]
- If required, the Checklist must be included in the project submittal package. Application submittal procedures can be found in Chapter 11: Land Development Procedures of the City's Municipal Code.
- The requirements in the Checklist will be included in the project's conditions of approval.
- The applicant must provide an explanation of how the proposed project will implement the requirements described herein to the satisfaction of the Planning Department.

Application Information

Contact Information

Project No./Name: Barrio Logan U-Stor-It

Property Address: 2209 National Avenue, San Diego, CA 92113

Applicant Name/Co.: Larry Nora

Contact Phone: 619-255-7478 Contact Email: larry@ccf-llc.com

Was a consultant retained to complete this checklist? ■ Yes ☐ No If Yes, complete the following

Consultant Name: John Swierk Contact Phone: 815-444-8444

Company Name: DDCA Architects Contact Email: jswierk@ddcaarchitects.com

Project Information

1. What is the size of the project (acres)? 35,130 s.f. (0.806 acres)
2. Identify all applicable proposed land uses:
   - ☐ Residential (indicate # of single-family units):
   - ☐ Residential (indicate # of multi-family units):
   - ■ Commercial (total square footage): 155,052 s.f. (5 levels)
   - ☐ Industrial (total square footage):
   - ☐ Other (describe):
3. Is the project or a portion of the project located in a Transit Priority Area? ■ Yes ☐ No
4. Provide a brief description of the project proposed:

Commercial self-storage building project with parking garage (ground level), two subterranean floors and two floors above the parking garage. An office for operations and retail rental space is located on ground level. The overall gross area of building for the 5 levels is 155,052 s.f. which consists of 2,388 s.f. of office/building operational space, 21,523 s.f. of garage, 1,837 s.f. retail rental space and the balance in storage/circulation area.

[2] Certain projects seeking ministerial approval may be required to complete the Checklist. For example, projects in a Community Plan Implementation Overlay Zone may be required to use the Checklist to qualify for ministerial level review. See Supplemental Development Regulations in the project's community plan to determine applicability.

CAP CONSISTENCY CHECKLIST QUESTIONS

Step 1: Land Use Consistency

The first step in determining CAP consistency for discretionary development projects is to assess the project's consistency with the growth projections used in the development of the CAP. This section allows the City to determine a project's consistency with the land use assumptions used in the CAP.

Step 1: Land Use Consistency		
Checklist Item (Check the appropriate box and provide explanation and supporting documentation for your answer)	Yes	No
A. Is the proposed project consistent with the existing General Plan and Community Plan land use and zoning designations?;[3] OR, B. If the proposed project is not consistent with the existing land use plan and zoning designations, and includes a land use plan and/or zoning designation amendment, would the proposed amendment result in an increased density within a Transit Priority Area (TPA)[4] and implement CAP Strategy 3 actions, as determined in Step 3 to the satisfaction of the Development Services Department?; OR, C. If the proposed project is not consistent with the existing land use plan and zoning designations, does the project include a land use plan and/or zoning designation amendment that would result in an equivalent or less GHG-intensive project when compared to the existing designations?	☑	☐
If "***Yes***," proceed to Step 2 of the Checklist. For question B above, complete Step 3. For question C above, provide estimated project emissions under both existing and proposed designation(s) for comparison. Compare the maximum buildout of the existing designation and the maximum buildout of the proposed designation. If "***No***," in accordance with the City's Significance Determination Thresholds, the project's GHG impact is significant. The project must nonetheless incorporate each of the measures identified in Step 2 to mitigate cumulative GHG emissions impacts unless the decision maker finds that a measure is infeasible in accordance with CEQA Guidelines Section 15091. Proceed and complete Step 2 of the Checklist.		

The project proposed is consistent with the General Plan which identifies the site for a self-storage facility; the project is consistent with the Peninsula Community Plan which designates the site for a self-storage facility. The project is also consistent with the requirements of the Barrio-Logan subdistrict B zone.

[3] This question may also be answered in the affirmative if the project is consistent with SANDAG Series 12 growth projections, which were used to determine the CAP projections, as determined by the Planning Department.

[4] This category applies to all projects that answered in the affirmative to question 3 on the previous page: Is the project or a portion of the project located in a transit priority area.

Step 2: CAP Strategies Consistency

The second step of the CAP consistency review is to review and evaluate a project's consistency with the applicable strategies and actions of the CAP. Step 2 only applies to development projects that involve permits that would require a certificate of occupancy from the Building Official or projects comprised of one and two family dwellings or townhouses as defined in the California Residential Code and their accessory structures.[5] All other development projects that would not require a certificate of occupancy from the Building Official shall implement Best Management Practices for construction activities as set forth in the Greenbook (for public projects).

Step 2: CAP Strategies Consistency			
Checklist Item (Check the appropriate box and provide explanation for your answer)	Yes	No	N/A
Strategy 1: Energy & Water Efficient Buildings			
1. *Cool/Green Roofs*. • Would the project include roofing materials with a minimum 3-year aged solar reflection and thermal emittance or solar reflection index equal to or greater than the values specified in the voluntary measures under California Green Building Standards Code (Attachment A)?; OR • Would the project roof construction have a thermal mass over the roof membrane, including areas of vegetated (green) roofs, weighing at least 25 pounds per square foot as specified in the voluntary measures under California Green Building Standards Code?; OR • Would the project include a combination of the above two options? Check "N/A" only if the project does not include a roof component. The standing seam roof panel color will have a min. SRI of 75 in compliance with the California Green Building Code	☑	☐	☐

[5] Actions that are not subject to Step 2 would include, for example: 1) discretionary map actions that do not propose specific development, 2) permits allowing wireless communication facilities, 3) special events permits, 4) use permits or other permits that do not result in the expansion or enlargement of a building (e.g., decks, garages, etc.), and 5) non-building infrastructure projects such as roads and pipelines. Because such actions would not result in new occupancy buildings from which GHG emissions reductions could be achieved, the items contained in Step 2 would not be applicable.

2. *Plumbing fixtures and fittings* With respect to plumbing fixtures or fittings provided as part of the project, would those low-flow fixtures/appliances be consistent with each of the following: Residential buildings: • Kitchen faucets: maximum flow rate not to exceed 1.5 gallons per minute at 60 psi; • Standard dishwashers: 4.25 gallons per cycle; • Compact dishwashers: 3.5 gallons per cycle; and • Clothes washers: water factor of 6 gallons per cubic feet of drum capacity? Nonresidential buildings: • Plumbing fixtures and fittings that do not exceed the maximum flow rate specified in Table A5.303.2.3.1 (voluntary measures) of the California Green Building Standards Code (See Attachment A); and • Appliances and fixtures for commercial applications that meet the provisions of Section A5.303.3 (voluntary measures) of the California Green Building Standards Code (See Attachment A)? Check "N/A" only if the project does not include any plumbing fixtures or fittings. Water closets and lavatories will be provided in restrooms to be under the maximum flow rates listed on Attachment A	☑	☐	☐

Strategy 3: Bicycling, Walking, Transit & Land Use			
3. *Electric Vehicle Charging* • Multiple-family projects of 17 dwelling units or less: Would 3% of the total parking spaces required, or a minimum of one space, whichever is greater, be provided with a listed cabinet, box or enclosure connected to a conduit linking the parking spaces with the electrical service, in a manner approved by the building and safety official, to allow for the future installation of electric vehicle supply equipment to provide electric vehicle charging stations at such time as it is needed for use by residents? • Multiple-family projects of more than 17 dwelling units: Of the total required listed cabinets, boxes or enclosures, would 50% have the necessary electric vehicle supply equipment installed to provide active electric vehicle charging stations ready for use by residents? • Non-residential projects: Of the total required listed cabinets, boxes or enclosures, would 50% have the necessary electric vehicle supply equipment installed to provide active electric vehicle charging stations ready for use? Check "N/A" only if the project is a single-family project or would not require the provision of listed cabinets, boxes, or enclosures connected to a conduit linking the parking spaces with electrical service, e.g., projects requiring fewer than 10 parking spaces. Code requires one designated parking spaces be provided for electric vehicles. This space will be adjacent to elevators with a charging station mounted to elevator enclosure wall alongside parking space.	☑	☐	☐
Strategy 3: Bicycling, Walking, Transit & Land Use (Complete this section if project includes non-residential or mixed uses)			
4. *Bicycle Parking Spaces* Would the project provide more short- and long-term bicycle parking spaces than required in the City's Municipal Code (Chapter 14, Article 2, Division 5)?[6] Check "N/A" only if the project is a residential project. Total number of short-term bicycle parking will be 3, the 2 required by code and additional 1. There will not be greater than 10 employees, therefor per the municipal code, the minimum 1 long-term bicycle parking is required and one additional per CAP to be provided. A permanent rack housing 2 bikes will be mounted in the parking garage level near the office entrances.	☑	☐	☐

[6] Non-portable bicycle corrals within 600 feet of project frontage can be counted towards the project's bicycle parking requirements.

5. *Shower facilities*

If the project includes nonresidential development that would accommodate over 10 tenant occupants (employees), would the project include changing/shower facilities in accordance with the voluntary measures under the California Green Building Standards Code as shown in the table below?

Number of Tenant Occupants (Employees)	Shower/Changing Facilities Required	Two-Tier (12" X 15" X 72") Personal Effects Lockers Required
0-10	0	0
11-50	1 shower stall	2
51-100	1 shower stall	3
101-200	1 shower stall	4
Over 200	1 shower stall plus 1 additional shower stall for each 200 additional tenant-occupants	1 two-tier locker plus 1 two-tier locker for each 50 additional tenant-occupants

Check "N/A" only if the project is a residential project, or if it does not include nonresidential development that would accommodate over 10 tenant occupants (employees).

Total number of employee occupants will be 10

☐ ☐ ☑

6. *Designated Parking Spaces*

If the project includes a nonresidential use in a TPA, would the project provide designated parking for a combination of low-emitting, fuel-efficient, and carpool/vanpool vehicles in accordance with the following table?

Number of Required Parking Spaces	Number of Designated Parking Spaces
0-9	0
10-25	2
26-50	4
51-75	6
76-100	9
101-150	11
151-200	18
201 and over	At least 10% of total

This measure does not cover electric vehicles. See Question 4 for electric vehicle parking requirements.

Note: Vehicles bearing Clean Air Vehicle stickers from expired HOV lane programs may be considered eligible for designated parking spaces. The required designated parking spaces are to be provided within the overall minimum parking requirement, not in addition to it.

Check "N/A" only if the project is a residential project, or if it does not include nonresidential use in a TPA.

2 of the 19 spaces provided will be for above mentioned vehicles only

☑ ☐ ☐

7. *Transportation Demand Management Program* If the project would accommodate over 50 tenant-occupants (employees), would it include a transportation demand management program that would be applicable to existing tenants and future tenants that includes: At least one of the following components: • Parking cash out program • Parking management plan that includes charging employees market-rate for single-occupancy vehicle parking and providing reserved, discounted, or free spaces for registered carpools or vanpools • Unbundled parking whereby parking spaces would be leased or sold separately from the rental or purchase fees for the development for the life of the development And at least three of the following components: • Commitment to maintaining an employer network in the SANDAG iCommute program and promoting its RideMatcher service to tenants/employees • On-site carsharing vehicle(s) or bikesharing • Flexible or alternative work hours • Telework program • Transit, carpool, and vanpool subsidies • Pre-tax deduction for transit or vanpool fares and bicycle commute costs • Access to services that reduce the need to drive, such as cafes, commercial stores, banks, post offices, restaurants, gyms, or childcare, either onsite or within 1,320 feet (1/4 mile) of the structure/use? Check "N/A" only if the project is a residential project or if it would not accommodate over 50 tenant-occupants (employees). Project will not accommodate over 50 employees.	☐	☐	☑

Step 3: Project CAP Conformance Evaluation (if applicable)

The third step of the CAP consistency review only applies if Step 1 is answered in the affirmative under option B. The purpose of this step is to determine whether a project that is located in a TPA but that includes a land use plan and/or zoning designation amendment is nevertheless consistent with the assumptions in the CAP because it would implement CAP Strategy 3 actions. In general, a project that would result in a reduction in density inside a TPA would not be consistent with Strategy 3.The following questions must each be answered in the affirmative and fully explained.

1. **Would the proposed project implement the General Plan's City of Villages strategy in an identified Transit Priority Area (TPA) that will result in an increase in the capacity for transit-supportive residential and/or employment densities?**

   Considerations for this question:
   - Does the proposed land use and zoning designation associated with the project provide capacity for transit-supportive residential densities within the TPA?
   - Is the project site suitable to accommodate mixed-use village development, as defined in the General Plan, within the TPA?
   - Does the land use and zoning associated with the project increase the capacity for transit-supportive employment intensities within the TPA?

2. **Would the proposed project implement the General Plan's Mobility Element in Transit Priority Areas to increase the use of transit?**

   Considerations for this question:
   - Does the proposed project support/incorporate identified transit routes and stops/stations?
   - Does the project include transit priority measures?

3. **Would the proposed project implement pedestrian improvements in Transit Priority Areas to increase walking opportunities?**

   Considerations for this question:
   - Does the proposed project circulation system provide multiple and direct pedestrian connections and accessibility to local activity centers (such as transit stations, schools, shopping centers, and libraries)?
   - Does the proposed project urban design include features for walkability to promote a transit supportive environment?

4. **Would the proposed project implement the City of San Diego's Bicycle Master Plan to increase bicycling opportunities?**

   Considerations for this question:
   - Does the proposed project circulation system include bicycle improvements consistent with the Bicycle Master Plan?
   - Does the overall project circulation system provide a balanced, multimodal, "complete streets" approach to accommodate mobility needs of all users?

5. **Would the proposed project incorporate implementation mechanisms that support Transit Oriented Development?**

   Considerations for this question:
   - Does the proposed project include new or expanded urban public spaces such as plazas, pocket parks, or urban greens in the TPA?
   - Does the land use and zoning associated with the proposed project increase the potential for jobs within the TPA?
   - Do the zoning/implementing regulations associated with the proposed project support the efficient use of parking through mechanisms such as: shared parking, parking districts, unbundled parking, reduced parking, paid or time-limited parking, etc.?

6. **Would the proposed project implement the Urban Forest Management Plan to increase urban tree canopy coverage?**

   Considerations for this question:
   - Does the proposed project provide at least three different species for the primary, secondary and accent trees in order to accommodate varying parkway widths?
   - Does the proposed project include policies or strategies for preserving existing trees?
   - Does the proposed project incorporate tree planting that will contribute to the City's 20% urban canopy tree coverage goal?

CLIMATE ACTION PLAN CONSISTENCY CHECKLIST
ATTACHMENT A

This attachment provides performance standards for applicable Climate Action Pan (CAP) Consistency Checklist measures.

Table 1 **Roof Design Values for Question 1: Cool/Green Roofs supporting Strategy 1: Energy & Water Efficient Buildings of the Climate Action Plan**

Land Use Type	Roof Slope	Minimum 3-Year Aged Solar Reflectance	Thermal Emittance	Solar Reflective Index
Low-Rise Residential	≤ 2:12	0.55	0.75	64
	> 2:12	0.20	0.75	16
High-Rise Residential Buildings, Hotels and Motels	≤ 2:12	0.55	0.75	64
	> 2:12	0.20	0.75	16
Non-Residential	≤ 2:12	0.55	0.75	64
	> 2:12	0.20	0.75	16

Source: Adapted from the California Green Building Standards Code (CALGreen) Tier 1 residential and non-residential voluntary measures shown in Tables A4.106.5.1 and A5.106.11.2.2, respectively. Roof installation and verification shall occur in accordance with the CALGreen Code.

CALGreen does not include recommended values for low-rise residential buildings with roof slopes of ≤ 2:12 for San Diego's climate zones (7 and 10). Therefore, the values for climate zone 15 that covers Imperial County are adapted here.

Solar Reflectance Index (SRI) equal to or greater than the values specified in this table may be used as an alternative to compliance with the aged solar reflectance values and thermal emittance.

Table 2	Fixture Flow Rates for Non-Residential Buildings related to Question 2: Plumbing Fixtures and Fittings supporting Strategy 1: Energy & Water Efficient Buildings of the Climate Action Plan
Fixture Type	**Maximum Flow Rate**
Showerheads	1.8 gpm @ 80 psi
Lavatory Faucets	0.35 gpm @60 psi
Kitchen Faucets	1.6 gpm @ 60 psi
Wash Fountains	1.6 [rim space(in.)/20 gpm @ 60 psi]
Metering Faucets	0.18 gallons/cycle
Metering Faucets for Wash Fountains	0.18 [rim space(in.)/20 gpm @ 60 psi]
Gravity Tank-type Water Closets	1.12 gallons/flush
Flushometer Tank Water Closets	1.12 gallons/flush
Flushometer Valve Water Closets	1.12 gallons/flush
Electromechanical Hydraulic Water Closets	1.12 gallons/flush
Urinals	0.5 gallons/flush

Source: Adapted from the California Green Building Standards Code (CALGreen) Tier 1 non-residential voluntary measures shown in Tables A5.303.2.3.1 and A5.106.11.2.2, respectively. See the California Plumbing Code for definitions of each fixture type.

Where complying faucets are unavailable, aerators rated at 0.35 gpm or other means may be used to achieve reduction.

Acronyms:
gpm = gallons per minute
psi = pounds per square inch (unit of pressure)
in. = inch

Table 3	Standards for Appliances and Fixtures for Commercial Application related to Question 2: Plumbing Fixtures and Fittings supporting Strategy 1: Energy & Water Efficient Buildings of the Climate Action Plan	
Appliance/Fixture Type	**Standard**	
Clothes Washers	Maximum Water Factor (WF) that will reduce the use of water by 10 percent below the California Energy Commissions' WF standards for commercial clothes washers located in Title 20 of the *California Code of Regulations.*	
Conveyor-type Dishwashers	0.70 maximum gallons per rack (2.6 L) (High-Temperature)	0.62 maximum gallons per rack (4.4 L) (Chemical)
Door-type Dishwashers	0.95 maximum gallons per rack (3.6 L) (High-Temperature)	1.16 maximum gallons per rack (2.6 L) (Chemical)
Undercounter-type Dishwashers	0.90 maximum gallons per rack (3.4 L) (High-Temperature)	0.98 maximum gallons per rack (3.7 L) (Chemical)
Combination Ovens	Consume no more than 10 gallons per hour (38 L/h) in the full operational mode.	
Commercial Pre-rinse Spray Valves (manufactured on or after January 1, 2006)	Function at equal to or less than 1.6 gallons per minute (0.10 L/s) at 60 psi (414 kPa) and • Be capable of cleaning 60 plates in an average time of not more than 30 seconds per plate. • Be equipped with an integral automatic shutoff. • Operate at static pressure of at least 30 psi (207 kPa) when designed for a flow rate of 1.3 gallons per minute (0.08 L/s) or less.	

Source: Adapted from the California Green Building Standards Code (CALGreen) Tier 1 non-residential voluntary measures shown in Section A5.303.3. See the California Plumbing Code for definitions of each appliance/fixture type.

Acronyms:
L = liter
L/h = liters per hour
L/s = liters per second
psi = pounds per square inch (unit of pressure)
kPa = kilopascal (unit of pressure)

U-STOR-IT (Barrio Logan) LLC
Preliminary Drainage Study

2209 National Avenue,
San Diego, CA 92113

Date Prepared:
July 23rd, 2018

Prepared for:
U-STOR-IT Barrio Logan LLC

Prepared By:
Omega Engineering Consultants
4340 Viewridge Ave, Suite B
San Diego, CA 92123
Ph: (858) 634-8620

Declaration of Responsible Charge:
I hereby declare that I am the engineer of work for this project, that I have exercised responsible charge over the design of the project as defined in section 6703 of the business and professions code, and that the design is consistent with current standards. I understand that the check of the project drawings and specifications by the City of San Diego is confined to a review only and does not relieve me, as an engineer of work, of my responsibilities for project design.

Patric T. de Boer RCE 83583
Registration Expires 3-31-2019

REGISTERED PROFESSIONAL ENGINEER
PATRIC T. DE BOER
No. C 83583
CIVIL
STATE OF CALIFORNIA

Table of Contents

Appendices

Site & Project Description

This Hydrology and Hydraulics report has been prepared as part of the redevelopment plan for the site at 2209 National Avenue into a self-storage facility. The project will remove the existing structure and paving on the site, and construct a self-storage structure. The site discharges to the public storm drain system at two locations to an adjacent alley which carries the storm water to Sampson Street or 26th Avenue. Then storm water flows via curb and gutter to the public storm drain system and then, directly to the San Diego Bay. See Figure 2 for the existing drainage limits. See Figure 3 for the proposed drainage limits.

Methodology

This drainage report has been prepared in accordance with current City of San Diego regulations and procedures, with the exception of the drainage basin weighted C values. These were calculated according to The County of San Diego Hydrology Manual. All of the proposed conduits and conveyances have been designed to intercept and convey the 100-year storm. The Modified Rational Method was used to compute the anticipated runoff. See the attached calculations for particulars. The following references have been used in preparation of this report:
(1) Handbook of Hydraulics, E.F. Brater & H.W. King, 6th Ed., 1976.
(2) Modern Sewer Design, American Iron & Steel Institute, 1st Ed., 1980.
(3) County of San Diego Hydrology Manual, 2003

Culvert Design and Analysis
The storm drain culverts were sized using the K' values from King's Handbook Appendix 7-14, (Appendix 7.0 of this report). The following formula was used:

Q= (K'/n)*d^(8/3)*s^(0.5)
K'= Discharge Factor
d= Diameter of Conduit (ft)
n= Manning's Coefficient
Q= Runoff Discharge (cfs)
s= Pipe Slope (ft/ft)

Rational Method
Q=CIA
Where:
Q= peak discharge, in cubic feet per second (cfs)
C= runoff coefficient, proportion of the rainfall that runs off the surface (no units)
= 0.90*(% impervious)+Cp*(1-% Impervious)) page 5, County Hydrology Manual
I = average rainfall intensity for a duration equal to the Tc for the area, (in/hr)
= $7.44*P6*Tc^{-0.645}$
A = drainage area contributing to the design location, in acres
Cp= Pervious Coefficient Runoff Value, County of San Diego Hydrology Manual minimum of 0.35
$Tc= 1.8*(1.1-C)*(Tc)^{0.5}*S^{0.33}$
Where:
S= Slope of drainage course*

Existing Conditions

The site location consists of an existing bank building located at the northerly corner of the site, and the banks associated parking and driveways. Water is conveyed at 4%-5% slopes via surface run-off to the adjacent alley, and discharges at points 1 and 2 entering the public storm drain via curb and gutter at 26th Street and Sampson Street. The public storm drain discharges at the San Diego Bay.

Proposed Conditions

The project proposes to demolish and remove the existing structure and hardscape, and construct a self-storage facility. The proposed improvements include the storage facility building, and a driveway. Two biofiltration basins will be constructed alongside the south westerly frontage of the site. The biofiltration basins will drain to the adjacent alley, at discharge points 1 and 2, before entering the public storm drain via a curb inlet at 26th street and Sampson Street. See the Storm Water Quality Management Plan (SWQMP) for details.

Existing Runoff Analysis

The existing site was modeled as two sub-basins, EX-1 & EX-2. Basin EX-1 contains the majority of the parking lot on the site, all of the site landscaping, and discharge point 1 located at the southeasterly corner of the site. EX-2 contains the entire bank building, some impervious surfaces, and discharge point 2 which is located at the southwesterly corner of the site. See Figure 2 for more information. As the existing surface conditions varied for each sub-basin, run-off coefficients were found using a weighted average with soils having a run-off coefficient of 0.35, and drive pavement/roofs having a run-off coefficient of 0.9. Runoff flow rates were determined using the rational method, which is summarized in the Methodology section of this report.

Below is a summary of the basin input data and resulting Q's:

Basin #	Area (ac)	C	Slope (%)	Q_{100} (cfs)
EX-1	0.56	0.79	4.0	2.82
EX-2	0.24	0.90	5.0	1.38

See the attached calculations for details.

Proposed Runoff Analysis

The proposed site was modeled as two sub-basins, A-1 and A-2. Basin A-1contains the majority of the building, a biofiltration basin, and discharge poin1 located at the southeasterly corner of the site. Basin A-2 contains the remaining portion of the building, a separate biofiltration basin, and another biofiltration basin located at the southwesterly corner of the site. See Figure 3 for details. As the proposed surface conditions varied for each sub-basin, run-off coefficients were found using a weighted average with soils having a run-off coefficient of 0.35, and drive pavement/roofs having a run-off coefficient of 0.9. Runoff flow rates were determined using the rational method, which is summarized in the Methodology section of this report.

Below is a summary of the basin input data and resulting Q's:

Basin #	Area (ac)	C	Slope (%)	Q_{100} (cfs)
A-1	0.56	0.88	1.5%	2.80
A-2	0.24	0.88	1.5%	1.35

See the attached calculations for details.

Results and Conclusions

The redevelopment of the site shall result in a decrease of 0.02 CFS for the 100 year storm event for discharge point 1, and a decrease of 0.03 CFS for discharge point 2.

It is the opinion of Omega Engineering Consultants that the project will not cause adverse effects to the downstream facilities or receiving waters. A separate Storm Water Quality Management Plan (SWQMP) has been prepared to discuss the water quality impacts for the proposed development.

2209 National Avenue San Dieg CA, 92113 10/10/2017

HYDROLOGY AND HYDRAULICS CALCS (Table No. 1)

BASIN	AREA (SF)	AREA (AC)	% Imp	"C" Value
EX-1	24,581	0.56	80%	0.79
EX-2	10,550	0.24	100%	0.90
PROP TOTAL	**35,131**	**0.81**		
A-1*	24,581	0.56	97%	0.88
A-2	10,550	0.24	97%	0.88
PROP TOTAL	**35,131**	**0.81**		

Basin Confluence	Symbol
-	-
-	-
-	-
-	-

(A) "CP#1" Confluence Point Number 1

(B) C value for bare ground is 0.35 (Table 3-1 County Hydrology Manual)
C value for impervious surfaces is 0.9
Basins with mixed surface type use a weighted average
of these 2 values. (impervious % x 0.9)+(pervious % x 0.35)

2209 National Avenue San Dieg CA, 92113
HYDROLOGY AND HYDRAULICS CALCS (Table No. 2)

10/10/2017

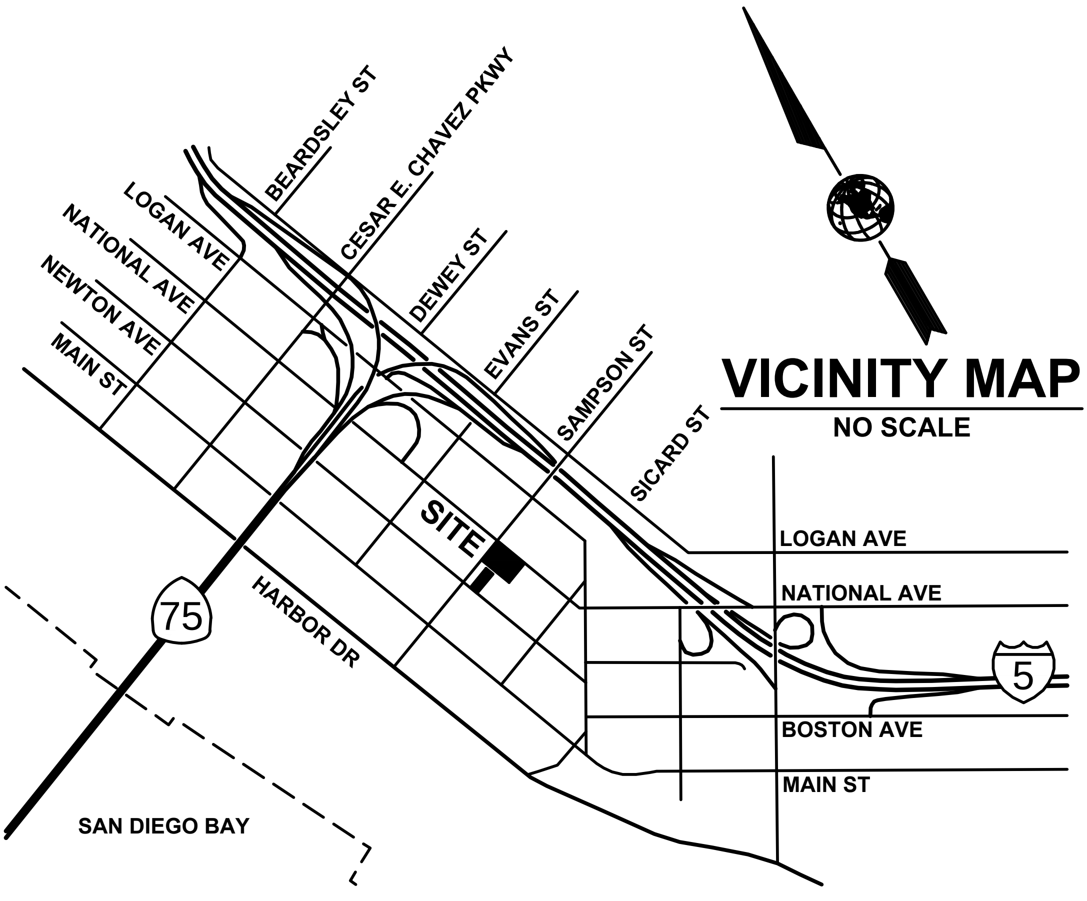
Sub-Basin	AREA Ac.	"C"	CA	L (ft) Travel	H (ft) (elev)	S(%) (avg.)	Tc min.	T tot mins	I in/hr	Q cfs	Q tot cfs		NOTES 85th % storm
EX-1	0.56	0.79	0.45	141.00	5.60	4.0	5.00	5.00	0.20	0.09	0.09		
								5.00	**0.20**		**0.09**		
						Existing Discharge Pt. 1 =					**0.09**	CFS	
EX-2	0.24	0.90	0.22	141.00	7.00	5.0	5.00	5.00	0.20	0.04	0.04		
								5.00	**0.20**		**0.04**		
						Existing Discharge Pt. 2 =					**0.04**	CFS	
A-1	0.56	0.88	0.50	312.00	4.68	1.50	6.01	6.01	0.20	0.10	0.10		
								6.01	**0.20**		**0.10**		
						Prposed Discharge Pt. 1 =					**0.10**	CFS	
A-2	0.24	0.88	0.21	212.00	3.18	1.50	5.00	5.00	0.20	0.04	0.04		
								5.00	**0.20**		**0.04**		
						Prposed Discharge Pt. 2 =					**0.04**	CFS	

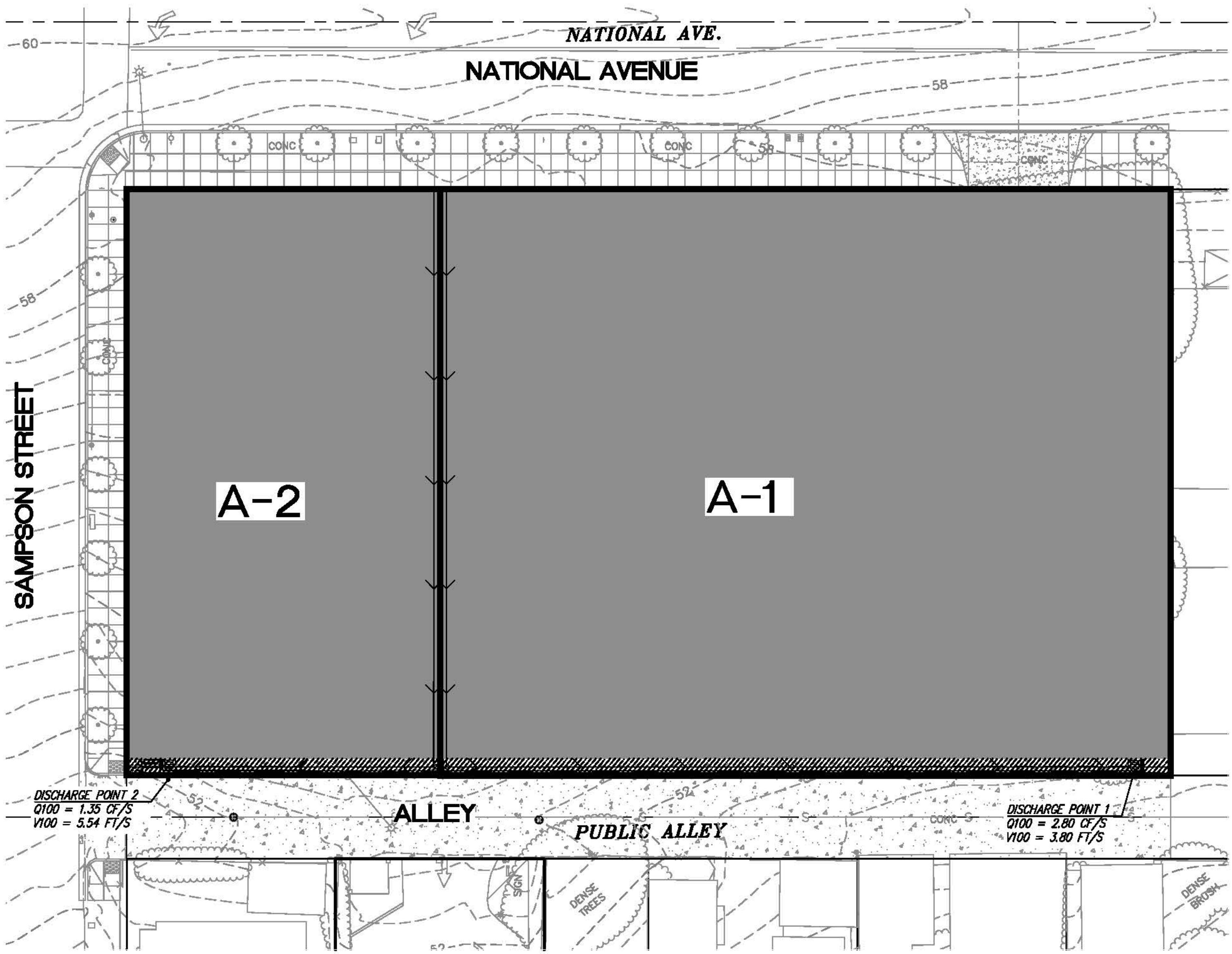
2209 National Avenue San Dieg CA, 92113
HYDROLOGY AND HYDRAULICS CALCS (Table No. 3)

10/10/2017

Sub-Basin	AREA Ac.	"C"	CA	L (ft) Travel	H (ft) (elev)	S(%) (avg.)	Tc min.	T tot mins	I in/hr	Q cfs	Q tot cfs		NOTES 100 year storm
													P(6)= 2.40
EX-1	0.56	0.79	0.45	141.00	5.60	4.0	5.00	5.00	6.32	2.82	2.82		
											2.82		
							Existing Discharge Pt. 1 =				**2.82**	**CFS**	
EX-2	0.24	0.90	0.22	141.00	7.00	5.0	5.00	5.00	6.32	1.38	1.38		
											1.38		
							Existing Discharge Pt. 2 =				**1.38**	**CFS**	
A-1	0.56	0.88	0.50	312.00	4.68	1.50	6.01	6.01	5.61	2.80	2.80		
											2.80		
							Proposed Discharge Pt. 1=				**2.80**	**CFS**	
A-2	0.24	0.88	0.21	212.00	3.18	1.50	5.00	5.00	6.32	1.35	1.35		
											1.35		
							Proposed Discharge Pt. 2=				**1.35**	**CFS**	

FIGURE 1


VICINITY MAP
NO SCALE
BEARDSLEY ST
CESAR E. CHAVEZ PKWY
LOGAN AVE
NATIONAL AVE
NEWTON AVE
MAIN ST
DEWEY ST
EVANS ST
SAMPSON ST
SICARD ST
SITE
LOGAN AVE
NATIONAL AVE
BOSTON AVE
MAIN ST
HARBOR DR
75
5
SAN DIEGO BAY



LEGEND:

- AREA LIMITS
- FLOW PATH ARROW INDICATES DIRECTION
- BASIN NUMBER — EX-#
- BUILDING AREA
- BIOFILTRATION AREA

BASIN DATA					
BASIN	AREA	IMPERVIOUS	IMPERVIOUS %	Q_{100}(CFS)	V_{100}(FT/S)
A-1	24,581 SF	23,881 SF	97%	2.80	3.50
A-2	10,550 SF	10,250 SF	97%	1.35	3.80

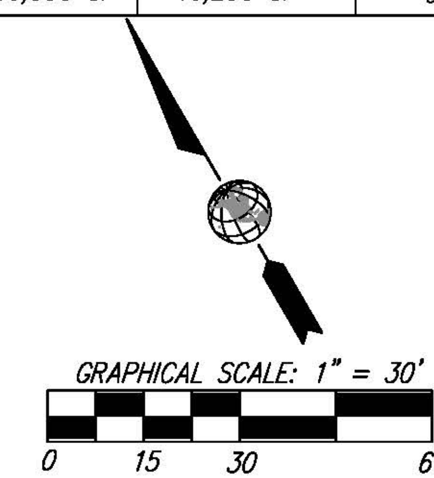
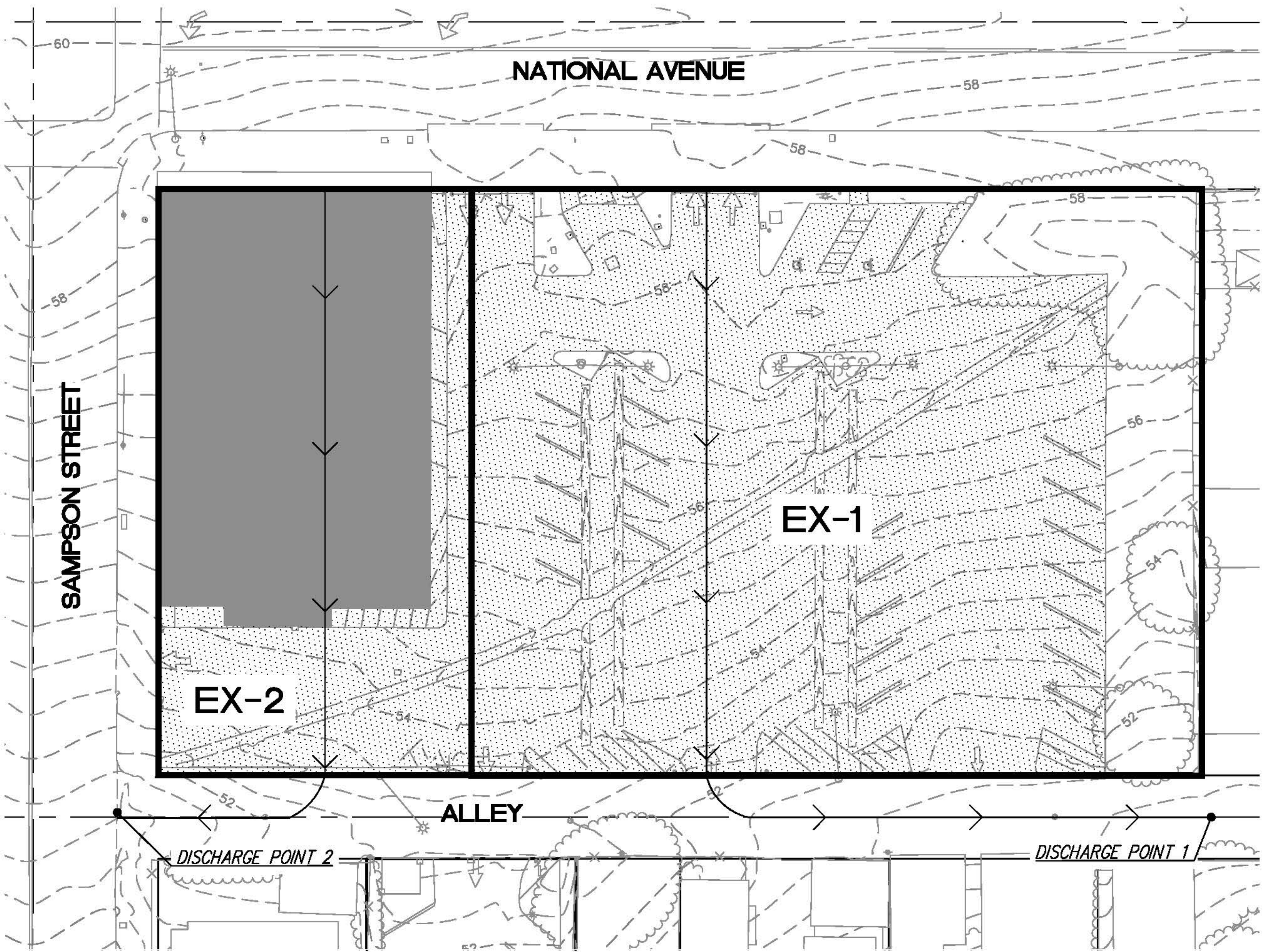


FIGURE 3
NATIONAL AVE SELF STORAGE
PROPOSED HYDROLOGY

OMEGA

OMEGA ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA 92123
PH:(858) 634-8620 FAX:(858) 634-8627



LEGEND:

AREA LIMITS

FLOW PATH ARROW INDICATES DIRECTION

BASIN NUMBER .... EX-#

BUILDING AREA

PAVEMENT AREA

LANDSCAPE AREA

BASIN DATA					
BASIN	AREA	IMPERVIOUS	IMPERVIOUS %	Q_{100}(CFS)	V_{100}(FT/S)
EX-1	24,581 SF	19,770 SF	80 %	2.82	3.52
EX-2	10,550 SF	10,550 SF	100 %	1.38	3.82

GRAPHICAL SCALE: 1" = 30'

0 15 30 60

FIGURE 2
NATIONAL AVE SELF STORAGE
EXISTING HYDROLOGY

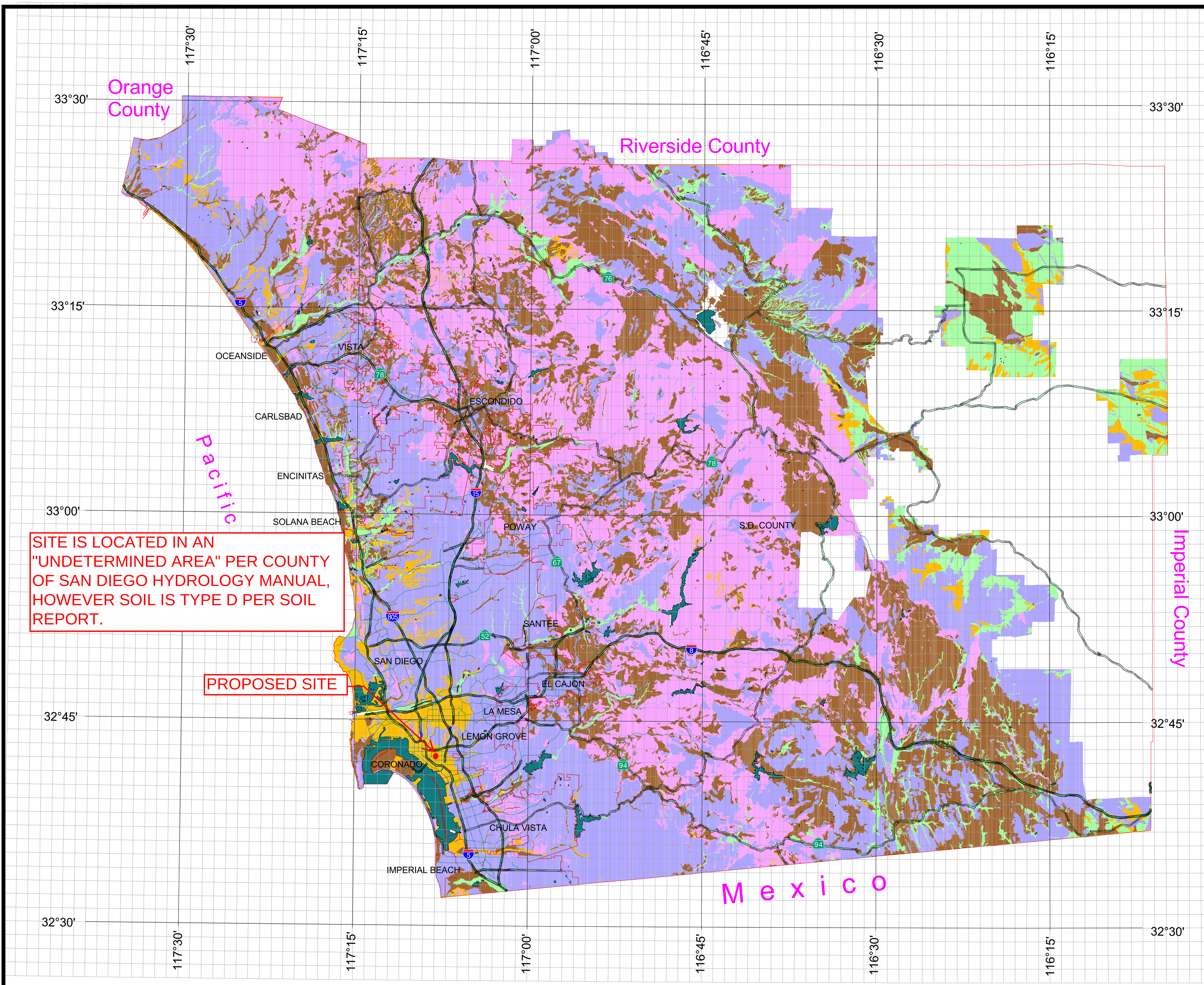
OMEGA

OMEGA ENGINEERING CONSULTANTS

4340 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA 92123
PH:(858) 634-8620 FAX:(858) 634-8627

Appendices

Appendix 1



County of San Diego Hydrology Manual

Soil Hydrologic Groups

Legend

Soil Groups

- Group A
- Group B
- Group C
- Group D
- Undetermined
- Data Unavailable

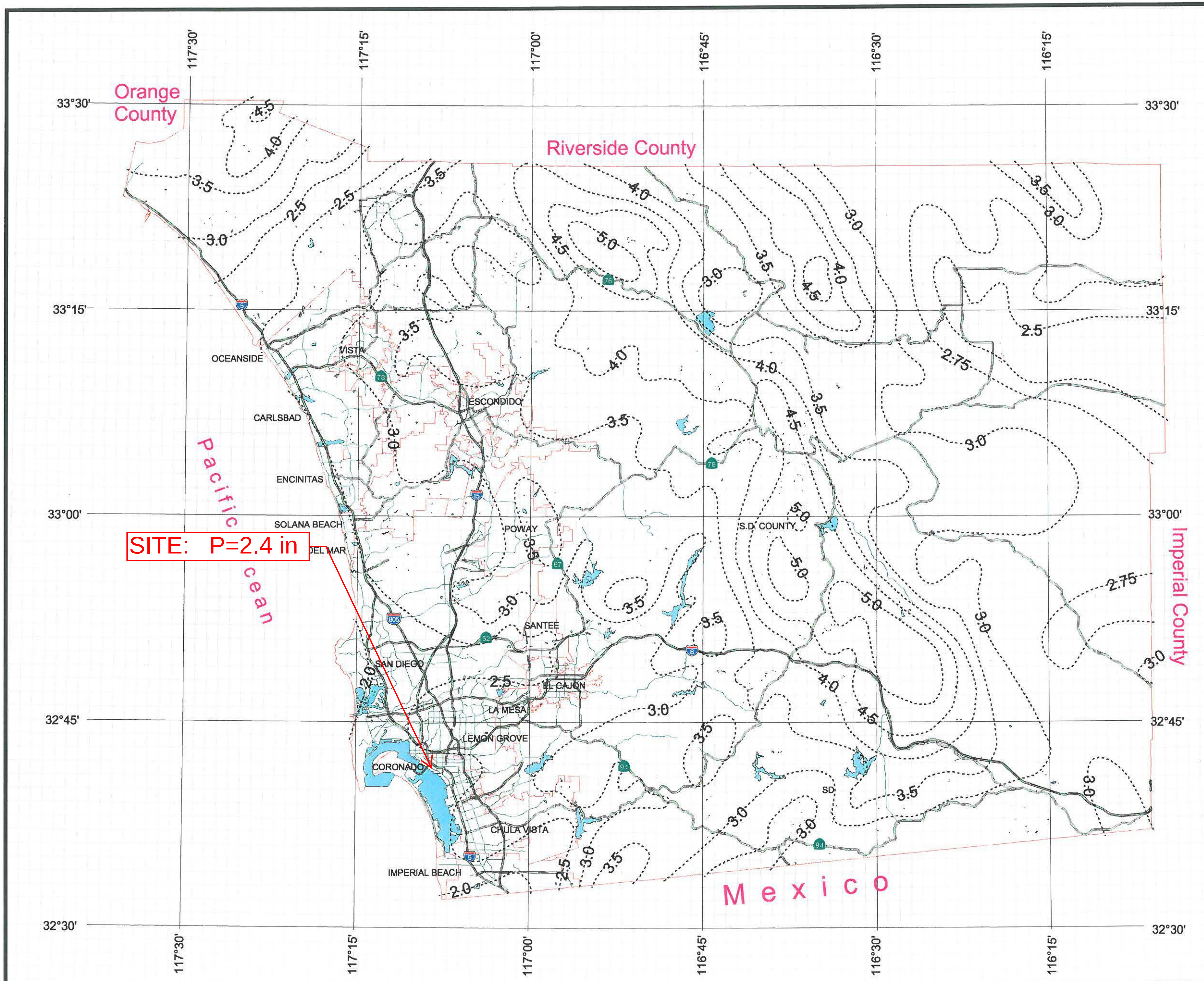
DPW GIS
Department of Public Works
Geographic Information Services

SanGIS
We Have San Diego Covered!

N S E W

3 0 3 Miles

Appendix 2



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours

Isopluvial (inches)

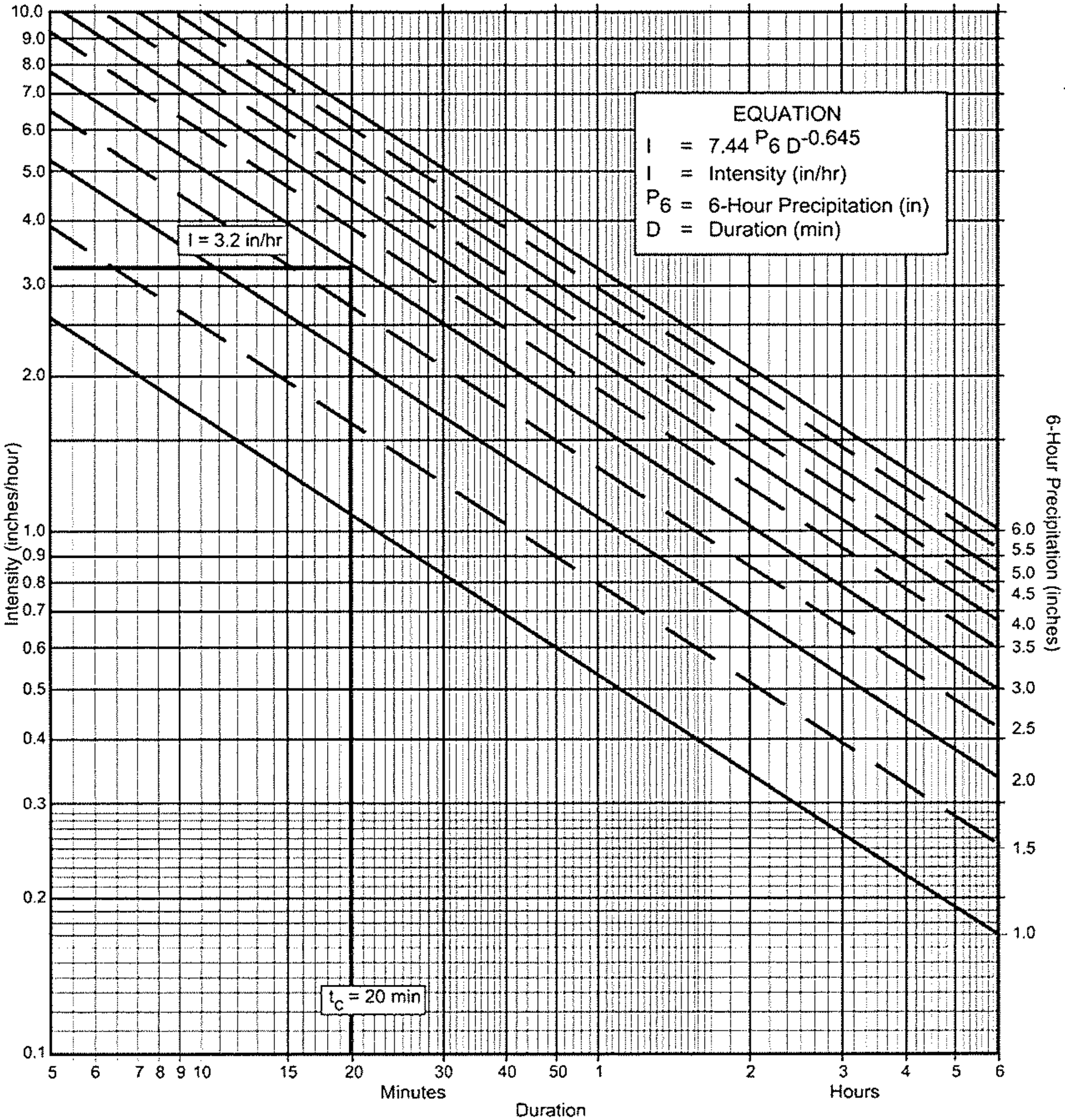
DPW GIS
Department of Public Works
Geographic Information Services

SanGIS
We Have San Diego Covered!

N
W E
S

3 0 3 Miles

Appendix 3



Directions for Application:

(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicaple to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application For

(a) Selected frequency ______ year

(b) P_6 = ______ in., P_{24} = ______, $\frac{P_6}{P_{24}}$ = ______ %(2)

(c) Adjusted $P_6^{(2)}$ = ______ in.

(d) t_x = ______ min.

(e) I = ______ in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

P6 / Duration	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

Intensity-Duration Design Chart - Example

Appendix 4

Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS

Land Use		Runoff Coefficient "C"				
			Soil Type			
NRCS Elements	County Elements	% IMPER.	A	B	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.30	0.35
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10	0.27	0.32	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.80	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).

DU/A = dwelling units per acre

NRCS = National Resources Conservation Service

Appendix 5

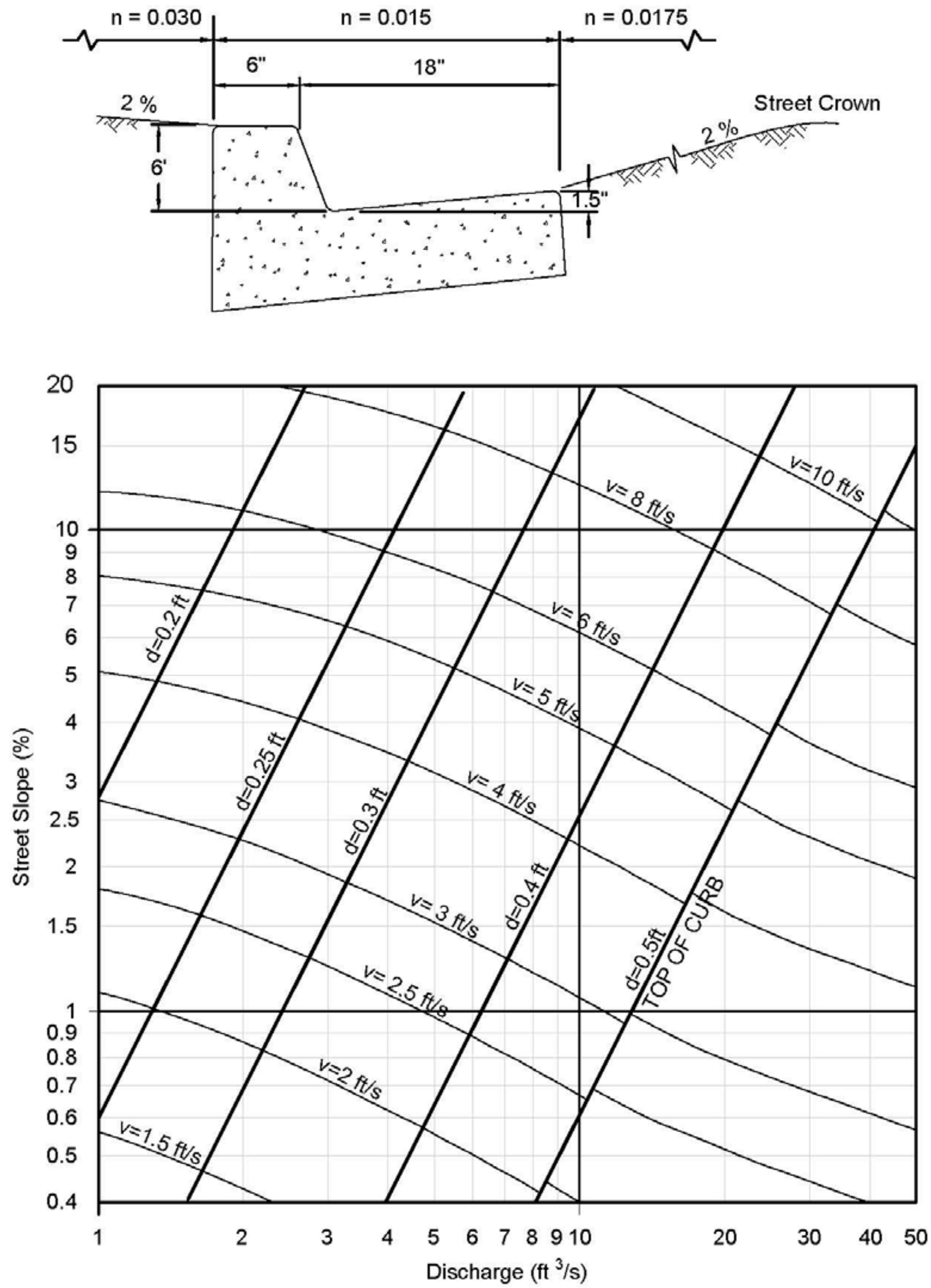


Figure 3-2: Gutter and Roadway Discharge-Velocity Chart (6" Curb)

GEOTECHNICAL AND FAULT INVESTIGATION

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

U STOR IT (BARRIO LOGAN)
SAN DIEGO, CALIFORNIA

DECEMBER 5, 2017
PROJECT NO. G2093-52-01

Project No. G2093-52-01
December 5, 2017

U STOR IT (Barrio Logan)
402 West Broadway, Suite 810
San Diego, California 92101

Attention: Mr. Lawrence Nora

Subject: GEOTECHNICAL AND FAULT INVESTIGATION
2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

Dear Mr. Nora:

In accordance with your request and our Proposal No. LG-17040, dated February 3, 2017, we herein submit the results of our geotechnical and fault rupture hazard investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards to assist in the design of the proposed building and improvements. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The site is considered suitable for the proposed building and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project.

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

Very truly yours,

GEOCON INCORPORATED

Matthew R. Love
RCE 84154

Shawn Foy Weedon
GE 2714

Rupert S. Adams
CEG 2561

MRL:SFW:RSA:ejc

(e-mail) Addressee

TABLE OF CONTENTS

TABLE OF CONTENTS (Concluded)

GEOTECHNICAL AND FAULT INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical and fault investigation for the proposed new self-storage facility in the Barrio Logan area of San Diego, California as shown on the Vicinity Map, Figure 1. The purpose of this geotechnical and fault investigation is to evaluate the surface and subsurface soil conditions, general site geology, and to identify geotechnical constraints that may impact the planned improvements to the property. In addition, this report provides 2016 CBC seismic design criteria; grading recommendations; shoring and tie-back recommendations; shallow foundation and concrete slab-on-grade recommendations; mat foundation recommendations; retaining wall and lateral load recommendations; and discussions regarding the local geologic hazards including faulting and seismic shaking.

This report is limited to the area proposed for the construction of the new development and associated improvements as shown on the Geologic Map, Figure 2. We used the Conceptual Grading Plan prepared by Omega Engineering (2017) as the base for the Geologic Map. Figure 3 present a geologic cross-section for the conditions encountered during our field investigation.

The scope of this investigation included reviewing readily available published and unpublished geologic literature, including available fault investigation reports for nearby sites (see List of References); performing engineering analyses; and preparing this geotechnical investigation report. We also drilled six geotechnical borings to a maximum depth of 50 feet (see Appendix A), excavated a fault trench across the site to a maximum depth of 9 feet (see Figure 4), performed four infiltration tests, sampled soil and performed laboratory testing. Appendix A presents the exploratory boring and trench logs. The results of the laboratory tests are presented in Appendix B and on the boring logs in Appendix A. Appendix C presents the results of the storm water management investigation.

Our geotechnical Borings B-3 and B-4 and associated infiltration tests P-1 and P-2 are located in the existing parking lot to the south of the property. This area was previously planned for additional site parking and potential stormwater management by the design team; however, this area is now not a part of the project. We have included the boring logs and infiltration test results from these borings in the report for informational purposes only. The locations of the offsite borings are shown on the Geologic Map, Figure 2.

2. SITE AND PROJECT DESCRIPTION

The property is located south of National Avenue and east of Sampson Street in the Barrio Logan area of San Diego, California. The rectangular property consists of a vacant commercial bank structure on the northwest corner of the property and the remainder consists of surface asphalt concrete parking. The northern bank property is relatively flat at an elevation of about 50 to 60 feet above Mean Sea Level (MSL).

We understand the planned development consists of a 3-story self-storage facility over 2 subterranean levels on the northern portion of the property. We expect the proposed structure would likely be supported on conventional shallow foundation systems founded in Old Paralic Deposits at a proposed pad elevation of 38 feet MSL. We understand that bio-filtration devices will be constructed on the southern portion of the property and will be lined to prevent infiltration into subgrade materials.

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

3. GEOLOGIC SETTING

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

The site is located on the western portion of the coastal plain. Marine sedimentary units make up the geologic sequence encountered on the site and consist of Pleistocene age Old Paralic Deposits Unit 6 (Qop_6; formerly known as the Bay Point Formation) underlain by Pliocene age San Diego Formation sediments. Old Paralic Deposits mapped as Unit 6 were deposited roughly 120k years ago and are synonymous with the Nestor Terrace. The Old Paralic Deposits represent deposition in a brackish water estuarine and near shore terrestrial environment (Kennedy, 1999), and consist of fine to coarse grained sand with varying amounts of silts, clays and gravel. The San Diego Formation located below the Old Paralic Deposits is in excess of 100 feet thick, but was not encountered during our investigation. The geologic conditions in the vicinity of the site are shown on the Regional Geologic Map, Figure 5.

The regional geology in the area is predominately controlled by the active Rose Canyon Fault Zone (RCFZ) which transitions from a strike slip fault to the north of the site to several faults that have oblique movements of both strike slip and normal faulting to the west and east. The San Diego Bay was created as a down dropped block within this fault zone. The zone extends to the south and branches into three segments, Spanish Bight, Coronado, and Silver Strand Faults. There are two active fault zones in downtown area of San Diego that have been included in state-designated Alquist-Priolo Earthquake Fault Zones: 1) near First Street and in the vicinity of 15th and 16th Streets and 2) the Downtown Graben (California Geological Survey, 2003). The graben appears to widen to the south towards San Diego Bay. The active fault mapped just east of 16th Street is possibly associated with the eastern limits of the graben. The western limit is roughly mapped along 12th Street. The Regional Fault Map, Figure 6, shows the faults in the downtown San Diego area.

4. SOIL AND GEOLOGIC CONDITIONS

Our field investigation indicates the site is underlain by one surficial soil type consisting of undocumented fill and one geologic units consisting of the Pleistocene age Old Paralic Deposits (map symbol Qop6). The boring logs (Appendix A) and Geologic Map (Figure 2) show the occurrence, distribution, and description of each unit encountered during our field investigation. The Geologic Cross-Section and Trench Log (Figures 3 and 4, respectively), presents a profile view of the underlying geologic conditions. The surficial soil and geologic units are described herein in order of increasing age.

4.1 Undocumented Fill (Qudf)

We encountered isolated pockets of undocumented fill associated with the previous site improvements within our geotechnical borings and fault trench. The fill thickness generally ranges from 6 inches to 3 feet, where encountered. The fill generally consists of medium dense and stiff, reddish brown to brown, clayey sand and clay with some gravel and deleterious materials. The existing fill is considered unsuitable for support of the proposed building structure. We expect the fill materials will be removed within the planned building areas during excavations to achieve finish grade elevations for the subterranean levels. Existing fill exposed at subgrade elevation for proposed adjacent street improvements should be processed, moisture conditioned as necessary and properly compacted. The existing fill material can be reused as properly compacted fill if relatively free from vegetation, debris, and contaminants.

4.2 Old Paralic Deposits (Qop_6)

Quaternary-age Old Paralic Deposits Unit 6 (formerly called the Bay Point Formation) underlies the existing fill soil. The upper 25 feet of the Old Paralic Deposits consists of a moderately cemented, medium dense to very dense, yellowish brown to reddish brown, silty and clayey sand with some gravel. The Old Paralic Deposit materials underlying the upper materials consists of an olive gray to gray brown, stiff to very stiff, sandy silt and clay. These materials were encountered to the maximum depth explored of 51½ feet. The Old Paralic Deposits possess a "very low" to "low" expansion potential (expansion index of 50 or less). Old Paralic Deposits are considered suitable for direct support of structural loads.

5. GROUNDWATER

We did not encounter groundwater in our geotechnical borings to the maximum depth explored of 51½ feet or an elevation of roughly 10 feet above MSL. It is typical to see groundwater from 0 to 5 feet above MSL in the downtown area. Based on a proposed finish floor elevation of about 38½ feet MSL, we do not expect groundwater to be encountered during construction of the proposed development. It is possible that perched seepage layers may be encountered during excavation and drilling operations due to adjacent irrigation and drainage practices. It is not uncommon for perched groundwater conditions to develop where none previously existed. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheets 13 and 17 defines the site with a *Hazard Category 13: Downtown Special Fault Zone*. Based on a review of the map (see Figure 7 - Downtown Special Fault Zone Map), a fault does not traverse the planned development area.

6.2 Faulting

By definition of California Geological Survey (CGS), an active fault is a fault that has had surface displacement in Holocene time (approximately 11,000 years). Potentially active faults are defined as faults with activities during the Pleistocene age (between 1,600,000 and 11,000 years ago). According to these definitions, Special Studies Zones mandated by the State of California (Alquist-Priolo) Geologic Hazards Zones Act was adopted. The purpose of this act is to assure that structures with human occupancy are not constructed across traces of active faults.

The site is located immediately south of the Rose Canyon Fault Zone in an area that is transitional between the predominately right-lateral slip faulting characteristic of the faults north of the downtown area and the predominately dip-slip faulting characteristic of faults making up the southern portion of the Rose Canyon Fault Zone (Treiman, 1993). South of the downtown area, the major faults that compose the southern end of the Rose Canyon Fault Zone are the Spanish Bight, Coronado, and Silver Strand Faults. The east side of this zone is represented by the La Nación Fault

(Treiman, 1993). Together, these faults define a wide and complexly faulted basin occupied by San Diego Bay and a narrow section of the continental shelf west of the Silver Strand.

Trenching by Lindvall and others (1990) on the Rose Canyon Fault in Rose Canyon several miles north of the site, by Owen Consultants (referenced by ICG, 1990) for the police station on a site southeast of the subject property, and by Kleinfelder Incorporated at a site near First Avenue and Market Street in the downtown area, have shown that Holocene soil (soil 11,000 years old or less) has been displaced by faulting within the Rose Canyon Fault Zone.

The California Geological Survey has issued a revised Alquist-Priolo Earthquake Fault Zone Map for the Point Loma Quadrangle (CGS, 2003) that includes portions of the downtown San Diego area. Fault splays associated with the Downtown Graben and the San Diego Fault are considered active by the State of California (Treiman, 2002, 2003) and Alquist-Priolo Earthquake Fault Zones have been established for these faults as shown on Figure 6 - Regional Fault Map.

A review of geologic literature and experience with the soil and geologic conditions in the general area, indicate that known active, potentially active, or inactive faults are not located at the site. The site is, however, located in close proximity to known faults. The property is not located within a State of California Earthquake Fault Zone; however, the site is located approximately 3,000 feet from the eastern active fault trace designated in downtown San Diego. The property is also located within the City of San Diego Special Studies Fault Zone (see Figure 7).

We reviewed several fault investigation reports for sites within the immediate areas. Based on our review of these documents, there is no indication of active faulting or off-fault deformation in the immediate site vicinity. We discuss the specific reports reviewed and the results in subsequent sections of this report.

6.3 Surface Fault Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the earth surface. We performed a site-specific fault rupture hazard investigation at the site that included excavation of an exploratory trench along an east-west trending transect across the site to evaluate the potential for faulting. The trench and exploration transect were oriented to specifically evaluate faults that trend N16W to N16E and 30 degrees from this anticipated trend. The results of our fault rupture hazard evaluation indicate the potential for surface fault rupture at the site is negligible due to the absence of active faults at the subject site. The details of our site-specific fault rupture hazard investigation are presented in Section 7 of this report.

6.4 Seismicity

The historic seismicity or instrumental seismic record in the San Diego area indicates that there have been numerous minor earthquakes in the San Diego Bay area, including events in 1964 and 1985 between M3 and 4+ (Treiman, 1993). Surface rupture has not been recorded with any of the seismic activity. Anderson and others (1989) indicate that the greatest peak acceleration recorded in the downtown area (at San Diego Light and Power) was 34 cm/sec^2 (0.03g) produced by an offshore earthquake in 1964 (M 5.6).

Anderson and others (1989) have also estimated recurrence times for major earthquakes that may affect the San Diego Region. By combining geologic data with their model for ground motion attenuation for each earthquake event, they have estimated the recurrence rate of various levels of peak ground acceleration in the San Diego area. The results of their work indicate that peak accelerations of 10 to 20 percent gravity (g) are expected approximately once every 100 years (Anderson and others, 1989). Higher peak accelerations will also occur but with a lower probability of occurrence or higher return period.

Lindvall and others (1991) have postulated a maximum likely slip rate of about 2 mm per year and a best estimate of about 1.5 mm per year, based on recent three-dimensional trenching on the Rose Canyon Fault in Rose Canyon several miles north of the site. They found stratigraphic evidence of at least three events during the past 8,100 years. The most recent surface rupture displaces the modern "A" horizon (topsoil), suggesting that this event probably occurred within the past 500 years.

Historically, the Rose Canyon Fault has exhibited low seismicity with respect to earthquakes in excess of magnitude 5.0 or greater. Earthquakes on the Rose Canyon Fault having a maximum magnitude of 6.5 are considered representative of the potential for seismic ground shaking within the property. The "maximum magnitude earthquake" is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework.

According to the computer program *EZ-FRISK* (Version 7.65), 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 Faults, located approximately 1.3 miles west of the site and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood/Rose Canyon Faults 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 Faults are 7.5 and 0.54g, respectively. Table 6.4.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration

(PGA) using Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008 acceleration-attenuation relationships.

TABLE 6.4.1
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	1	7.50	0.46	0.40	0.54
Rose Canyon	1	6.90	0.43	0.40	0.50
Coronado Bank	13	7.40	0.24	0.18	0.23
Palos Verdes Connected	13	7.70	0.26	0.19	0.26
Elsinore	42	7.85	0.14	0.09	0.11
Earthquake Valley	46	6.80	0.08	0.06	0.05

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2007) NGA USGS 2008 in the analysis. Table 6.4.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.4.2
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	0.57	0.50	0.61
5% in a 50 Year Period	0.39	0.34	0.41
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 California Building Code (CBC) guidelines currently adopted by the City of San Diego.

6.5 Seiches and Tsunamis

Seiches are free or standing-wave oscillations of an enclosed water body that continue, pendulum fashion, after the original driving forces have dissipated. Seiches usually propagate in the direction of longest axis of the basin. The site located approximately 2,000 feet from San Diego Bay and is at an elevation of approximately 50 to 60 feet above Mean Sea Level (MSL); therefore, the potential of seiches impacting the site is considered to be 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 may include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first-order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). The largest tsunami recorded in San Diego since 1950 occurred on May 22, 1960, which had maximum run-up amplitudes of 2.1 feet (0.7 meters) [URS, 2004]. Wave heights and run-up elevations from tsunamis along the San Diego Coast have historically fallen within the normal range of the tides. Our review of the map titled *Tsunami Inundation Map for Emergency Planning, State of California, County of San Diego, Point Loma Quadrangle, June 1, 2009*, by CEMA, CGS, and USC, shows that the site is not located within the mapped tsunami hazard zone.

6.6 Liquefaction

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

6.7 Hydroconsolidation

Hydroconsolidation is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the effected soil and overlying foundations or improvements supported thereon. Dry to damp (with a degree of saturation less than about 70 percent), loose to dense sand are typically prone to hydroconsolidation. Potentially compressible soil underlying the proposed structures and existing fill is typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydroconsolidation of the soil exists. The potential for hydroconsolidation can be mitigated by remedial grading and the use of stiffer foundation systems. Based on the results of the laboratory testing, hydroconsolidation potential ranges from about 0.1 to 3.5 percent within the Old Paralic Deposits. We expect the upper 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation within the upper portion of the Old Paralic Deposits ranges up to about 4¼ inches.

6.8 Landslides

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

7. SITE-SPECIFIC FAULT RUPTURE HAZARD INVESTIGATION

7.1 Purpose and Scope

No splays of the Rose Canyon Fault Zone were mapped at the site and the site does not fall within a State of California Alquist-Priolo Earthquake Fault Zone. However, the site is located within a City of San Diego Downtown Special Fault Zone and a site-specific fault rupture hazard investigation is required to evaluate the potential for surface fault rupture at the site.

The purpose of our investigation is to evaluate the presence or absence of faults bisecting the site that may impact the proposed development and to assess the age and continuity of on-site stratigraphy. Our investigation conforms to CGS *Guidelines for Evaluating the Hazard of Surface Fault Rupture* (CGS Note 49), Appendix D of the *City of San Diego Guidelines for Geotechnical Reports* (2011), and current geologic standards-of-practice for the evaluation of potential surface fault rupture.

7.2 Literature Review

We reviewed the following fault and/or geotechnical investigations within the immediate area of the site as shown on the Fault Study Map, Figure 8:

- *2025 Harbor Drive (Geocon, Inc., 2000; Project No. 06155-22-06);*
- *S. Evans Street, Main Street and Newton Avenue (Geocon, Inc., 1993; Project No. 04749-31-02).*

Based on our review of these documents, active faulting or off-fault deformation in the immediate site vicinity is not present. Trenches were excavated on nearby sites to the northwest (Geocon, 1993 and 2000), and no active or potentially active faults were observed at these sites

The closest known active faults are located approximately 4,000 feet to the west within the state-designated Alquist-Priolo Earthquake Fault Zone as shown on the Downtown Special Fault Zone Map, Figure 7. The trend of nearby active faults ranges from N16W to N16E.

7.3 Field Exploration

To investigate the presence or absence of faults at the site, we observed a trench excavation across the property, through the existing asphalt parking lot (Figure 2). As previously described, the predominant trend of documented active and potentially active faults in the area is N16W to N16E. The orientation of our exploratory trench (N70W to N83W) was selected to evaluate this trend and a 30-degree variation of this trend in either direction. Our fault trench does not provide coverage for the southwestern section of the subject property that is proposed to be a parking lot and/or a storm water management device. A detailed log of the south-facing wall of the trench is provided in Figure 4.

7.4 Trench Stratigraphy

The sediments exposed in the trench consist of Old Paralic Deposits, mapped as Unit 6 (Kennedy and Tan, 2008). The San Diego Formation, which often underlies the Old Paralic Deposits in the downtown San Diego area, was not encountered below the site to the maximum depth explored. We classified the sediments within the trench in accordance with the Unified Soil Classification System (USCS) as well as applicable soil taxonomy criteria. The Old Paralic Deposits were divided into three distinct, continuous or relatively continuous Horizons, E, B and C, which were further subdivided where other dominant soil characteristics were observed. An A-Horizon was also uncounted in limited areas, which may have been removed during original site grading. Detailed descriptions of the units are presented on the fault trench log (Figure 4).

The Old Paralic Deposits exposed in the trench generally consist of dense to very dense, brown, reddish brown, grayish brown and yellowish brown, silty and clayey, fine- to coarse-grained sand with variable amounts of fine angular gravel. Beds were generally massive, except for localized channeling. We also observed district lateral variations in grain size within the same beds related to changes in deposition. The base of the exposed Old Paralic Deposits was characterized by a medium- to coarse-grained sand unit that is weakly laminated and locally cross-bedded. The entire stratigraphic sequence is moderately to highly oxidized, with the exception of the lowest portion of the trench, which is unoxidized in some areas. The Old Paralic Deposits are interpreted to be continuous laterally and vertically within the fault trench exposure, and in the small diameter borings to maximum depth explored.

The primary marker bed that infers an un-faulted stratigraphic sequence across the site is the medium- to coarse-grained sand unit observed in the lower third of the trench referred to as Qop_6 (Cv) on the fault trench log (Figure 4). This unit is massive and locally laminated and/or overprinted by laminar oxide films (b-Lams) and in some areas, cross-bedded with well-defined foresets. In general, this unit does not appear to be related to overlying soil development, and is therefore considered equivalent to a C-Horizon. The upper contact with the overlying B-Horizon is sub-horizontal and undulatory implying localized erosion and scour prior to deposition of the overlying sediments.

The overlying B-Horizon is also continuous and unbroken along the length of the fault trench (Figure 4). However, there is some lateral variation within this unit related to changes in sediment deposition, variability in the accumulation of illuvial clays, secondary development of interstitial carbonate and silicate cements and construction of the existing building and parking lot. These lateral variations are typically observed to occur over several feet. For example, a lateral transition from a clayey sand to a clayey sand with gravel, rather than abrupt changes across a discontinuity that may be related to faulting.

Characteristic features observed in the fault trench that infer unbroken/unfaulted stratigraphy at the site are summarized in Table 7.4.

TABLE 7.4
SUMMARY OF MARKER BED CHARACTERISTICS

Fault Trench Unit No.	General Stratigraphic Description
Ci-iii	Continuous B-Horizon, locally subdivided into Bt-, Bk-, and Bkm- Horizons
Cv	Continuous C-Horizon

7.5 Absence of Faulting

As shown on Figure 4, the Pleistocene age geologic units are laterally continuous across the trench. The primary evidence for the absence of active faulting are:

1. No faults documented in the immediate area by Geocon Inc., or other consultants, were observed to project toward the site.

2. The Old Paralic Deposits (minimum age of 120,000 years) were observed to be laterally continuous in the exploratory trench and on adjacent sites to the west (Geocon, 1993), and no faults or fault-related features were observed.

The age, lateral continuity, and lack of deformation of these distinct geologic units, provide clear evidence for continuous, unfaulted, pre-Holocene age sediments across the site and rules out active faulting. Therefore, it is our opinion active, potentially active or inactive faulting is not present on the property. Structural setbacks will not be required for the planned development.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

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

8.1.2 With the exception of possible moderate to strong seismic shaking, we did not observe significant geologic hazards or know of them to exist on the site that would adversely affect the proposed project.

8.1.3 The site is not located within a State of California Earthquake Fault Zone but is located within a fault study zone established by the City of San Diego. Our review of fault investigations for the adjacent properties and our observations during our exploratory operations indicate that there is no evidence of active or potentially active faults traversing the site. The exposed stratigraphic section of Pleistocene aged Old Paralic Deposits observed during trenching is generally horizontal to sub-horizontal and unbroken. We did not observe evidence of shearing, fracturing or offset along sub-vertical discontinuity. It is our opinion that active or potentially active faulting does not pass beneath the site and building setbacks will not be required.

8.1.4 Restrictions on future development at the site are not necessary with respect to the hazard of surface fault rupture. However, a future earthquake originating on a nearby splay of the Rose Canyon Fault could produce very strong near-field ground motions at the site that should be taken into consideration during project design. Also, there is a potential for ground cracking or ground shatter associated with strong ground shaking during an earthquake event on nearby faults to occur beneath the site. The findings of our study are limited to detection of existing seismogenic faults (deep-seated structures) that propagate to the near surface and cannot predict the location of ground shatter associated with strong ground shaking.

8.1.5 Our field investigation indicates the site is underlain by undocumented fill overlying Old Paralic Deposits. The Old Paralic Deposits are considered suitable for the support of settlement-sensitive structures.

8.1.6 We did not encounter groundwater during our field investigation to the maximum depth explored of 51½ feet below the former ground surface or at approximate elevation of 8½ feet above MSL. It is typical to see groundwater from 0 to 5 feet above MSL in the subject area. The proposed bottom elevation of the excavation for the subterranean structure is at least 30 feet above groundwater. Therefore, we do not expect groundwater will be encountered during construction of the proposed development.

8.1.7 The proposed structure can be supported on conventional shallow foundations system founded in Old Paralic Deposits.

8.1.8 We expect the temporary excavations for the parking garage will be supported by a soldier pile and, if necessary, tieback anchor system.

8.1.9 Based on our review of the project plans, we opine the planned development can be constructed in accordance with our recommendations provided herein. We do not expect the planned development will destabilize or result in settlement of adjacent properties or the existing public improvements and street right-of-ways located adjacent to the site if the recommendations of this report are incorporated into project design.

8.1.10 We performed a storm water management investigation to help evaluate the potential for infiltration on the property. Based on the results of our field infiltration testing and laboratory testing, we opine full or partial infiltration on the property should be considered infeasible as discussed in Appendix C.

8.2 Excavation and Soil Conditions

8.2.1 Excavations within the Old Paralic Deposits should generally be possible with moderate to heavy effort using conventional heavy-duty equipment. Localized cemented or very hard zones will likely be encountered that will require very heavy effort to excavate with oversize material generated. The Old Paralic Deposits also can contain cobble and cohesionless sand layers. The contractors should be prepared to handle the potential for seepage and caving during the construction operations.

8.2.2 The soil encountered in our field investigation is considered to be “non-expansive” (expansion index [EI] of 20 or less) as defined by 2016 California Building Code (CBC) Section 1803.5.3. However, some of the soil may be classified and “expansive” (expansion index of greater than 20). Table 8.2 presents soil classifications based on the expansion index. Based on the results of our laboratory testing, presented in Appendix A, we expect the on-site materials will possess a “very low” to “low” expansion potential (expansion index of 50 or less).

TABLE 8.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

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

8.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess “S0” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

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

8.3 Grading

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

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

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

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

8.3.5 Based on our understanding of the project and the results of our prior field investigation, we expect the existing fill and some of the Old Paralic Deposits will be removed during the excavations for the planned subterranean levels and the clayey/silty sand materials of the Old Paralic Deposits will be exposed at the base of the subterranean levels. The actual extent of removals shall be determined in the field by Geocon Incorporated.

8.3.6 Excavated soil that is generally free of deleterious debris and contamination can be placed as fill and compacted in layers to the design finish-grade elevations, if necessary. Fill and backfill materials that will require placement for elevators or adjacent surface improvements should be placed in loose thicknesses of 6 to 8 inches and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM Test Method D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

8.3.7 Import fill (if necessary) should consist of granular materials with a “very low” to “low” expansion potential (EI of 50 or less) free of deleterious material or stones larger than 3 inches and should be compacted as recommended herein. Geocon Incorporated should be notified of the import source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

8.4 Excavation Slopes, Shoring, and Tiebacks

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

8.4.2 Temporary excavations should be made in conformance with OSHA requirements. Undocumented fill should be considered a Type C soil in accordance with OSHA requirements. Compacted fill materials can be considered a Type B soil (Type C soil if seepage or groundwater is encountered) and the Old Paralic Deposits can be considered a Type A soil (Type B soil if seepage or groundwater is encountered). In general, special shoring requirements will not be necessary if temporary excavations will be less than 4 feet in height and raveling of the excavations foes not occur. Temporary excavations greater than 4 feet in height, however, should be sloped back at an appropriate inclination. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

8.4.3 The design of temporary shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging. Excavations exceeding 15 feet (with a level backfill) may require soil nails, tieback anchors, or internal bracing to provide additional wall restraint.

8.4.4 Temporary shoring with a level backfill should be designed using a lateral pressure envelope acting on the back of the shoring and applying a pressure equal to 18H, 12H, or 14H, for a triangular, rectangular, or trapezoidal distribution, respectively, where H is the height of the shoring in feet (resulting pressure in pounds per square foot) as shown in Figure 9. These pressures assume a shoring height of up to about 25 feet and we should be contacted if deeper excavations are planned. Triangular distribution should be used for cantilevered shoring and, the trapezoidal and rectangular distribution should be used for multi-braced systems such as tieback anchors and rakers. The project shoring engineer should determine the applicable soil distribution for the design of the temporary shoring system. Additional lateral earth pressure due to the surcharging effects from construction equipment, sloping backfill, planned stockpiles, adjacent structures and/or traffic loads should be considered, where appropriate, during design of the shoring system.

8.4.5 Passive soil pressure resistance for embedded portions of soldier piles can be based on an equivalent passive soil fluid weight of 400D + 500 psf where D is the depth of embedment, in feet (resulting in pounds per square foot), as shown on Figure 10. This passive resistance assumes we do not encounter the groundwater during the installation of the soldier piles. The passive resistance can be assumed to act over a width of three pile diameters. Typically, soldier piles are embedded a minimum of 0.5 times the maximum height of the excavation (this depth is to include footing excavations) if tieback anchors are not employed. The project structural engineer should determine the actual embedment depth.

8.4.6 Drilled shafts for the soldier piles should be observed by Geocon Incorporated prior to the placement of steel reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata, and design depths. If unexpected soil conditions are encountered, foundation modifications may be required.

8.4.7 Lateral movement of shoring is associated with vertical ground settlement outside of the excavation. Therefore, it is essential that the soldier pile and tieback system allow very limited amounts of lateral displacement. Earth pressures acting on a lagging wall can cause movement of the shoring toward the excavation and result in ground subsidence outside of the excavation. Consequently, horizontal movements of the shoring wall should be accurately monitored and recorded during excavation and anchor construction.

8.4.8 Survey points should be established at the top of the pile on at least 20 percent of the soldier piles. An additional point located at an intermediate point between the top of the pile and the base of the excavation should be monitored on at least 20 percent of the piles if tieback anchors will be used. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter until the permanent support system is constructed.

8.4.9 The shoring system should be designed to limit horizontal and vertical soldier pile movement to a maximum of 1 inch and ½ inch, respectively. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary. The magnitude of movement for intermediate depths and distances from the shoring wall can be linearly interpolated.

8.4.10 The project civil engineer should provide the approximate location, depth, and pipe type of the underground utilities adjacent to the site to the shoring engineer to help select the appropriate shoring type and design. The shoring system should be designed to limit horizontal and vertical soldier pile movement to a maximum of 1 inch and ½ inch, respectively. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary. The magnitude of movement for intermediate depths and distances from the shoring wall can be linearly interpolated. We understand the City of San Diego may require the developer to prepare a hold harmless agreement for the planned construction and development regarding potential damage to the existing utilities and improvements.

8.4.11 Tieback anchors employed in shoring should be designed such that anchors fully penetrate the Active Zone behind the shoring. The Active Zone can be considered the wedge of soil from the face of the shoring to a plane extending upward from the base of the excavation at a 29-degree angle from vertical, as shown on Figure 11. Normally, tieback anchors are contractor-designed and installed, and there are numerous anchor construction methods available. Non-shrinkage grout should be used for the construction of the tieback anchors.

8.4.12 Experience has shown that the use of pressure grouting during formation of the bonded portion of the anchor will increase the soil-grout bond stress. A pressure grouting tube should be installed during the construction of the tieback. Post grouting should be performed if adequate capacity cannot be obtained by other construction methods.

8.4.13 Anchor capacity is a function of construction method, depth of anchor, batter, diameter of the bonded section, and the length of the bonded section. Anchor capacity should be evaluated using the strength parameters shown in Table 8.4.

TABLE 8.4
SOIL STRENGTH PARAMETERS FOR TEMPORARY SHORING

Description	Cohesion (psf)	Friction Angle (degrees)
Old Paralic Deposits (Qop)	450	33

8.4.14 Grout should only be placed in the tieback anchor's bonded section prior to testing. Tieback anchors should be proof-tested to at least 130 percent of the anchor's design working load. Following a successful proof test, the tieback anchors should be locked off at 80 percent of the allowable working load. Tieback anchor test failure criteria should be established in project plans and specifications. The tieback anchor test failure criteria should be based upon a maximum allowable displacement at 130 percent of the anchor's working load (anchor creep) and a maximum residual displacement within the anchor following stressing. Tieback anchor stressing should only be conducted after sufficient hydration has occurred within the grout. Tieback anchors that fail to meet project specified test criteria should be replaced or additional anchors should be constructed.

8.4.15 Lagging should keep pace with excavation and tieback anchor construction. The excavation should not be advanced deeper than three feet below the bottom of lagging at any time. These unlagged gaps of up to three feet should only be allowed to stand for short periods of time in order to decrease the probability of soil instability and should never be unsupported overnight. Backfilling should be conducted when necessary between the back of lagging and excavation sidewalls to reduce sloughing in this zone and all voids should be filled by the end of each day. Further, the excavation should not be advanced further than four feet below a row of tiebacks prior to those tiebacks being proof tested and locked off.

8.4.16 If tieback anchors are employed, an accurate survey of existing utilities and other underground structures adjacent to the shoring wall should be conducted. The survey should include both locations and depths of existing utilities. Locations of anchors should be adjusted as necessary during the design and construction process to accommodate the existing and proposed utilities.

8.4.17 If a raker system is employed, the rakers should not be inclined steeper than 1:1 (horizontal to vertical) to provide an excavation to the raker foundation system with an inclination less than 1:1. A shallow or deep foundation system can be used for the raker system.

8.4.18 Shallow foundations for the raker system should consist of continuous strip footings and/or isolated spread footings. Continuous and isolated footings should be at least 24 inches wide and extend at least 12 inches below lowest adjacent pad grade. Steel reinforcement for the footings should be designed by the project structural engineer. Foundations may be designed for an allowable soil bearing pressure of 4,000 psf for footings bearing in the Old Paralic Deposits.

8.4.19 The condition of existing buildings, streets, sidewalks, and other structures/improvements around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements and other improvements. Underground utilities sensitive to settlement should be videotaped prior to construction to check the integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation and upon existing buildings. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter. Inclinometers should be installed and monitored behind any shoring sections that will be advanced deeper than 30 feet below the existing ground surface.

8.5 Seismic Design Criteria

8.5.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 8.5.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements should be designed using a Site Class C. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 8.5.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 8.5.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.210g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.466g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.000	Table 1613.3.3(1)
Site Coefficient, F_V	1.334	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.210g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.622g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.807g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.414g	Section 1613.3.4 (Eqn 16-40)

8.5.2 Table 8.5.2 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 8.5.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.542g	Figure 22-7
Site Coefficient, F_{PGA}	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.542g	Section 11.8.3 (Eqn 11.8-1)

8.5.3 Conformance to the criteria in Tables 8.5.1 and 8.5.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.6 Conventional Shallow Foundations

8.6.1 The proposed structure can be supported on a conventional shallow foundation system bearing on the properly compacted fill. Foundations for the structures should consist of continuous strip footings and/or isolated spread footings. Continuous footings should be at least 12 inches wide and extend at least 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 24 inches and depth of 24 inches. Figure 12 presents a footing dimension detail depicting the depth to lowest adjacent grade.

8.6.2 Steel reinforcement for continuous footings should consist of at least four No. 4 steel reinforcing bars placed horizontally in the footings, two near the top and two near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer. The minimum reinforcement recommended herein is based on soil characteristics only (Expansion Index of 50 or less) and is not intended to replace reinforcement required for structural considerations.

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

8.6.4 The recommended allowable bearing capacity for foundations with minimum dimensions described herein and bearing in formational materials at least 10 feet below the ground surface is 6,000 pounds per square foot (psf). An additional 1,000 psf can be added to the allowable bearing capacity for excavations of 20 feet or greater below the ground surface. The allowable soil bearing pressure may be increased by an additional 500 psf for each additional foot of depth and 300 psf for each additional foot of width, to a maximum allowable bearing capacity 8,000 psf. The values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces. These values are based on an anticipated maximum excavation depth of 25 feet.

8.6.5 Total and differential settlement of the building founded on the Old Paralic Deposits is expected to be less than ½-inch for a 9-foot square footing. The total and differential settlement for a 16-foot square footing is 1 inch and ½ inch, respectively.

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

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

8.7 Concrete Slabs-on-Grade

8.7.1 Interior concrete slabs-on-grade for the subterranean parking structure should be at least 5 inches thick. As a minimum, reinforcement for slabs-on-grade should consist of No. 4 reinforcing bars placed at 18 inches on center in both horizontal directions.

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

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

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

8.7.5 To control the location and spread of concrete shrinkage cracks, crack control joints should be provided. The crack control joints should be created while the concrete is still fresh using a grooving tool, or shortly thereafter using saw cuts. The structural engineer should take into consideration criteria of the American Concrete Institute when establishing crack control spacing patterns.

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

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

8.8 Concrete Flatwork

8.8.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and, when in excess of 8 feet square, should be reinforced with 6 x 6 - W2.9/W2.9 (6 x 6 - 6/6) welded wire mesh or No. 3 reinforcing bars at 18 inches on center in both directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be checked prior to placing concrete.

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

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

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

8.9 Retaining Walls

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

8.9.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform (rectangular) pressure of 7H psf and 13H psf should be added to the active soil pressure where the planned walls are 8 feet or less and the portion of walls greater than 8 feet, respectively. 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. In addition, the loading from adjacent structures should be incorporated into the design of the planned retaining walls by the structural engineer.

8.9.3 The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted free-draining backfill material (EI of 50 or less) with no hydrostatic forces or imposed surcharge load. Figures 13 and 14 present typical retaining wall drain details for conventional and soldier pile walls. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

8.9.4 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure. A seismic load of 18H psf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. We used the site specific peak ground acceleration, PGA_M, of 0.542g calculated from ASCE 7-10 Section 11.8.3.

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

8.10 Lateral Loading

8.10.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 350 pounds per cubic foot (pcf) should be used for the design of footings or shear keys poured neat in compacted fill. The passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

8.10.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.35 should be used for design. The friction coefficient may be reduced depending on the vapor barrier or waterproofing material used for construction in accordance with the manufacturer's recommendations (typically a reduced friction coefficient of about 0.2 to 0.25).

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

8.11 Preliminary Pavement Recommendations

8.11.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the

R-Value of the subgrade soil encountered at final subgrade elevation. Based on the results of our R-value testing of the subgrade soils, we have assumed an R-Value of 6 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 8.11.1 presents the preliminary flexible pavement sections.

TABLE 8.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	6	3	10
Driveways for automobiles and light-duty vehicles	5.5	6	3	12
Medium truck traffic areas	6.0	6	3.5	13
Driveways for heavy truck traffic	7.0	6	4	16

8.11.2 The subgrade soils for pavement areas should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above the optimum moisture content. The depth of subgrade compaction should be approximately 12 inches.

8.11.3 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 (Greenbook)*.

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

8.11.5 A rigid Portland Cement concrete (PCC) pavement section should be placed in driveway entrance aprons, trash bin loading/storage areas and loading dock areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 8.11.2.

TABLE 8.11.2
RIGID PAVEMENT DESIGN PARAMETERS

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

8.11.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.11.3.

TABLE 8.11.3
RIGID PAVEMENT RECOMMENDATIONS

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

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

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

8.11.9 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 20 feet for the slabs and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report. The depth of the crack-control joints should be at least ¼ of the slab thickness when using a conventional saw, or at least 1 inch when using early-entry saws on slabs 9 inches or less in thickness, as determined by the referenced ACI report discussed in the pavement section herein. Cuts at least ¼ inch wide are required for sealed joints, and a ⅜ inch wide cut is commonly recommended. A narrow joint width of 1/10 to 1/8 inch-wide is common for unsealed joints.

8.11.10 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed at the as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

8.11.11 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, cross-gutters, or sidewalk so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

8.12 Site Drainage and Moisture Protection

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

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

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

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

8.13 Improvement/Grading and Foundation Plan Review

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

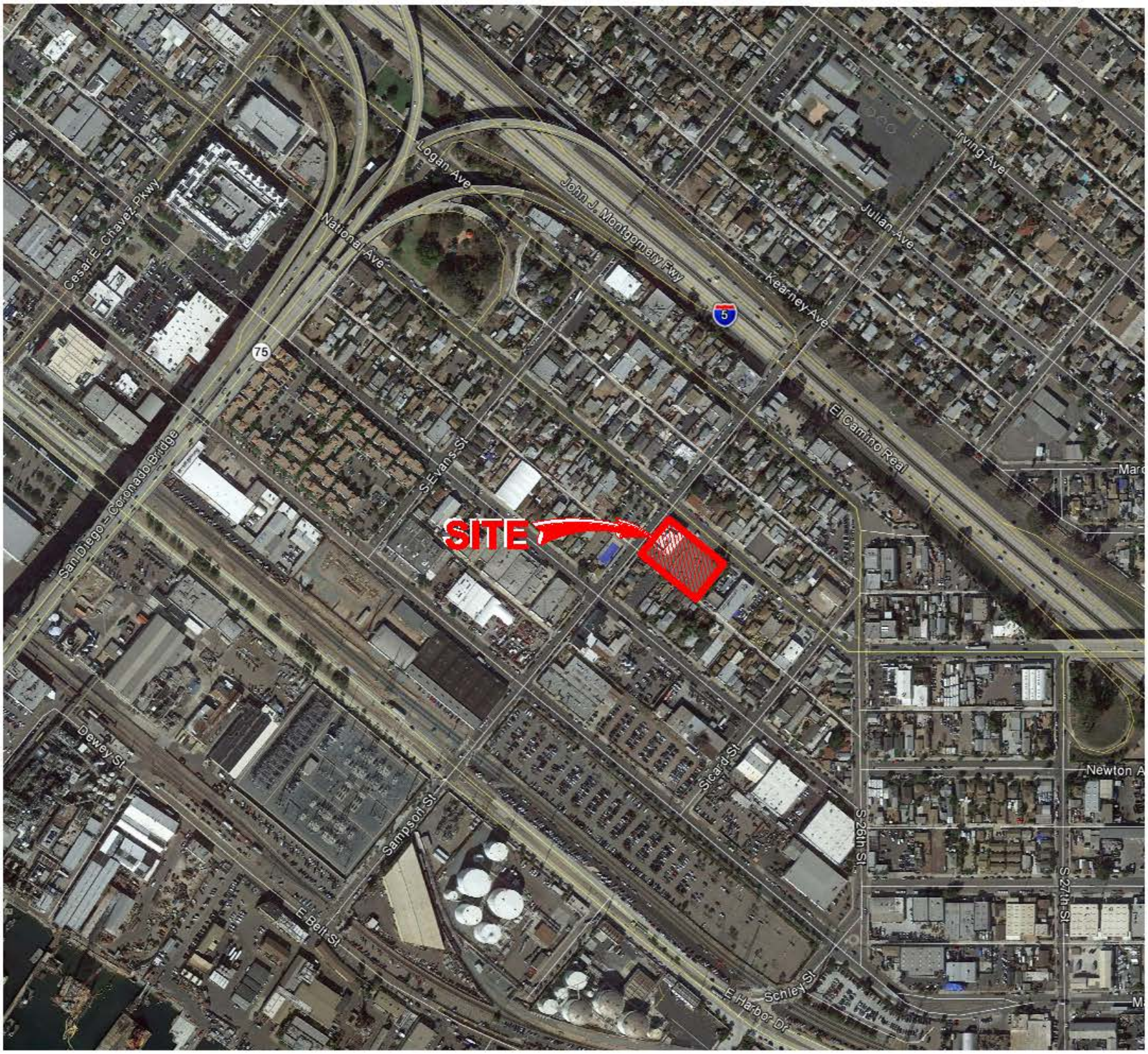
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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



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.

N

NO SCALE

VICINITY MAP

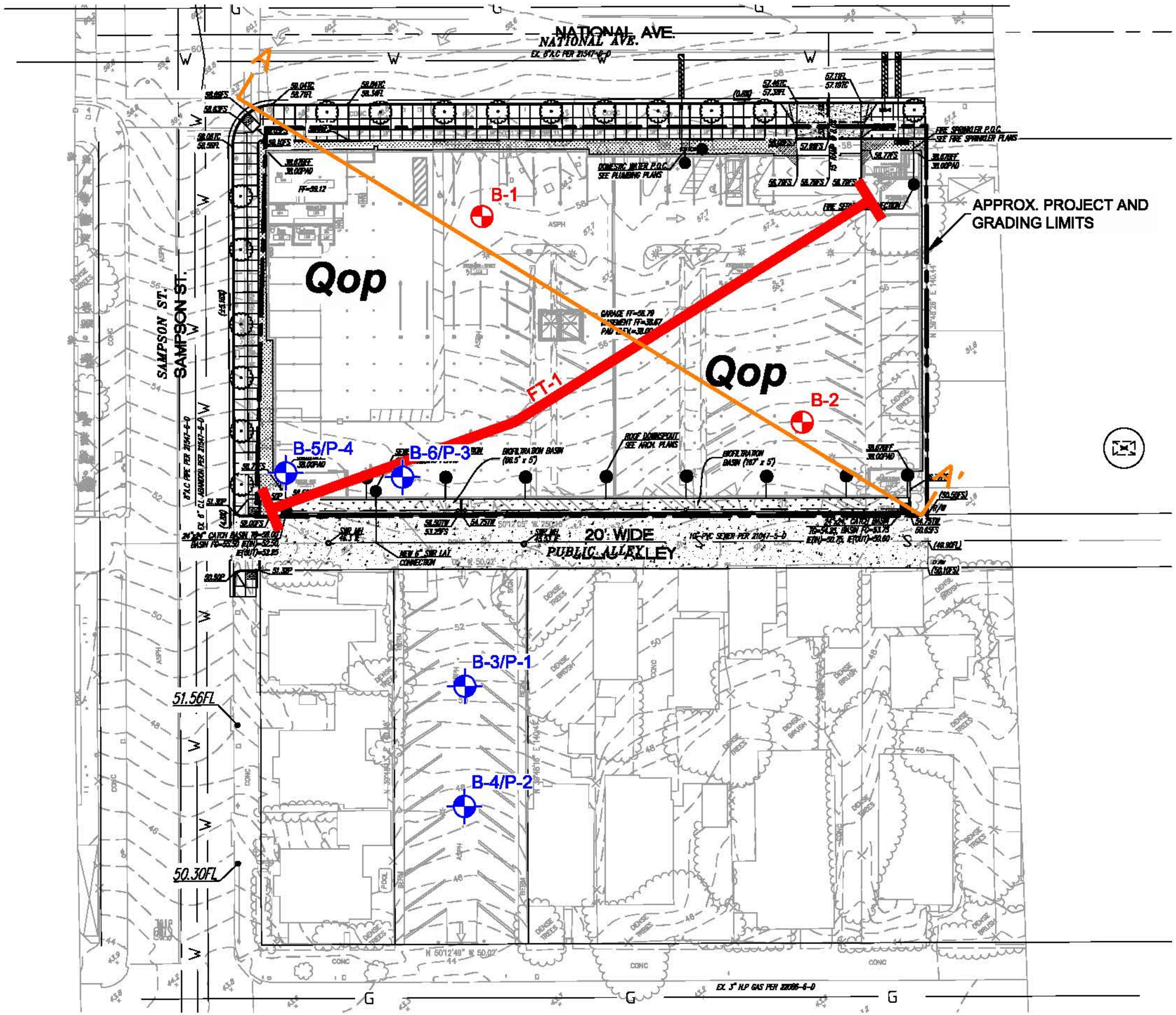
GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 1

Plotted:12/04/2017 4:39PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\DETAILS\G2093-52-01_Vicinity Map.dwg

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA



0' 20' 40' 60' 80'
SCALE 1"= 40' (On 11x17)

GEOCON LEGEND

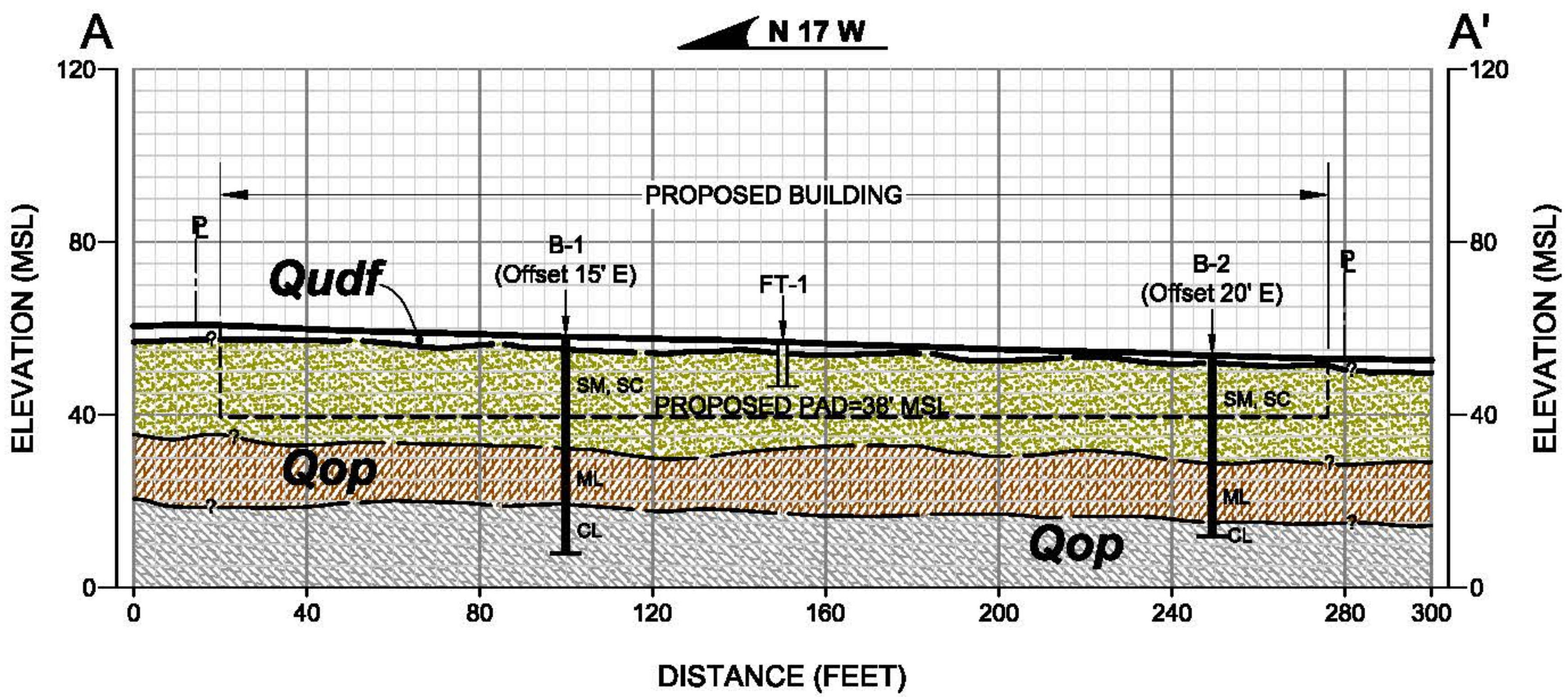
- **Qop**........OLD PARALIC DEPOSITS
- B-2........APPROX. LOCATION OF BORING
- B-5/P-6........APPROX. LOCATION OF BORING AND PERCOLATION TEST
- ....APPROX. LOCATION OF FAULT TRENCH
- A A'........GEOTECHNICAL CROSS - SECTION

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

GEOLOGIC MAP

PROJECT NO. G2093 - 52 - 01
FIGURE 2
DATE 12 - 05 - 2017

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION A-A'

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

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qop ....OLD PARALIC DEPOSITS

Silty/Clayey Sand

Silt

Clay

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

FT-1 ....APPROX. LOCATION OF TRENCH

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

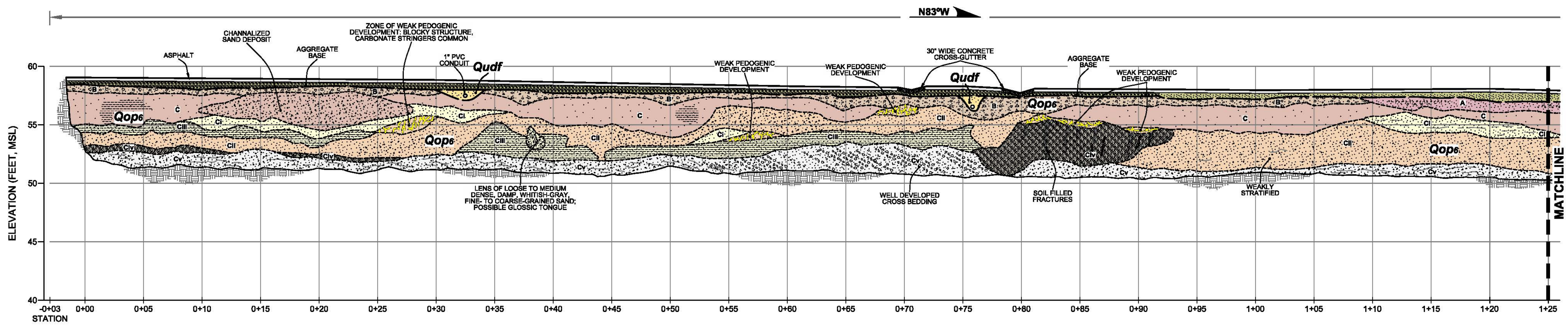
........APPROX. LOCATION OF INTERFORMATIONAL CONTACT (Queried Where Uncertain)

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

GEOLOGIC CROSS - SECTION

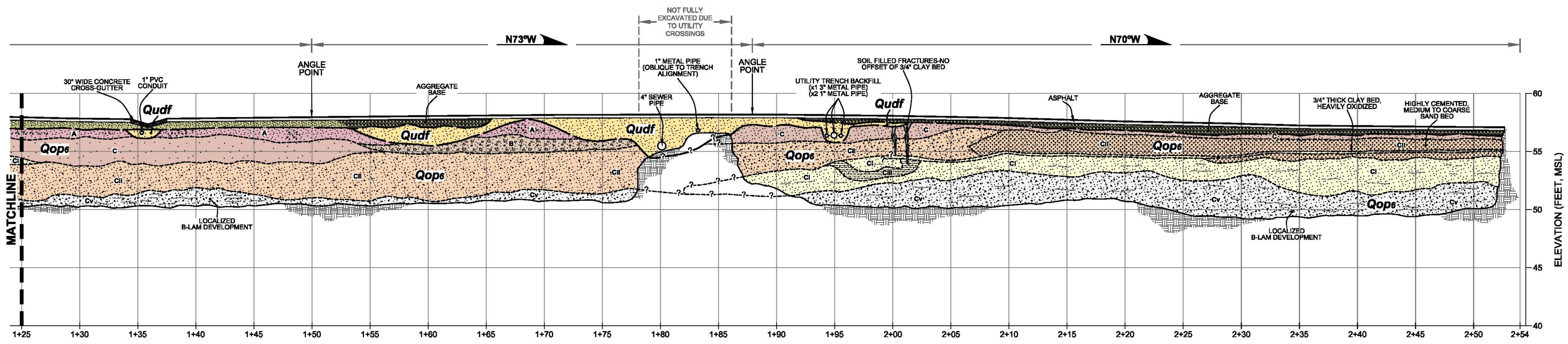
PROJECT NO. G2093 - 52 - 01
FIGURE 3
DATE 12 - 05 - 2017

Plotted:12/04/2017 4:22PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\SHEETS\G2093-52-01 CrossSection.dwg



FAULT TRENCH

SCALE: 1" = 5' (Vert. = Horiz.)



FAULT TRENCH

SCALE: 1" = 5' (Vert. = Horiz.)

PAVEMENT SECTION:

a. Station 0+00 to 0+92: 3 to 4 inches asphaltic concrete over 6 inches of aggregate base. At least one overlay present, with petromat observed in some areas.

b. Station 0+92 to 1+52: 3 to 4 inches of asphaltic concrete over dense, dry to damp, gray, medium grained Silty Sand (SM).

c. Station 1+52 to 1+65: 3 to 4 inches asphaltic concrete over 6 inches of aggregate base.

d. Station 1+65 to 1+91.5: 4 inches asphaltic concrete over subgrade soil consisting of medium dense, damp to moist, brown to grayish brown Clayey Sand (SC) and Silty Sand (SM).

e. Station 1+91.5 to 2+52.5: 3 inches asphaltic concrete over 6 inches of aggregate base.

Qudf ....**UNDOCUMENTED FILL:** Loose to medium dense, damp to moist, brown, yellowish brown to grayish brown (mottled) Silty Sand (SM) and Clayey Sand (SC) mixture; trace rock fragments <1.5 inches and occasional trash and debris observed. Fill soil is confined to zones of localized trench backfill with the exception of station 1+54 to 2+05 where fill is present below the pavement section to depths up to 4 feet below existing grade.

Qop6 ....**OLD PARALIC DEPOSITS (Late to middle Pleistocene):** Poorly sorted, moderately permeable, reddish-brown, inter-fingered strandline beach, estuarine and colluvial deposits of siltstone, sandstone and conglomerate. These deposits rest on the 22-23m Nestor terrace (Kennedy and Tan, 2008). This unit is further subdivided on the fault trench log as follows:

A ....Stiff, moist, red to reddish-brown Clay (CL); trace fine gravel and coarse sand. Localized pedogenic development with 4-6 inch tall peds in some areas. Carbonate stringers common, locally reworked during paving operations. Possibly equivalent to an A-Horizon.

B ....Dense, damp to moist, brown to grayish-brown, fine grained Silty Sand (SM); massive with occasional fine, <1/4-inch angular gravel. Locally reworked and/or bisected by trench backfill. Equivalent to an E-Horizon due to low organic content and light color characteristic of oxide leaching.

C ....Dense, damp, brown, fine to medium grained Silty Sand (SM) with trace clay; occasional fine, <1/4-inch angular gravel and <1/8 inch manganese nodules. Pinhole porosity common throughout, generally massive, but locally channelized as noted on log. Equivalent to a B-Horizon, but can be further subdivided as noted:

i ....Dense, damp to moist, pale yellowish-brown to grayish-brown (Mottled), Clayey Sand (SC); discontinuous lenses, pods and films of clay throughout (translocated clays), laterally discontinuous. Equivalent to a Bt-Horizon.

ii ....Very Dense, dry to damp, pale reddish-brown to orange brown, fine to coarse grained Silty Sand (SM); weakly to moderately cemented by interstitial carbonate as noted on log. Localized zones of weak pedogenic development noted at contact with overlying B-Horizon, often with localized fracture infills. Equivalent to a Bk-Horizon. Grades laterally into dense to very dense, reddish-brown, medium to coarse grained Clayey Sand with gravel. Heavily cemented with non-carbonate cement north of station 2+06 (Bm-Horizon)

iii ....Very dense, dry, white to pale brown, fine grained Sand; heavily cemented, oxide coatings noted along fractures. Laterally discontinuous with variable thickness. Equivalent to a Bkm-Horizon.

iv ....Dense, damp, gray to grayish-brown, very fine grained Sandy Clay (SC-CL); laterally discontinuous, interfingers with sand below.

v ....Loose to medium dense, whitish-gray to orange brown, medium to coarse grained Sand (SP); Laminated and locally cross bedded. Subhorizontal B-Lams defined by oxide grain coatings noted in some areas. Equivalent to a C-Horizon.

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

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

........INTERSTITIAL CARBONATE DEVELOPMENT

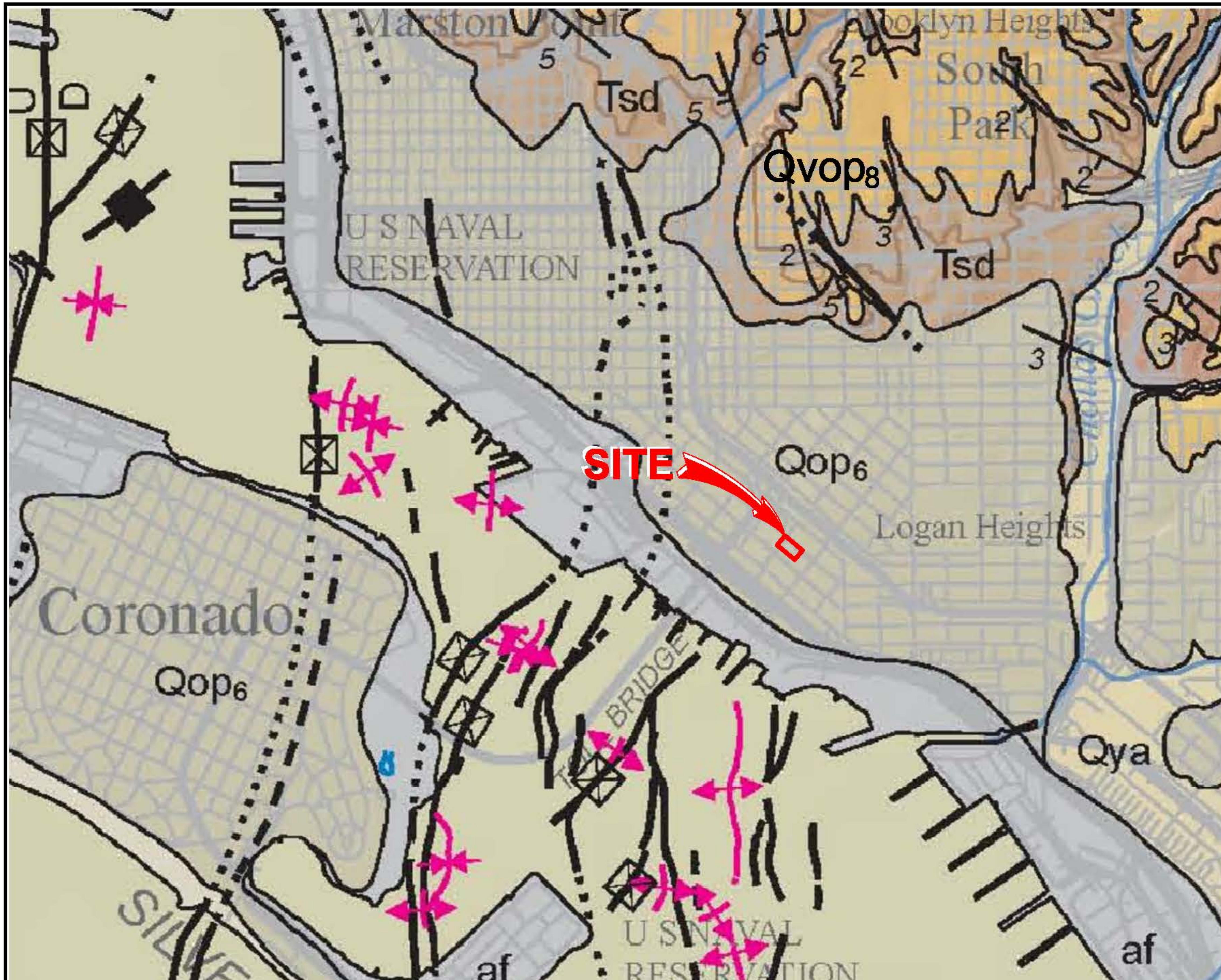
........HEAVY CEMENTATION

FAULT TRENCH FT-1

NATIONAL AND SAMPSON
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" = 5'	DATE 12 - 05 - 2017		
PROJECT NO. G2093 - 52 - 01		FIGURE 4	
SHEET 1 OF 1			



2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

0' 2000' 4000'
SCALE 1"= 2000' (On 11x17)

GEOCON LEGEND

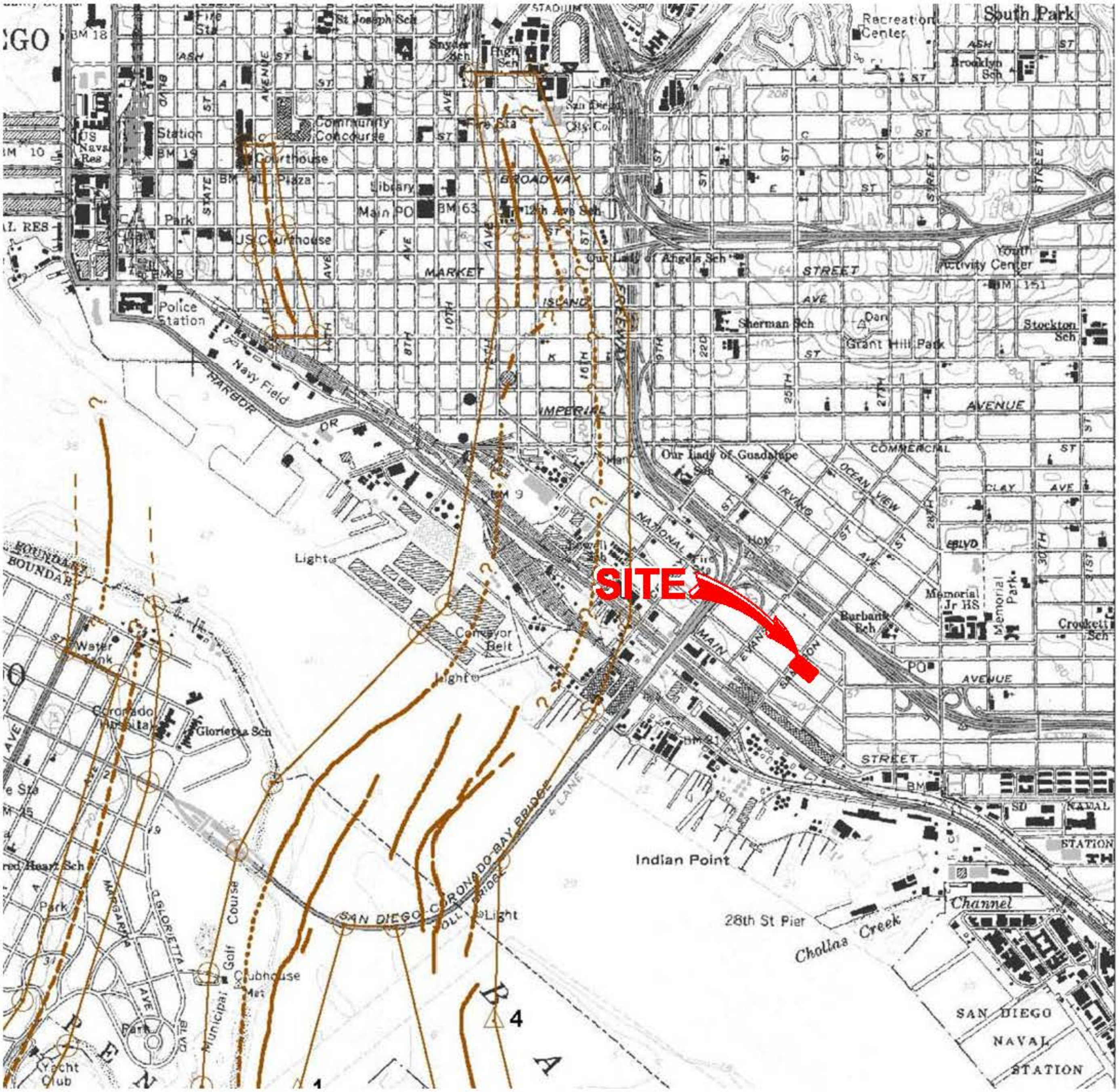
af ....ARTIFICIAL FILL
Qop6 ....OLD PARALIC DEPOSITS
Qvop8 ....VERY OLD PARLIC DEPOSITS
Tsd ....SAN DIEGO FORMATION

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

PROJECT NO. G2093 - 52 - 01

REGIONAL GEOLOGIC MAP

FIGURE 5
DATE 12 - 05 - 2017



REFERENCE:
CALIFORNIA GEOLOGICAL SURVEY, STATE OF CALIFORNIA EARTHQUAKE FAULT ZONE, POINT LOMA QUADRANGLE, MAY 1, 2003

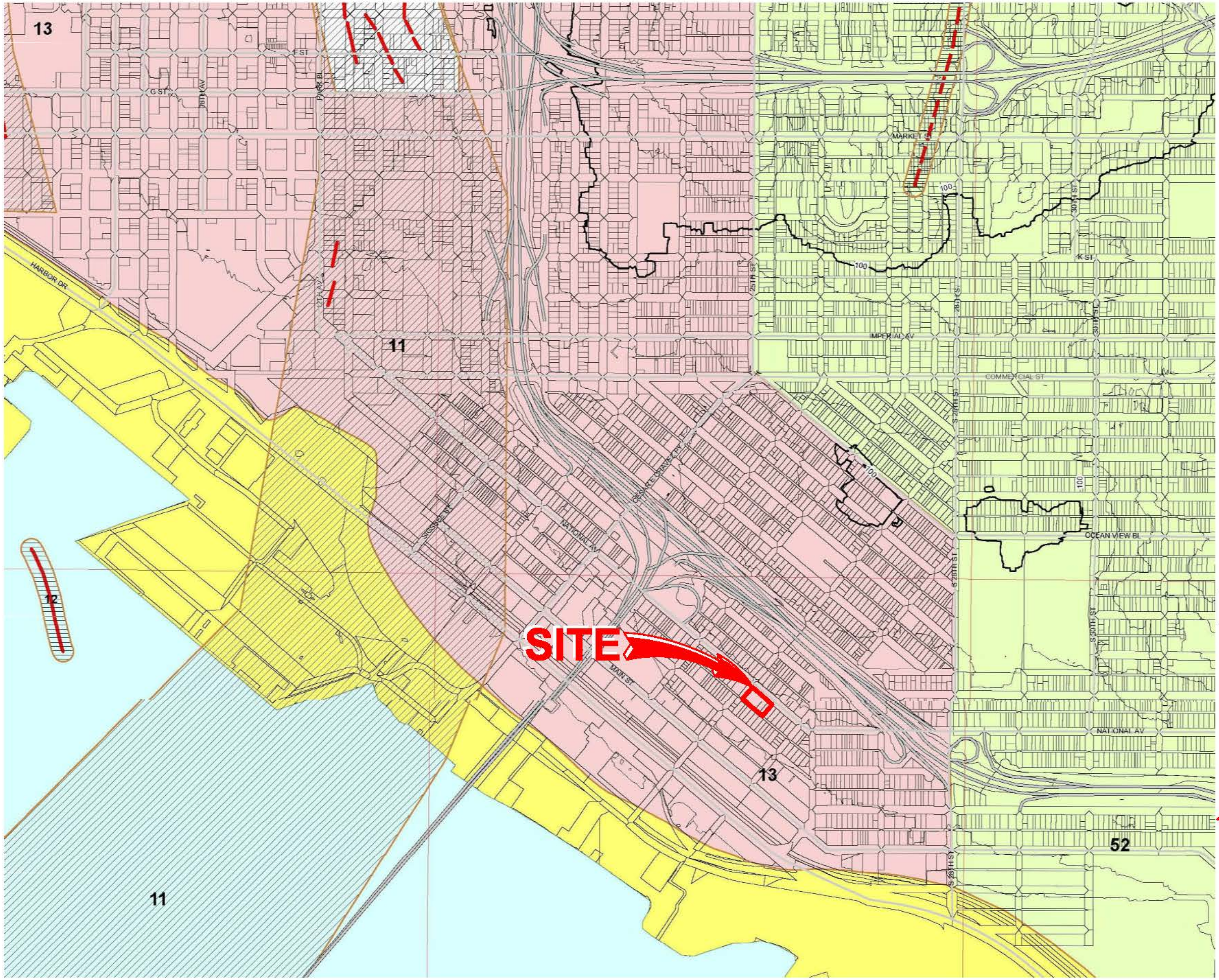
N
NO SCALE

REGIONAL FAULT MAP

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 6



2209 NATIONAL AVENUE
DIEGO, CALIFORNIA

0' 1000' 2000'
SCALE 1"= 1000' (On 11x17)

........DOWNTOWN SPECIAL FAULT ZONE

........HIGH POTENTIAL--SHALLOW GROUNDWATER MAJOR DRAINAGES, HYDRAULIC PILLS

........OTHER LEVEL AREAS, GENTLY SLOPING TO STEEP TERRAIN, FAVORABLE GEOLOGIC STRUCTURE, LOW RISK

........WATER

11 ....ACTIVE ALQUIST-PRIOLO EARTHQUAKE FAULT ZONE

12 ....POTENTIALLY ACTIVE, INACTIVE, PRESUMED INACTIVE OR ACTIVELY UNKNOWN

........FAULT (Dashed Where Inferred)

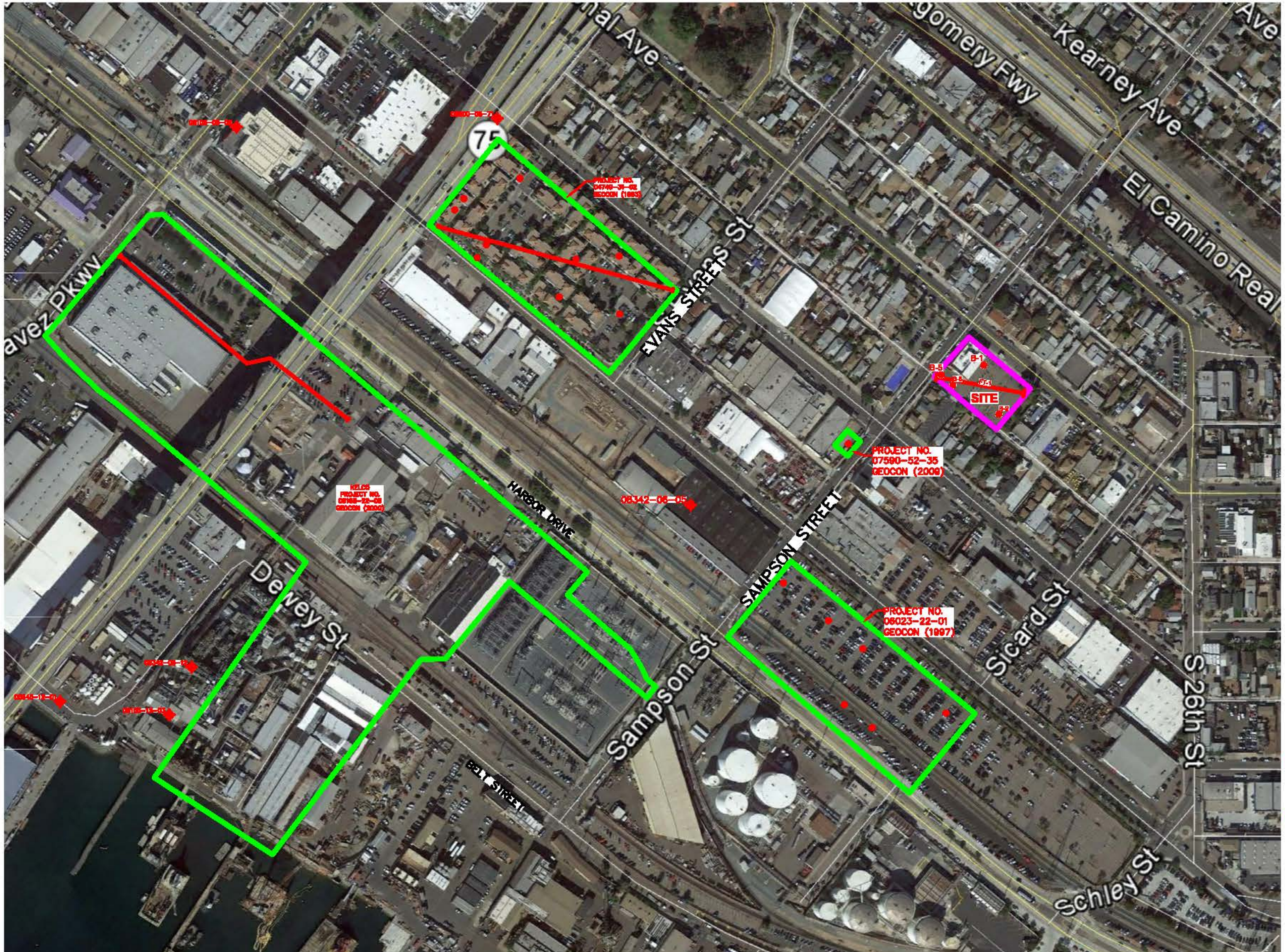
GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

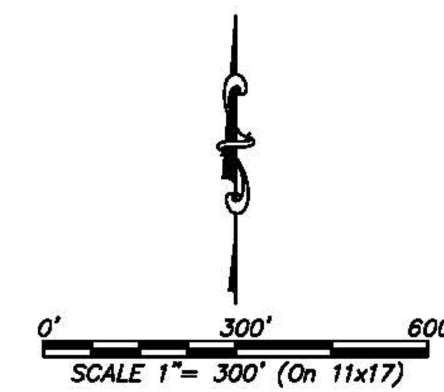
PROJECT NO. G2093 - 52 - 01

DOWNTOWN SPECIAL FAULT ZONE MAP

FIGURE 7
DATE 12 - 05 - 2017

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA





GEOCON LEGEND

B-6 ....APPROX. LOCATION OF BORING

FT-1 ....APPROX. LOCATION OF FAULT TRENCH

........APPROX. LOCATION OF FAULT TRENCH, COMPANY THAT PERFORMED THE TRENCH (Year Reported)

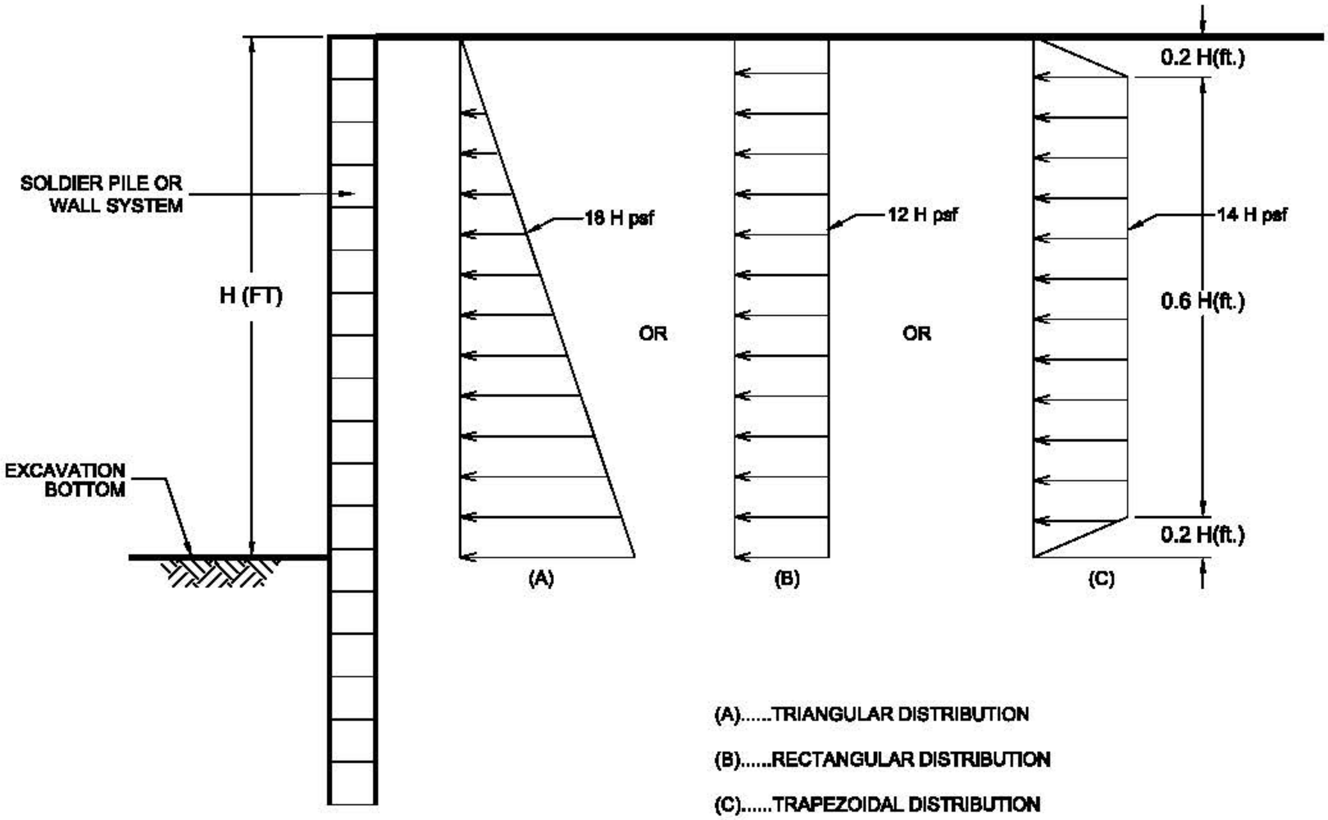
........APPROX. LOCATION OF PREVIOUS PROJECT BOUNDARY

GEOCON INCORPORATED

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

FAULT STUDY MAP

PROJECT NO. G2093 - 52 - 01
FIGURE 8
DATE 12 - 05 - 2017



(A)......TRIANGULAR DISTRIBUTION

(B)......RECTANGULAR DISTRIBUTION

(C)......TRAPEZOIDAL DISTRIBUTION

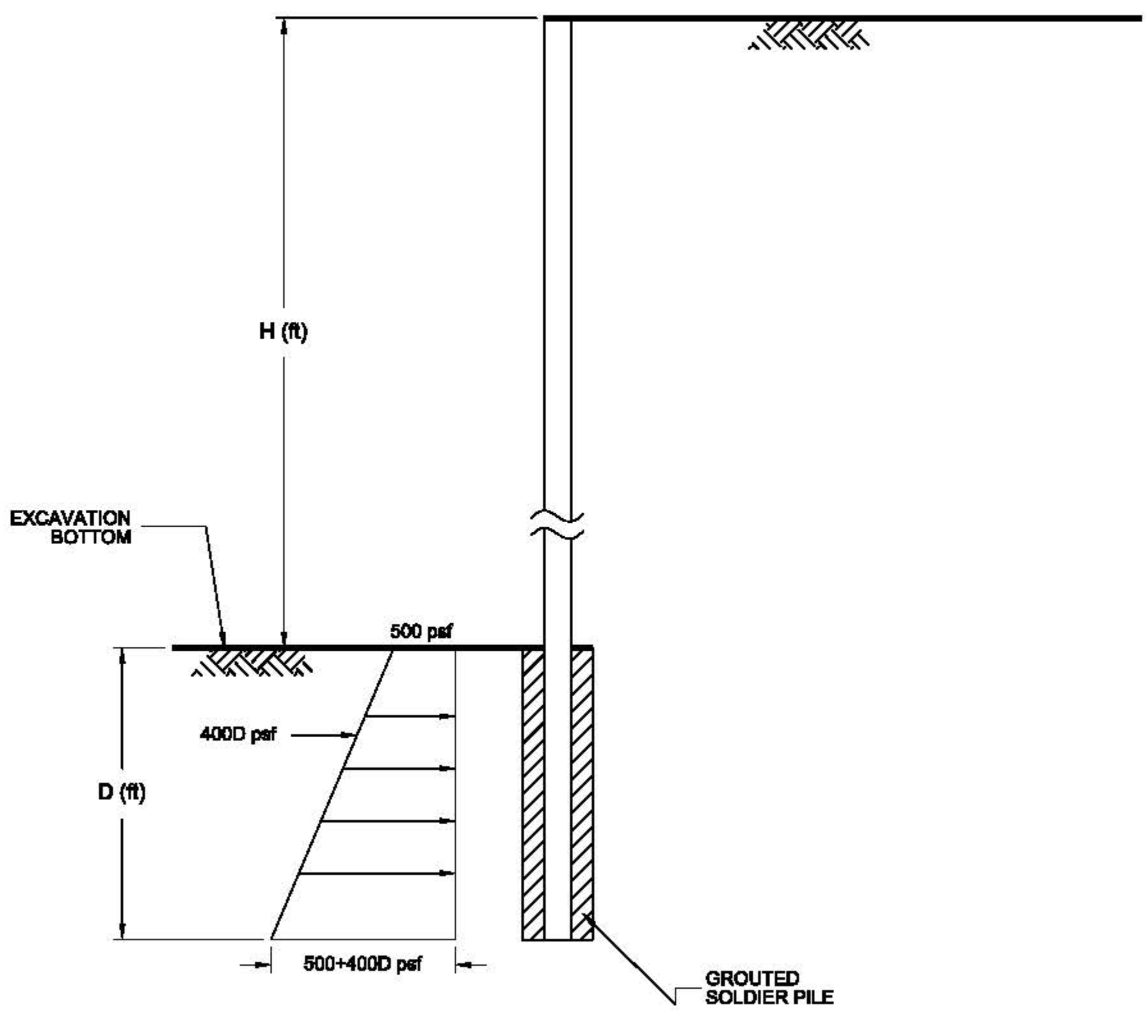
NO SCALE

LATERAL ACTIVE PRESSURES FOR TEMPORARY SHORING

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 9



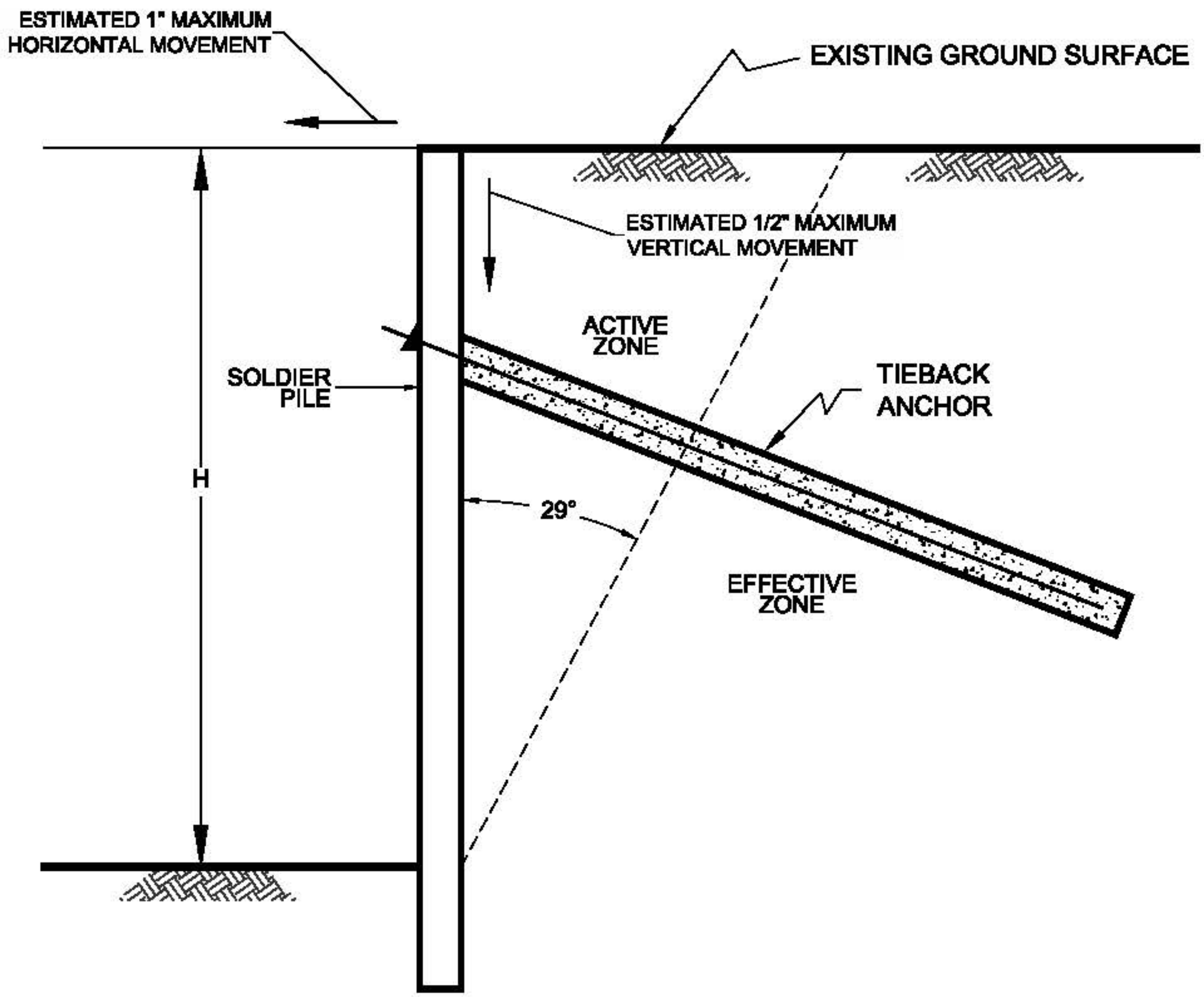
NO SCALE

SOLDIER PILE PASSIVE PRESSURE DISTRIBUTION

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 10



NOTE: NO ESTIMATED MOVEMENT AT EFFECTIVE ZONE

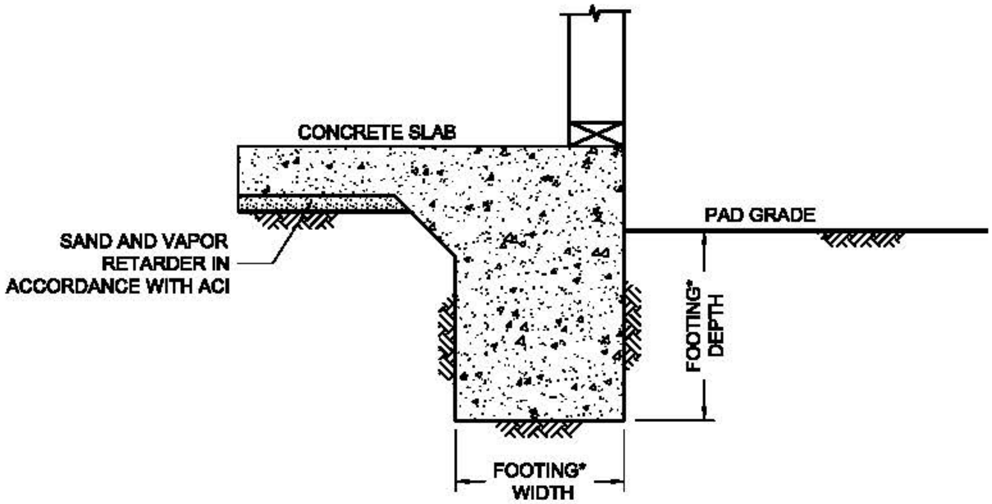
NO SCALE

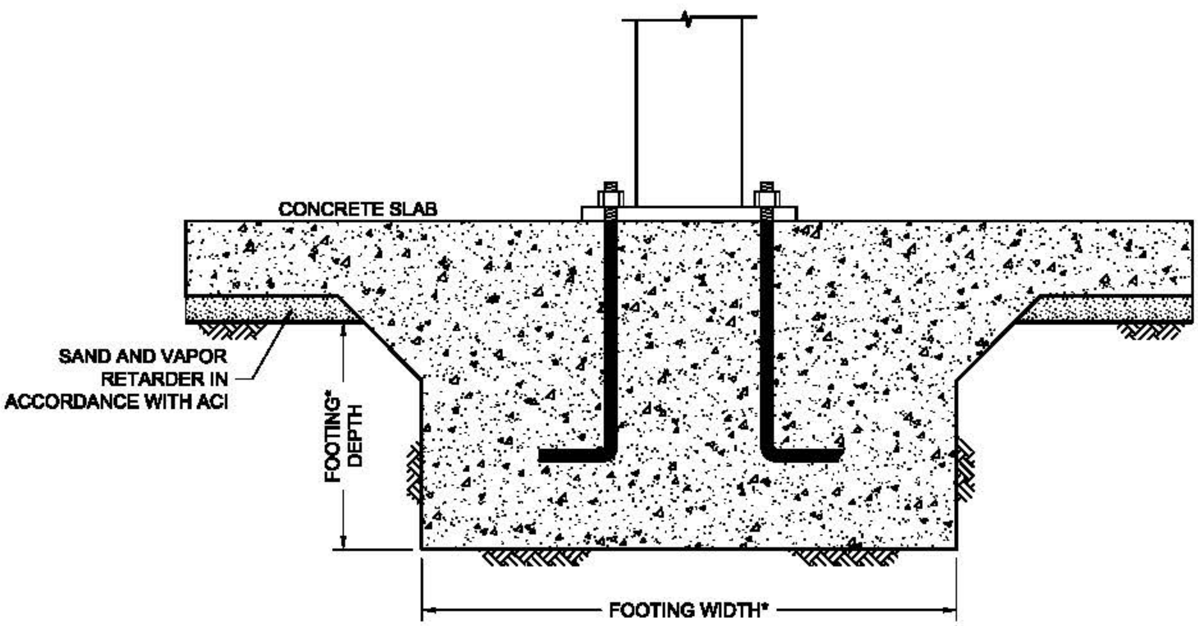
RECOMMENDED EFFECTIVE ZONE FOR TIEBACK ANCHORS

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 11





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

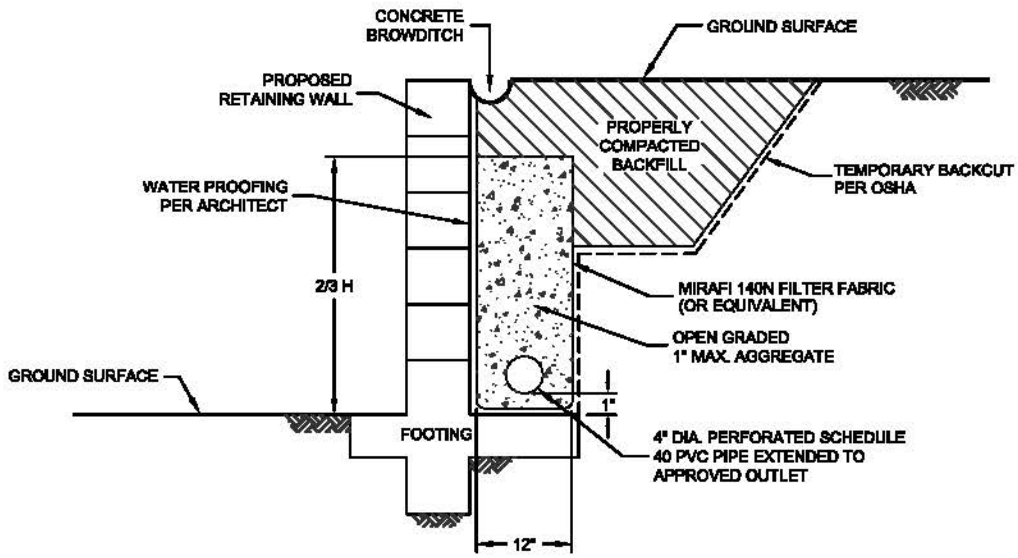
NO SCALE

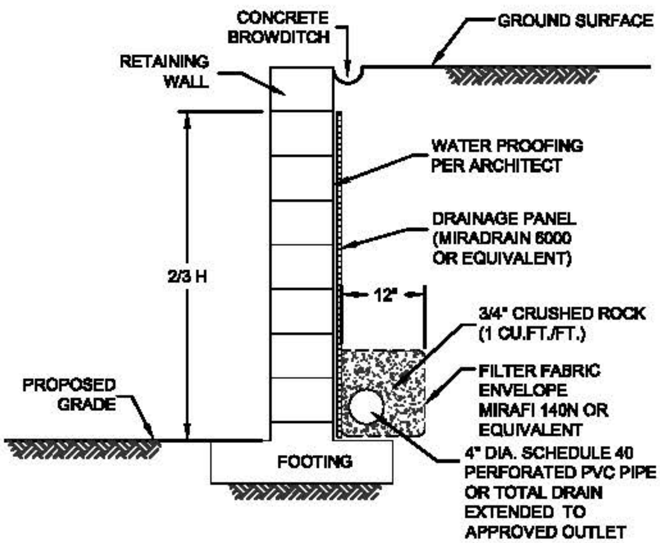
WALL / COLUMN FOOTING DIMENSION DETAIL

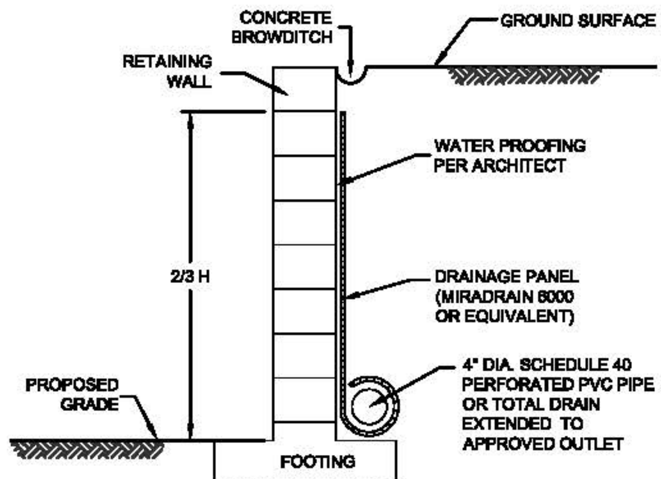
GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 12







NOTE :

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

NO SCALE

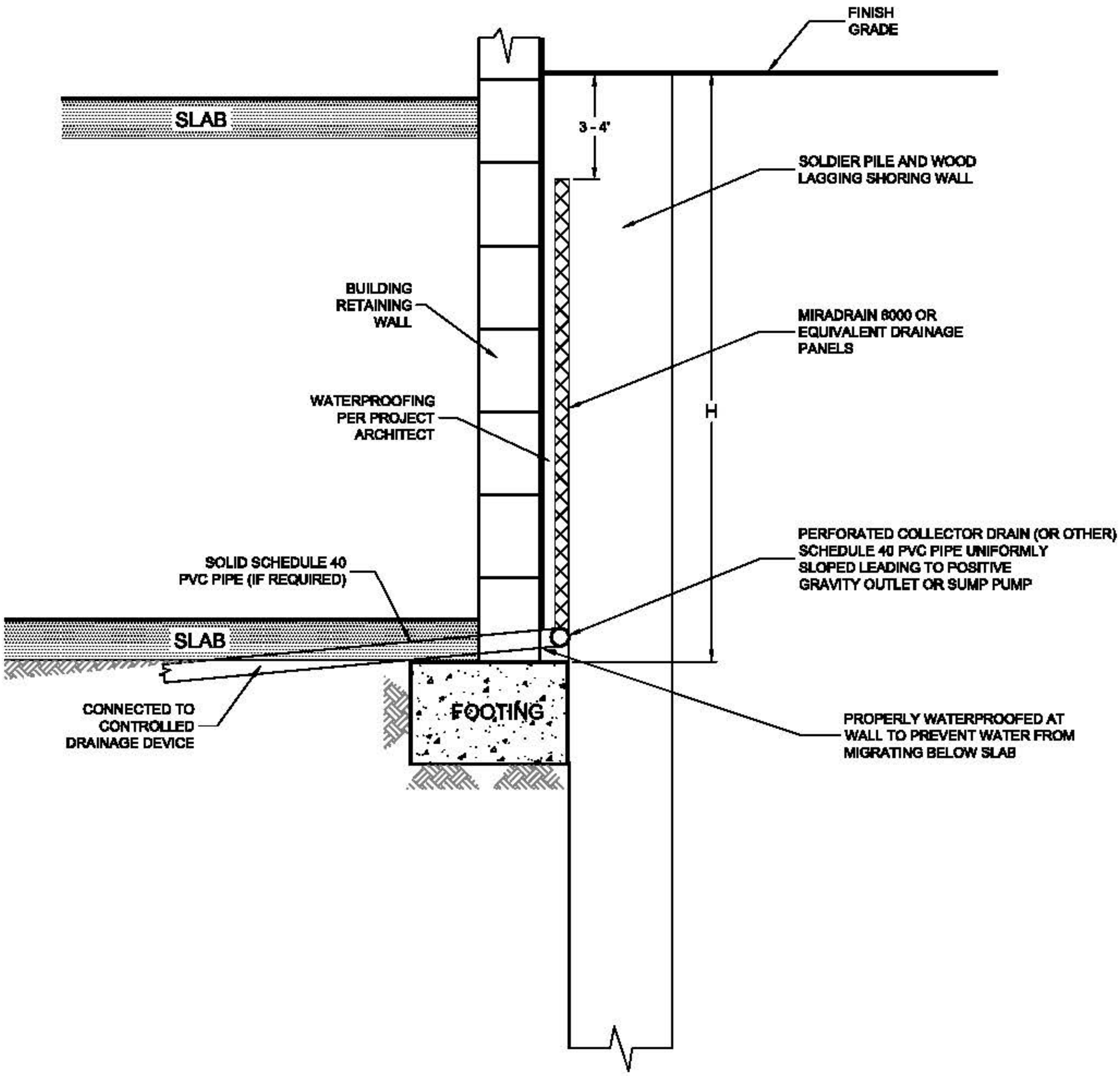
TYPICAL RETAINING WALL DRAIN DETAIL

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 13

Plotted:12/04/2017 4:45PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\DETAILS\Typical Retaining Wall Drainage Detail (RWDD7A).dwg



NO SCALE

SOLDIER PILE WALL DRAINAGE DETAIL

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 14

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on February 22, 2017, that consisted of a visual site reconnaissance, drilling 6 exploratory borings and conducting 4 infiltration tests. The approximate locations of the borings and infiltration tests are shown on the Geologic Map, Figure 2.

The exploratory borings, performed by Pacific Drilling Company, were advanced to depths of 5 to 51½ feet using a Marl M-5 truck-mounted drill rig equipped with 6-inch diameter augers. We obtained samples during our subsurface exploration using a California split-spoon sampler. The sampler is composed of steel and are driven to obtain the soil samples. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 2.875 inches. Up to 18 rings are placed inside the sampler that is 2.4 inches in diameter and 1 inch in height. We obtained ring samples in moisture-tight containers at appropriate intervals and transported them to the laboratory for testing. We also obtained disturbed bulk soil samples from the borings for laboratory testing. The type of sample is noted on the exploratory boring logs.

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

We visually classified and logged the soil encountered in the excavations in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual Manual Procedure D 2488).

PROJECT NO. G2093-52-01

BORING B 1

ELEV. (MSL.) 60' DATE COMPLETED 02-22-2017

EQUIPMENT MARL M-5 BY: M. LOVE

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**3-INCH AC / 3-INCH BASE**			
		SC	**UNDOCUMENTED FILL (Qudf)** Reddish brown, moist, medium dense, Clayey, fine to medium SAND			
2		SM	**OLD PARALIC DEPOSITS (Qop)** Light reddish brown, moist, medium dense, Silty, fine SAND			
4						
6	B1-1			24	99.5	10.5
8						
10	B1-2	SC	Reddish brown and yellowish brown, moist, very dense, Clayey, fine to medium SAND; slight cementation	50/5"	116.1	13.5
12	B1-3					
14						
16	B1-4		-Becomes light brown	50/3"	126.6	11.8
18						
20	B1-5			50/5.5"	101.4	24.7
22						
24						
26	B1-6	ML	Grayish brown, moist, very stiff, fine, Sandy SILT; slight mottling	35	101.0	25.9
	B1-7					
28						

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

G2093-52-01.GPJ

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

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

GEOCON

BORING B 1

ELEV. (MSL.) **60'** DATE COMPLETED **02-22-2017**

EQUIPMENT **MARL M-5** BY: **M. LOVE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1-8			ML	-Becomes laminated	30	83.8	38.5
32								
34								
36	B1-9					36	94.2	31.3
38								
40	B1-10			CL	Dark gray, moist, very stiff, CLAY; laminated, slight mottling	29	91.7	32.7
42								
44								
46	B1-11					40	86.2	36.4
48								
50	B1-12					32	90.6	33.3
					BORING TERMINATED AT 51.5 FEET No groundwater encountered			

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

G2093-52-01.GPJ

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

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

GEOCON

BORING B 2

ELEV. (MSL.) **53'** DATE COMPLETED **02-22-2017**

EQUIPMENT **MARL M-5** BY: **M. LOVE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3-INCH AC / 4-INCH BASE**			
				SP	**OLD PARALIC DEPOSITS (Qop)** Reddish brown, damp, dense, fine to medium SAND; trace gravel			
	B2-1				-Becomes very dense	30/5"	108.5	5.9
10	B2-2			SM	Light brown, damp, very dense, Silty, fine SAND; porous	50/5"	109.6	18.9
	B2-3				-Becomes dense	71	112.8	8.9
	B2-4							
20	B2-5				-Slight oxidation staining	50/4"	107.7	20.8
	B2-6			ML	Gray brown, moist, very stiff, fine Sandy SILT	44	101.4	25.1

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

G2093-52-01.GPJ

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

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

GEOCON

BORING B 2

ELEV. (MSL.) 53' DATE COMPLETED 02-22-2017

EQUIPMENT MARL M-5 BY: M. LOVE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-7			ML		38	95.9	28.5
32								
34								
36								
38								
40	B2-8			CL	Dark olive gray, moist, very stiff, CLAY; laminated, trace sand	42	89.7	32.9
					BORING TERMINATED AT 41.5 FEET No groundwater encountered			

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

G2093-52-01.GPJ

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

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

GEOCON

BORING B 3 (OFF SITE)

ELEV. (MSL.) 50' DATE COMPLETED 02-22-2017

EQUIPMENT MARL M-5 BY: M. LOVE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3-INCH AC / 3-INCH BASE**			
2				CL	**UNDOCUMENTED FILL (Qudf)** Dark reddish brown, moist, stiff, CLAY with gravel			
4	B3-1			SC	**OLD PARALIC DEPOSITS (Qop)** Reddish brown and yellowish brown, moist, Clayey, fine to medium SAND	50/5"	124.7	11.9
					BORING TERMINATED AT 5 FEET No groundwater encountered			

Figure A-3,
Log of Boring B 3 (OFF SITE), Page 1 of 1

G2093-52-01.GPJ

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

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

PROJECT NO. G2093-52-01

BORING B 4 (OFF SITE)

ELEV. (MSL.) **47'** DATE COMPLETED **02-22-2017**

EQUIPMENT **MARL M-5** BY: **M. LOVE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4-INCH AC / 5-INCH BASE**			
2				CL	**OLD PARALIC DEPOSITS (Qop)** Reddish brown, moist, stiff, CLAY; trace sand			
4	B4-1			SC	Reddish brown and yellowish brown, moist, very stiff, fine to medium grained, Clayey SAND	40	119.6	14.8
					BORING TERMINATED AT 5 FEET No groundwater encountered			

Figure A-4,
Log of Boring B 4 (OFF SITE), Page 1 of 1

G2093-52-01.GPJ

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

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

GEOCON

BORING B 5

ELEV. (MSL.) **54'** DATE COMPLETED **02-22-2017**

EQUIPMENT **MARL M-5** BY: **M. LOVE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4-INCH AC / 5-INCH BASE**			
				SM	**OLD PARALIC DEPOSITS (Qop)** Yellowish brown, damp, very dense, Silty, fine to medium SAND; trace gravel			
2								
	B5-1					81	119.1	12.6
4				SP	Light reddish brown, damp, medium dense, fine to medium SAND			
	B5-2					19	112.8	5.4
6								
8	B5-3					26	114.8	7.0
					BORING TERMINATED AT 8.5 FEET No groundwater encountered			

Figure A-5,
Log of Boring B 5, Page 1 of 1

G2093-52-01.GPJ

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

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

GEOCON

BORING B 6

ELEV. (MSL.) **54'** DATE COMPLETED **02-22-2017**

EQUIPMENT **MARL M-5** BY: **M. LOVE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3-INCH AC / 5-INCH BASE**			
				SM	**OLD PARALIC DEPOSITS (Qop)** Light reddish brown, moist, very dense, Silty, fine to medium SAND; slight lamination			
2								
	B6-1					50/5"	118.1	6.6
4	B6-2							
	B6-3					50/4"	118.8	8.6
6					BORING TERMINATED AT 6 FEET No groundwater encountered			

Figure A-6,
Log of Boring B 6, Page 1 of 1

G2093-52-01.GPJ

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

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

GEOCON

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current and generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We selected samples to test for in-place density and moisture content, shear strength, expansion potential, water-soluble sulfate content, R-Value, gradation, and consolidation characteristics. The results of our laboratory tests are summarized on Tables B-I through B-V and Figures B-1 through B-5 and on the boring logs in Appendix A.

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

Sample No.	Dry Density (pcf)	Moisture Content (%)	Peak [Ultimate[1]] Cohesion (psf)	Peak [Ultimate[1]] Angle of Shear Resistance (degrees)
B1-2	116.1	13.5	34 [31]	950 [600]
B1-6	101.0	25.9	26 [26]	900 [650]

[1] Ultimate at end of test at 0.2-inch deflection.

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

Sample No.	Geologic Unit	Moisture Content (%)		Dry Density (pcf)	Expansion Index	ASTM Soil Expansion Classification	2016 CBC Expansion Classification
		Before Test	After Test				
B1-3	Qop	7.0	11.9	123.2	0	Very Low	Non-Expansive
B1-7	Qop	9.5	16.8	111.9	14	Very Low	Non-Expansive

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

Sample No.	Depth (Feet)	Description (Geologic Unit)	R-Value
B6-2	3-5	Light reddish brown, Silty SAND (Qop)	6

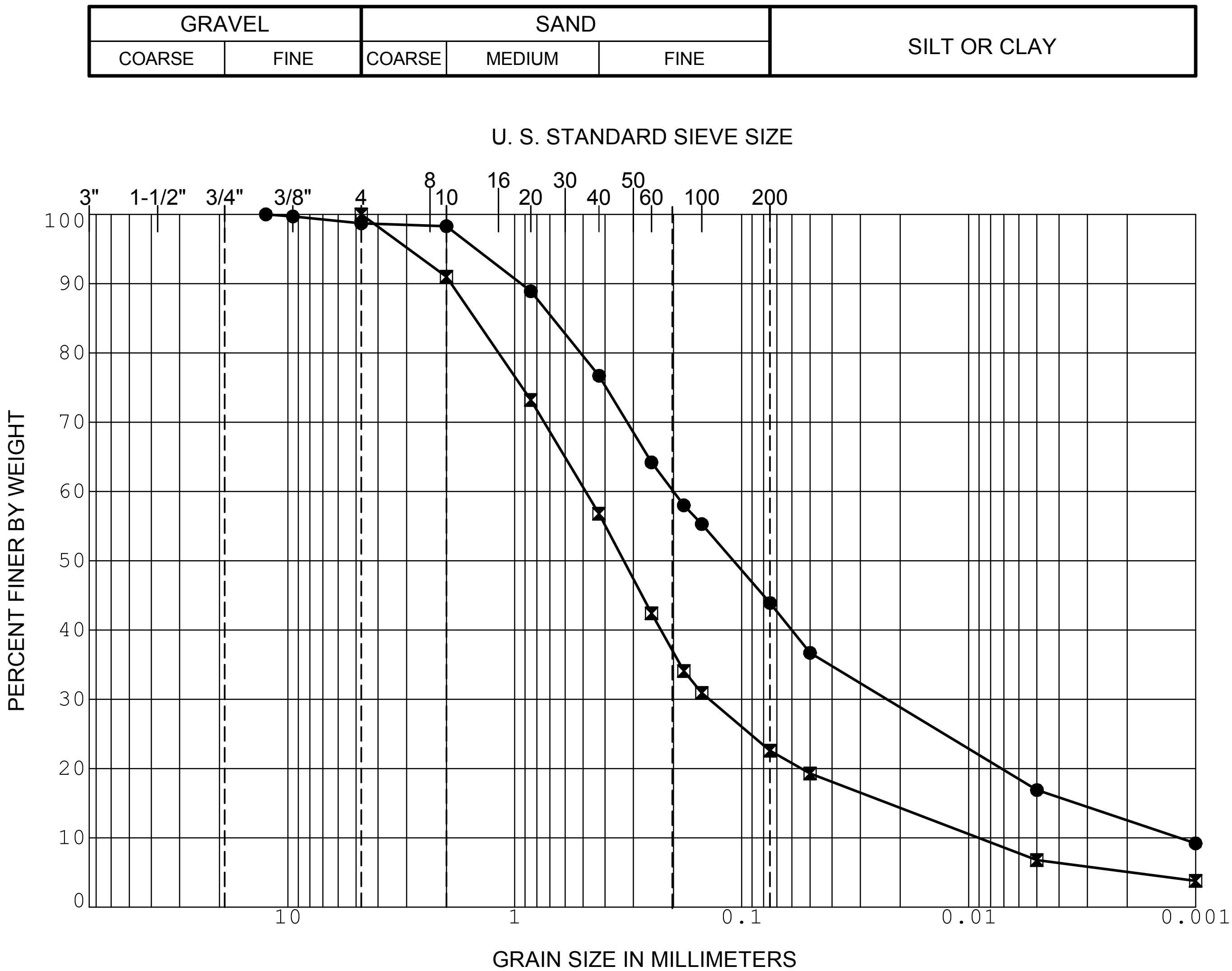
TABLE B-IV
SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (%)	ACI 318-14 Sulfate Class
B1-7	0.009	S0

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

Sample No.	Depth (feet)	Geologic Unit	Hand Penetrometer Reading, Unconfined Compression Strength (tsf)	Undrained Shear Strength (ksf)
B1-1	6	Qop	3.5	3.5
B1-4	16	Qop	4.5+	4.5+
B1-5	21	Qop	3.5	3.5
B1-9	36	Qop	4.0	4.0
B1-10	41	Qop	4.0	4.0
B1-11	46	Qop	3.5	3.5
B1-12	51	Qop	3.0	3.0
B2-1	5	Qop	4.5+	4.5+
B2-5	21	Qop	4.5+	4.5+
B2-6	26	Qop	4.5+	4.5+
B2-7	31	Qop	4.5+	4.5+
B2-8	41	Qop	4.5	4.5
B3-1	5	Qop	4.5+	4.5+
B5-1	3	Qop	4.5+	4.5+
B5-2	6	Qop	3.0	3.0
B5-3	8	Qop	4.0	4.0
B6-1	3	Qop	4.5+	4.5+

PROJECT NO. G2093-52-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B4-1	4.0	SC - Clayey SAND				
⊠	B6-3	5.0	SM - Silty SAND				
▲							

GRADATION CURVE

U STOR IT BURRIO LOGAN

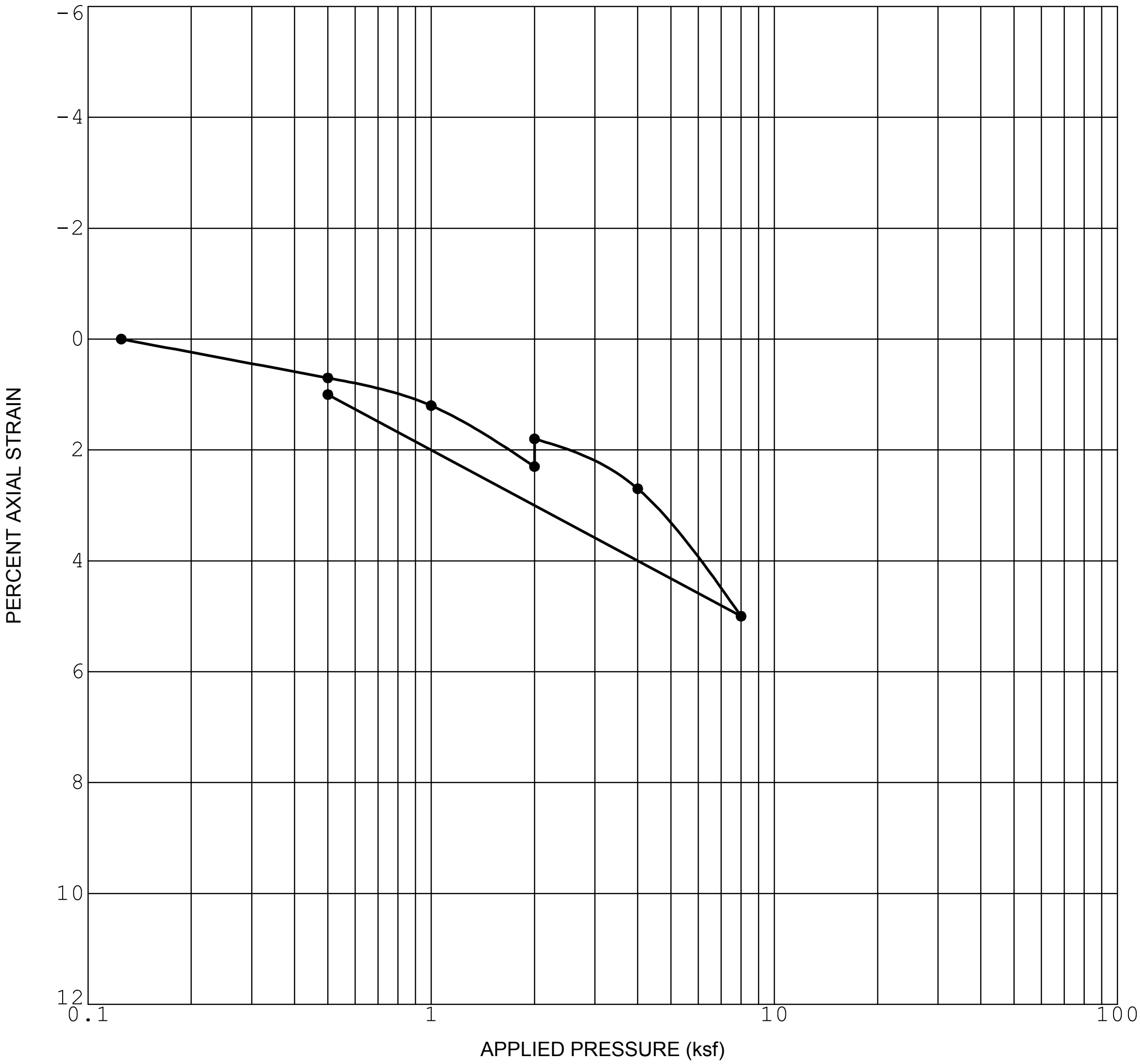
SAN DIEGO, CALIFORNIA

G2093-52-01.GPJ

Figure B-1

GEOCON

SAMPLE NO. B1-8



Initial Dry Density (pcf)	83.8
Initial Water Content (%)	38.5

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

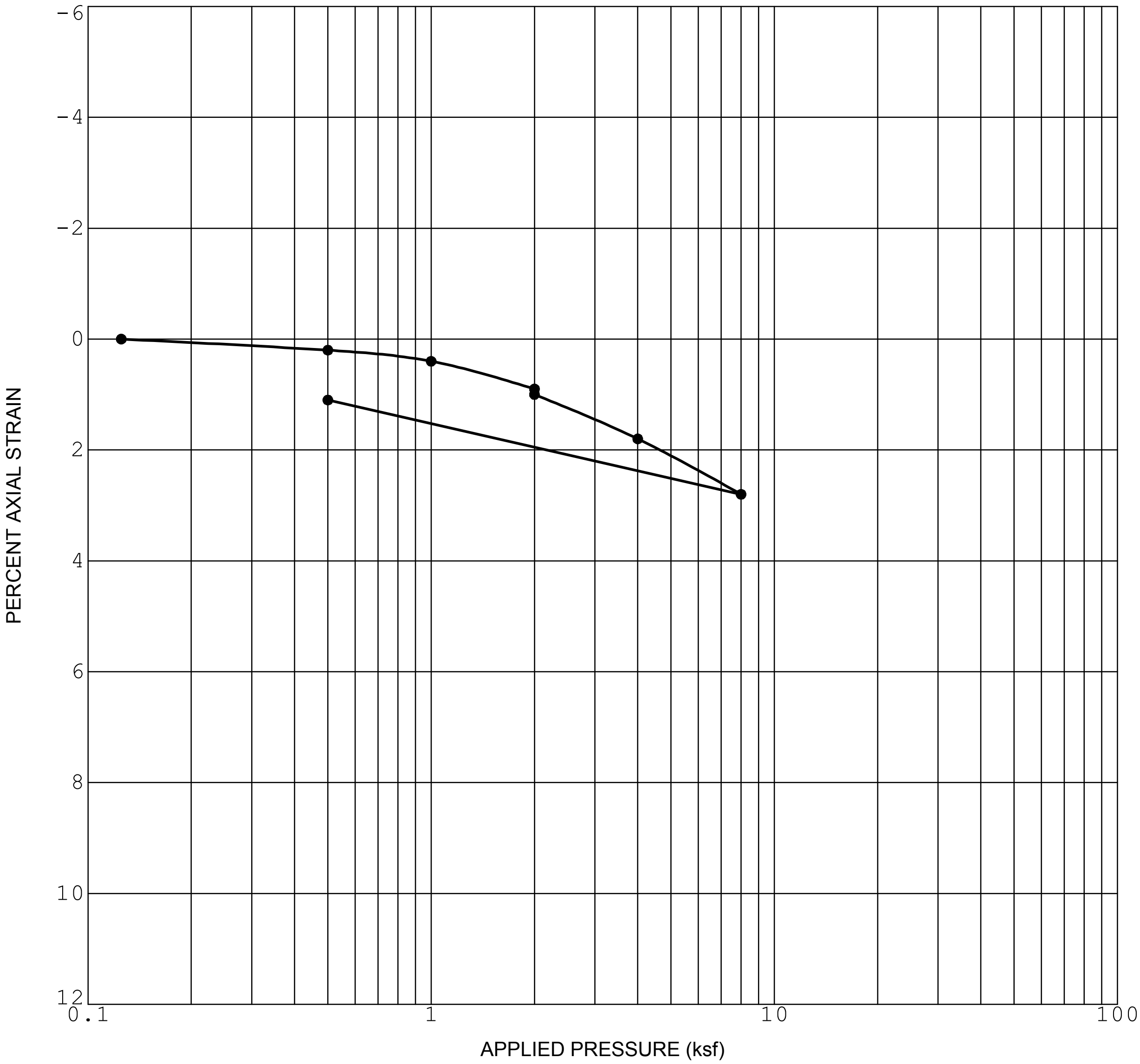
CONSOLIDATION CURVE

U STOR IT BURRIO LOGAN

SAN DIEGO, CALIFORNIA

PROJECT NO. G2093-52-01

SAMPLE NO. B2-2



Initial Dry Density (pcf)	109.6		**Initial Saturation (%)**	97.5
Initial Water Content (%)	18.9		**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

U STOR IT BURRIO LOGAN

SAN DIEGO, CALIFORNIA

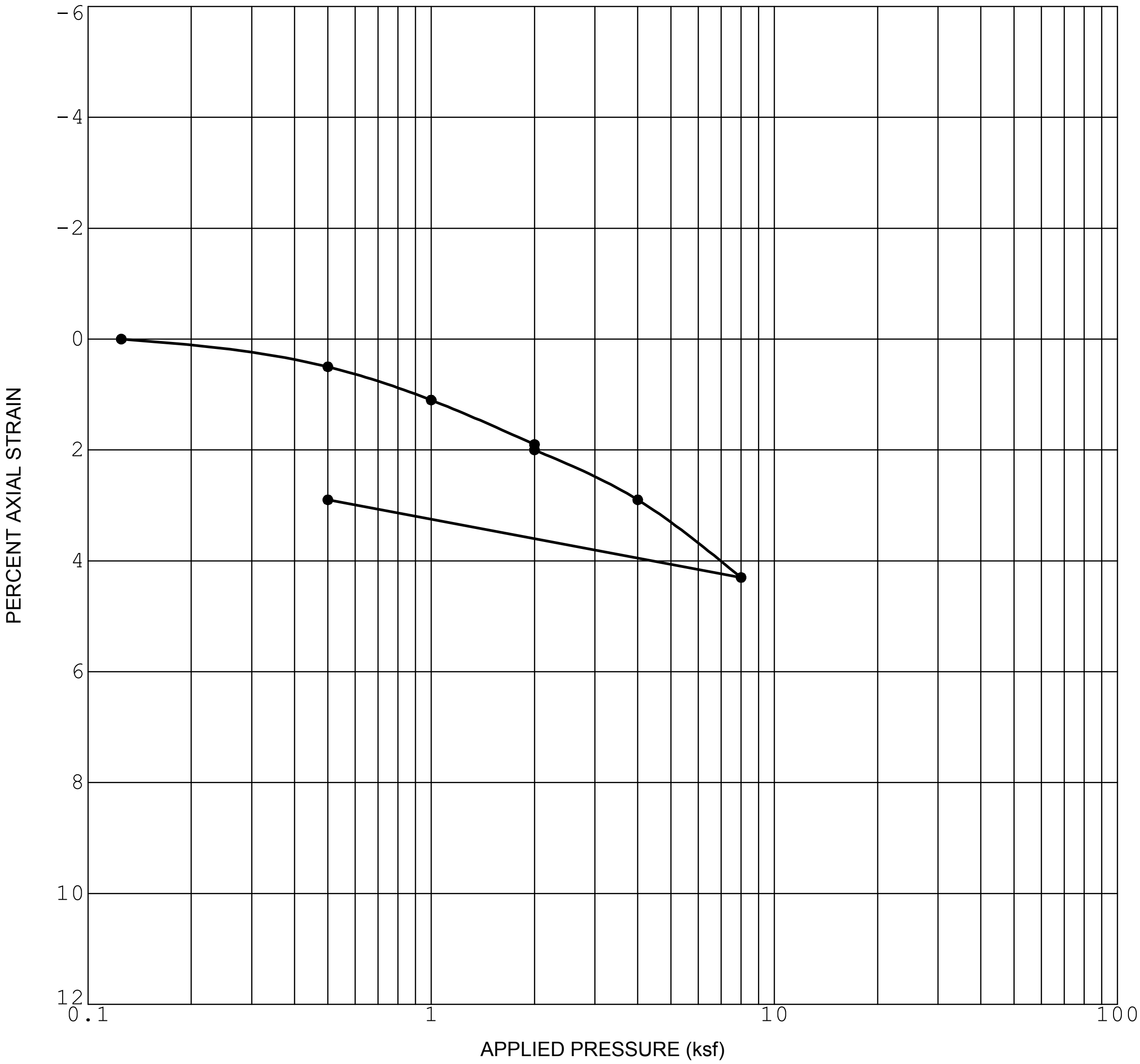
G2093-52-01.GPJ

Figure B-3

GEOCON

PROJECT NO. G2093-52-01

SAMPLE NO. B4-1



Initial Dry Density (pcf)	119.6
Initial Water Content (%)	14.8

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

CONSOLIDATION CURVE

U STOR IT BURRIO LOGAN

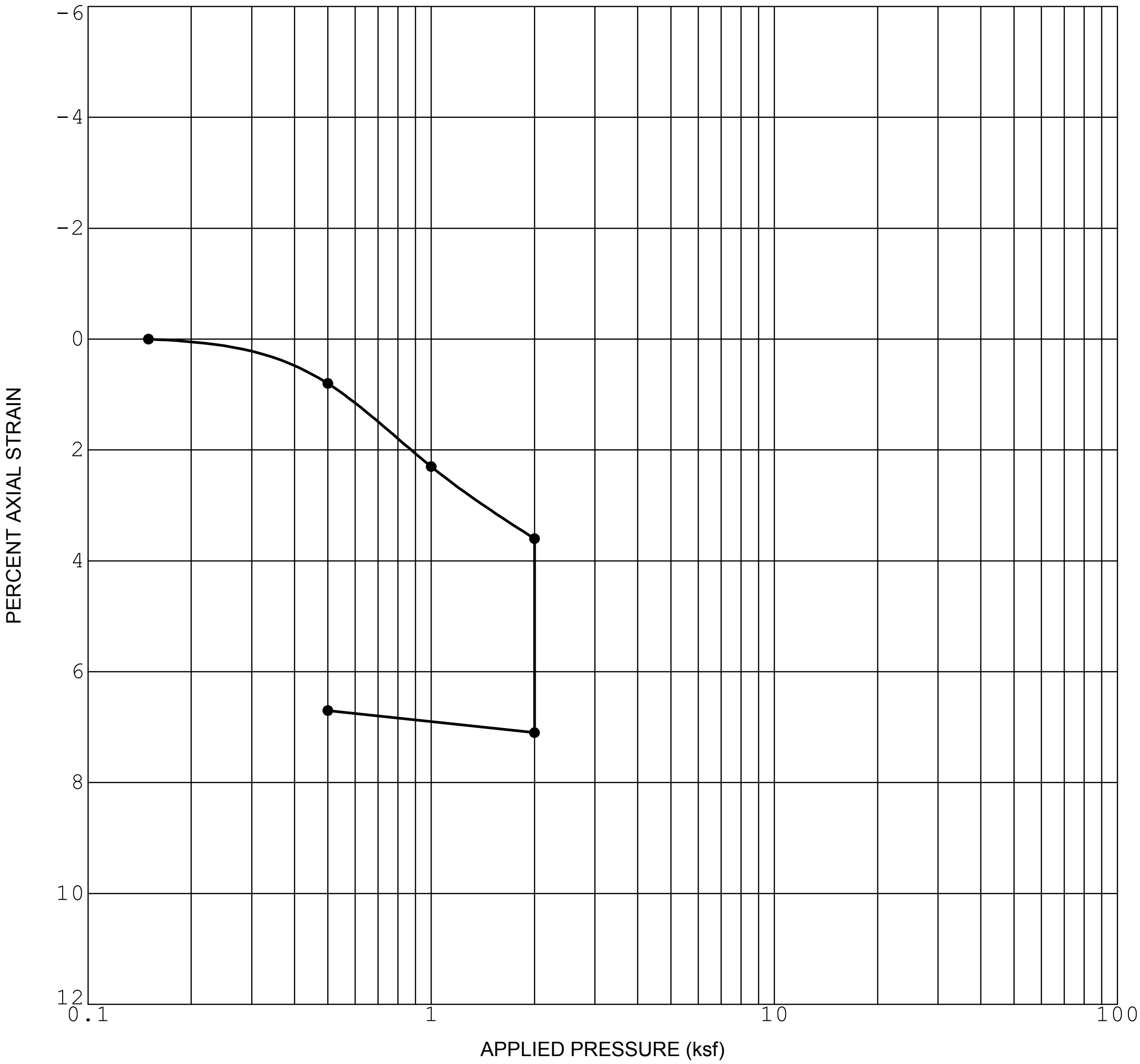
SAN DIEGO, CALIFORNIA

G2093-52-01.GPJ

Figure B-4

GEOCON

SAMPLE NO. B6-3



Initial Dry Density (pcf)	118.8	**Initial Saturation (%)**	57.4
Initial Water Content (%)	8.6	**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

U STOR IT BURRIO LOGAN

SAN DIEGO, CALIFORNIA

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

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 hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

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

TABLE C-I
HYDROLOGIC SOIL GROUP DEFINITIONS

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

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

In Situ Testing

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

TABLE C-II
SOIL PERMEABILITY DEFINITIONS

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

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

We performed 2 Aardvark Permeameter tests at the locations shown on the attached Geologic Map, Figure 2. The test borings were 6-inches in diameter. The results of the tests provide parameters regarding the saturated hydraulic conductivity and infiltration characteristics of on-site soil and geologic units. Table C-III presents the results of the estimated field saturated hydraulic conductivity and estimated infiltration rates obtained from the Aardvark Permeameter tests. The field sheets are also attached herein. The designer of storm water devices should apply an appropriate factor of safety. Soil infiltration rates from in-situ tests can vary significantly from one location to another due to the heterogeneous characteristics inherent to most soil. Based on a discussion in the County of Riverside *Design Handbook for Low Impact Development Best Management Practices*, the infiltration rate should be considered equal to the saturated hydraulic conductivity rate.

TABLE C-III
FIELD PERMEAMETER INFILTRATION TEST RESULTS

Test No. [1]	Geologic Unit	Test Elevation (feet MSL)	Field-Saturated Infiltration Rate (inch/hour)	Worksheet Infiltration Rate[2] (inch/hour)
P-3	Qop	49	0.024	0.012
P-4	Qop	50	0.002	0.001
		Average:	0.013	0.007

1 Infiltration tests P-1 and P-2 were performed outside of the project limits and have not been taken into consideration for this assessment. The field sheets for tests P-1 and P-2 are included herein for reference only.
1 Using a Factor of Safety of 2.0.

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

TABLE C-IV
INFILTRATION CATEGORIES

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

*Using a Factor of Safety of 2.

STORM WATER MANAGEMENT CONCLUSIONS

The Geologic Map, Figure 2, depicts the existing property, the approximate lateral limits of the geologic units, the locations of the field excavations and the in-situ infiltration test locations. The following presents a discussion of the soil types on site regarding storm water infiltration feasibility.

Soil Types

Undocumented Fill (Qudf) – Undocumented fill is present across the site. The undocumented fill was not tested or observed during placement and should be considered highly variable. Water that is allowed to migrate within the undocumented fill soil cannot be controlled due to lateral migration potential, would destabilize support for the existing improvements, and would shrink and swell. Therefore, full and partial infiltration should be considered infeasible within the undocumented fill. We anticipate that the undocumented fill will be completely removed during excavations for the proposed subterranean levels.

Old Paralic Deposits – The surficial soils on the property are underlain by Old Paralic Deposits. Based on the boring logs, laboratory tests and our observations, the Old Paralic Deposits are highly variable due to the sedimentary nature of the materials. The Old Paralic Deposits have a greater propensity for lateral water migration over vertical water migration. The infiltration rates within the Old Paralic Deposits are considered to be very low due to the dense nature of the materials. In addition, the Old Paralic Deposits possess hydroconsolidation potential as discussed herein. As a result, full and partial infiltration should be considered infeasible.

Compacted Fill – We expect that compacted fill, if any, will be comprised of on-site materials that will consist predominantly of silty and clayey sand. The fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density. In our experience, compacted fill using the on-site materials does not possess infiltration rates appropriate with infiltration and the water would destabilize the existing fill causing distress to existing and proposed improvements. The intent of the compacted fill is to support structures and infrastructure (utilities, pavement, and flatwork). Therefore, full and partial infiltration should be considered infeasible.

Infiltration Rates

The results of the infiltration rates within the Old Paralic Deposits ranges from 0.002 to 0.024 inches per hour with an average of 0.013 inches per hour (average of 0.007 inches per hour including a factor of safety of 2.0). Therefore, based on the results of the field infiltration tests, the laboratory tests and our experience, full and partial infiltration should be considered infeasible within the Old Paralic Deposits. Mitigation for very low infiltration rates does not exist.

Groundwater Elevations

We did not encounter groundwater during the drilling operations at the property to the maximum depth of 50 feet or an elevation of about 10 feet MSL. We expect groundwater is present at an elevation of 0 to 5 feet MSL. The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. Therefore, infiltration would be considered feasible above an elevation of 15 feet MSL.

New or Existing Utilities

Utilities are located adjacent to the property on the northern, western, and southern property boundaries and existing streets. Therefore, full infiltration near these utilities should be considered infeasible within these areas. The setback for infiltration devices would be a minimum of a 1:1 plane from 5 feet outside the invert of the deepest adjacent utility. Mitigation measures to prevent water from infiltrating the utilities consist of installing cutoff walls around the utilities and installing subdrains and/or installing liners. Liners would be the preferred option because of the potential for lateral migration within the Old Paralic Deposits.

Soil or Groundwater Contamination

We are unaware of contaminated soil or groundwater on the property. Therefore, infiltration associated with this risk is considered feasible. We should be contacted if contaminated soil exists on the property.

Slopes and Other Geologic Hazards

Slopes are not currently planned or exist on the property that would be affected by potential infiltration locations. As discussed herein, the Old Paralic Deposits possess a hydroconsolidation potential ranging from 0.1 to 3.5 percent. We expect the upper 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation up to about 4¼ inches. Therefore, infiltration in regards the geologic hazards would be considered infeasible.

Existing and Planned Structures

Existing structures are located along the western, eastern and southern property lines. If water is allowed to infiltrate into the soil, the water could migrate laterally and into other properties in the vicinity of the subject site and negatively affect other buildings and improvements in the area (e.g. saturating soil adjacent to existing foundations). Therefore, infiltration near these structures or any other proposed structures should be considered infeasible within these areas, and setbacks for infiltration should be incorporated. Mitigation for existing structures consists of not allowing water infiltration within a 1:1 plane from 20 feet below the existing foundations.

Storm Water Management Devices

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

Storm Water Standard Worksheets

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

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

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

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

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

TABLE C-VI
FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A[1]

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

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

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition</th><th colspan="2">Worksheet C.4-1</th></tr>
<tr><td colspan="4"><u>Part 1 - Full Infiltration Feasibility Screening Criteria</u>
Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>1</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
We encountered field infiltration rates of:
P-3: 0.024 inches/hour (0.012 with a FOS of 2.0)
P-4: 0.002 inches/hour (0.001 with a FOS of 2.0)

These tests results in an average of about 0.013 inches/hour (0.007 with a FOS of 2.0).

The results of the infiltration tests indicate rates of less than 0.5 inches per hour (including the factor of safety); therefore, infiltration is not considered 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>2</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Undocumented fill and Old Paralic Deposits underlie the property. Water that would be allowed to infiltrate would migrate laterally outside of the property limits to the existing right-of-ways (located to the south) and toward existing and proposed structures (located to the north and west). The Old Paralic Deposits possess hydroconsolidation potential ranging from 0.1 to 3 percent. We expect the upper 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation is up to about 4¼ inches. Therefore, infiltration in regards the geologic hazards would be considered infeasible. Liners and subdrains should be incorporated into the design and construction of the planned storm water devices to prevent saturation and potential hydroconsolidation of the soil supporting the existing or proposed development.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not encounter groundwater during the drilling operation at the property to the maximum depth of 50 feet or an elevation of 10 feet MSL. Groundwater is anticipated to be present at an elevation of 0 to 5 feet MSL. The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. Therefore, infiltration due to groundwater elevations would be considered feasible above an elevation of 15 feet MSL. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We do not expect full infiltration would cause water balance issues including change of ephemeral streams or discharge of contaminated water to surface waters. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2	Not Full Infiltration	

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

Worksheet C.4-1 Page 3 of 4

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 encountered field infiltration rates of:

P-3: 0.024 inches/hour (0.012 with a FOS of 2.0)

P-4: 0.002 inches/hour (0.001 with a FOS of 2.0)

These tests results in an average of about 0.013 inches/hour (0.007 with a FOS of 2.0).

The results of the infiltration tests indicate rates of less than 0.05 inches per hour (including the factor of safety); therefore, infiltration is not considered feasible.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

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:

Undocumented fill and Old Paralic Deposits underlie the property. Water that would be allowed to infiltrate could migrate laterally outside of the property limits to the existing right-of-ways (located to the south) and toward existing and proposed structures (located to the north and west). The Old Paralic Deposits possess hydroconsolidation potential ranging from 0.1 to 3 percent. We expect the upper 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation is up to about 4¼ inches. Therefore, infiltration in regards the geologic hazards would be considered infeasible. Liners and subdrains should be incorporated into the design and construction of the planned storm water devices to prevent saturation and potential hydroconsolidation of the soil supporting the existing or proposed development.

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

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not encounter groundwater during the drilling operation at the property to the maximum depth of 50 feet or an elevation of 10 feet MSL. Groundwater is anticipated to be present at an elevation of 0 to 5 feet MSL. The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. Therefore, infiltration due to groundwater elevations would be considered feasible above an elevation of 15 feet MSL. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not provide a study regarding water rights. However, these rights are not typical in the San Diego County area. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	No Infiltration	

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

GEOCON

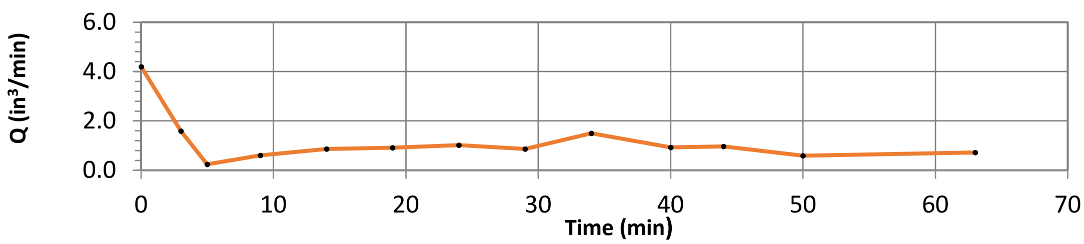
Aardvark Permeameter Data Analysis

Project Name:	2209 National Ave.	Date:	2/22/2017	
Project Number:	G2093-52-01	By:	JML	
Test Number:	P-3	Ref. EL (feet, MSL):		54.0
		Bottom EL (feet, MSL):		49.1

Borehole Diameter, d (in.):	6.00
Borehole Depth, **H** (in):	59.00
Distance Between Reservoir & Top of Borehole (in.):	29.50
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	80.25
Head Height Calculated, **h** (in.):	4.77
Head Height Measured, **h** (in.):	4.00
Distance Between Constant Head and Water Table, **L** (in.):	1145.00

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	3.00	0.455	12.60	4.200
3	2.00	0.115	3.18	1.592
4	4.00	0.035	0.97	0.242
5	5.00	0.110	3.05	0.609
6	5.00	0.155	4.29	0.858
7	5.00	0.165	4.57	0.914
8	5.00	0.185	5.12	1.025
9	5.00	0.155	4.29	0.858
10	6.00	0.325	9.00	1.500
11	4.00	0.135	3.74	0.935
12	6.00	0.210	5.82	0.969
13	13.00	0.275	7.62	0.586
14	17.00	0.440	12.18	0.717
			Steady Flow Rate, Q (in^3/min):	0.757



Soil Matric Flux Potential, Φ_m

Φ_m= 0.016 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 4.03E-04 in/min 0.024 in/hr

GEOCON

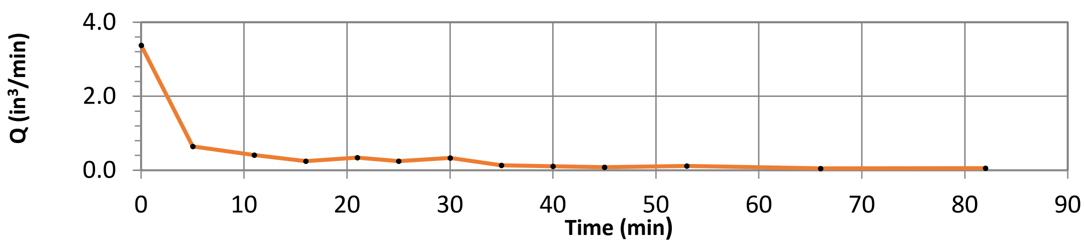
Aardvark Permeameter Data Analysis

Project Name:	2209 National Ave.	Date:	2/22/2017
Project Number:	G2093-52-01	By:	JML
Test Number:	P-4	Ref. EL (feet, MSL):	54.0
		Bottom EL (feet, MSL):	50.0

Borehole Diameter, d (in.):	6.00
Borehole Depth, **H** (in):	48.00
Distance Between Reservoir & Top of Borehole (in.):	29.00
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	68.75
Head Height Calculated, **h** (in.):	4.73
Head Height Measured, **h** (in.):	4.50
Distance Between Constant Head and Water Table, **L** (in.):	1156.50

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.610	16.89	3.378
3	6.00	0.140	3.88	0.646
4	5.00	0.075	2.08	0.415
5	5.00	0.045	1.25	0.249
6	4.00	0.050	1.38	0.346
7	5.00	0.045	1.25	0.249
8	5.00	0.060	1.66	0.332
9	5.00	0.025	0.69	0.138
10	5.00	0.020	0.55	0.111
11	8.00	0.025	0.69	0.087
12	13.00	0.055	1.52	0.117
13	16.00	0.030	0.83	0.052
14	25.00	0.050	1.38	0.055
			Steady Flow Rate, Q (in^3/min):	0.055



Soil Matric Flux Potential, Φ_m

Φ_m= 0.001098419 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.79E-05 in/min 0.002 in/hr

GEOCON

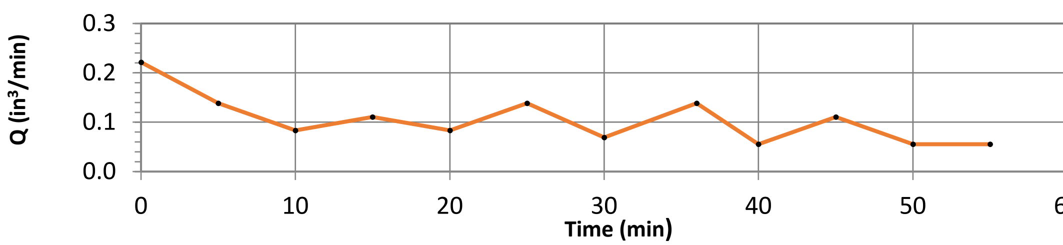
Aardvark Permeameter Data Analysis

Project Name:	2209 National Ave.	Date:	2/22/2017
Project Number:	G2093-52-01	By:	JML
Test Number:	P-1	Ref. EL (feet, MSL):	50.0
		Bottom EL (feet, MSL):	45.4

Borehole Diameter, d (in.):	8.00
Borehole Depth, **H** (in):	55.00
Distance Between Reservoir & Top of Borehole (in.):	30.50
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	77.25
Head Height Calculated, **h** (in.):	4.76
Head Height Measured, **h** (in.):	5.25
Distance Between Constant Head and Water Table, **L** (in.):	1150.25

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.040	1.11	0.222
3	5.00	0.025	0.69	0.138
4	5.00	0.015	0.42	0.083
5	5.00	0.020	0.55	0.111
6	5.00	0.015	0.42	0.083
7	5.00	0.025	0.69	0.138
8	6.00	0.015	0.42	0.069
9	4.00	0.020	0.55	0.138
10	5.00	0.010	0.28	0.055
11	5.00	0.020	0.55	0.111
12	5.00	0.010	0.28	0.055
13	5.00	0.010	0.28	0.055
			Steady Flow Rate, Q (in^3/min):	0.055



Soil Matric Flux Potential, Φ_m

Φ_m= 0.00101035 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.57E-05 in/min 0.002 in/hr

GEOCON

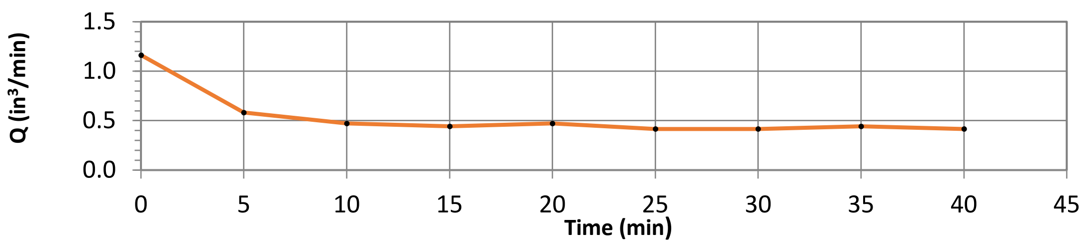
Aardvark Permeameter Data Analysis

Project Name:	2209 National Ave.	Date:	2/22/2017
Project Number:	G2093-52-01	By:	JML
Test Number:	(Offsite) P-2	Ref. EL (feet, MSL):	47.0
		Bottom EL (feet, MSL):	42.8

Borehole Diameter, d (in.):	6.00
Borehole Depth, **H** (in):	50.00
Distance Between Reservoir & Top of Borehole (in.):	29.00
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	70.75
Head Height Calculated, **h** (in.):	4.74
Head Height Measured, **h** (in.):	4.00
Distance Between Constant Head and Water Table, **L** (in.):	1154.00

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.210	5.82	1.163
3	5.00	0.105	2.91	0.582
4	5.00	0.085	2.35	0.471
5	5.00	0.080	2.22	0.443
6	5.00	0.085	2.35	0.471
7	5.00	0.075	2.08	0.415
8	5.00	0.075	2.08	0.415
9	5.00	0.080	2.22	0.443
10	5.00	0.075	2.08	0.415
11	6.00	0.080	2.22	0.369
12	4.00	0.060	1.66	0.415
			Steady Flow Rate, Q (in^3/min):	0.415



Soil Matric Flux Potential, Φ_m

Φ_m= 0.009 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.21E-04 in/min 0.013 in/hr

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2093-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

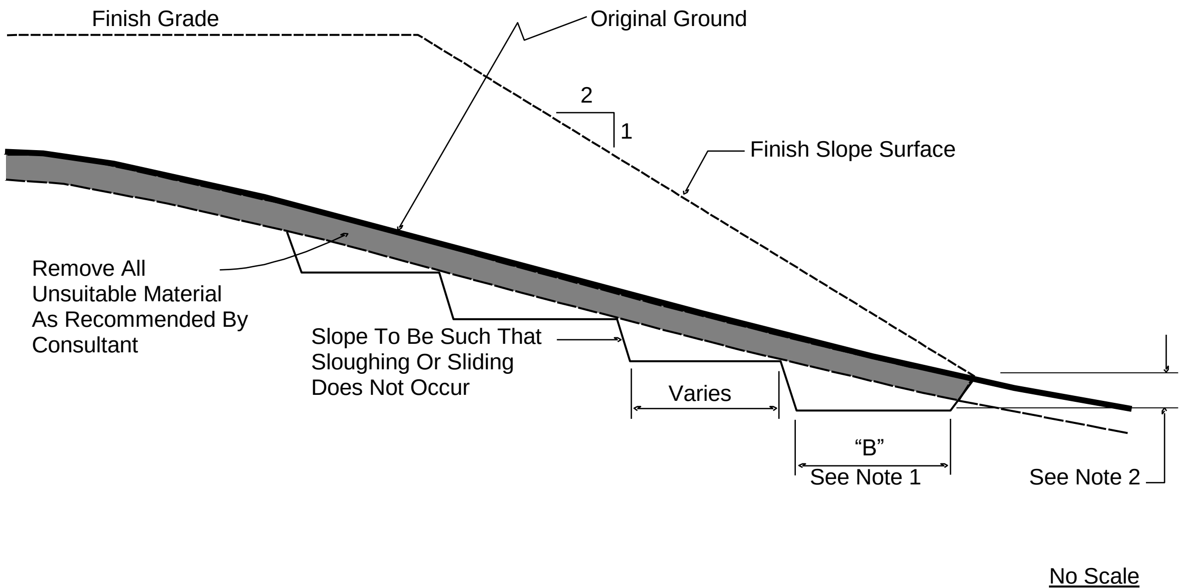
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

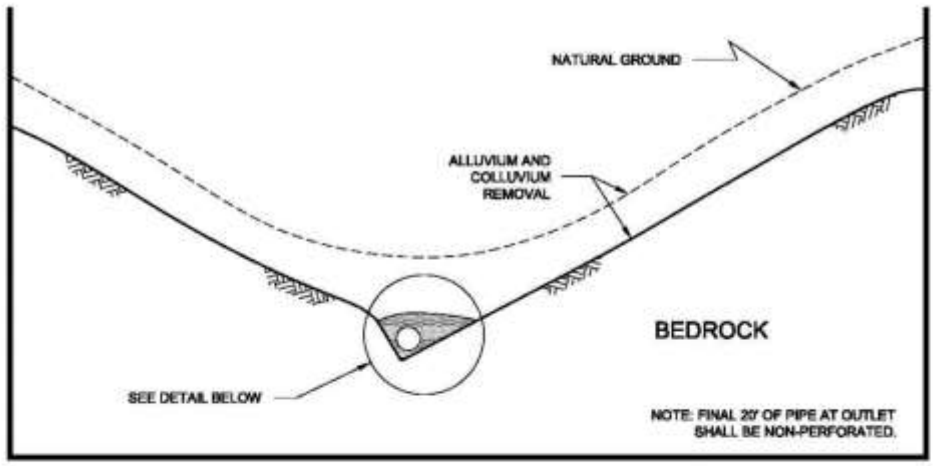
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

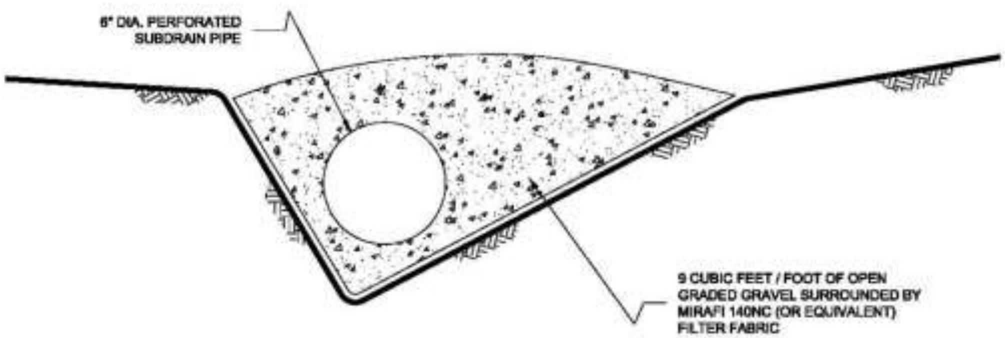
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

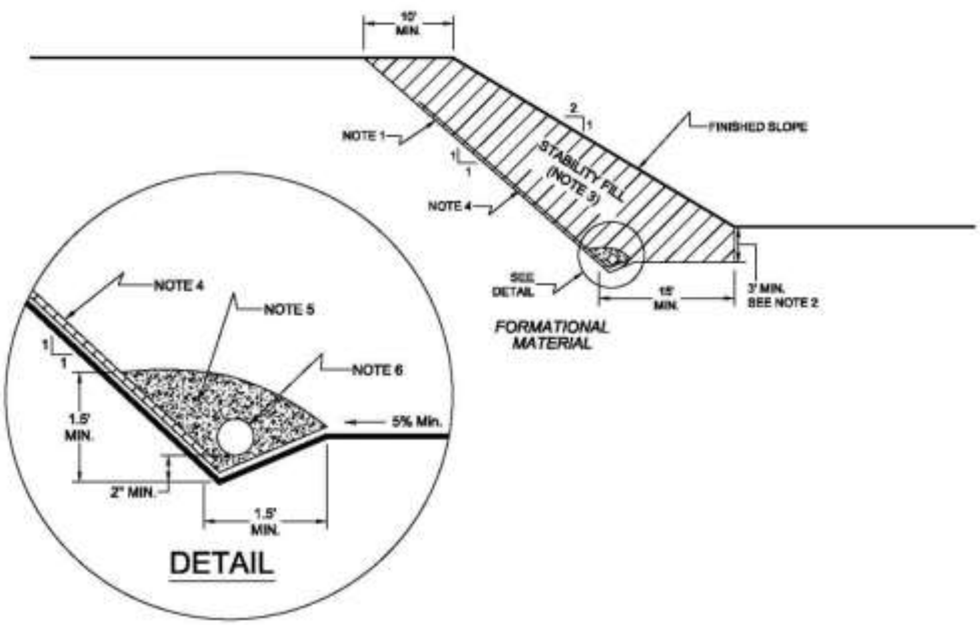
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

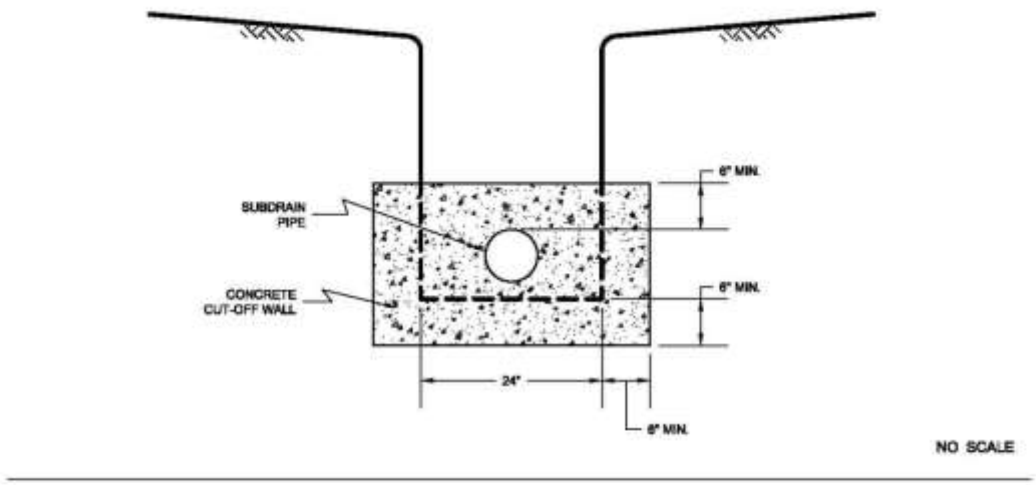
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

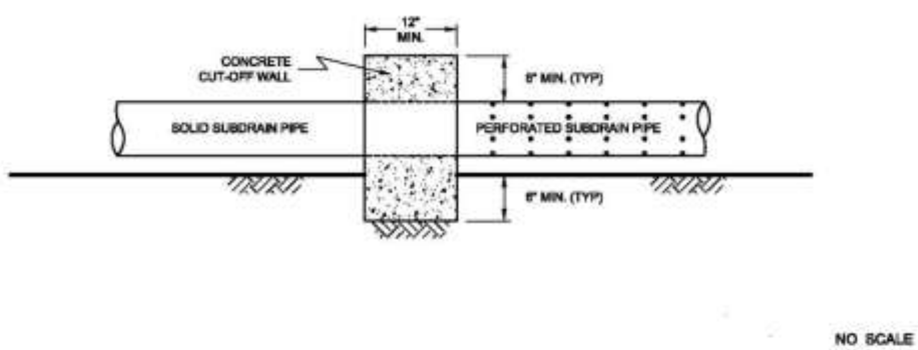
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



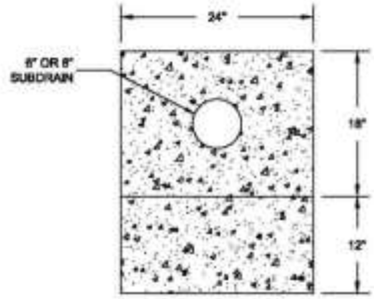
SIDE VIEW



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

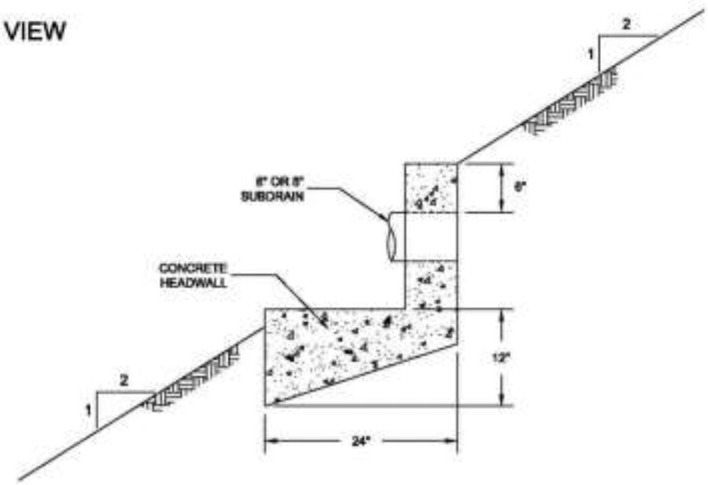
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



NOTE: HEADWALL SHOULD OUTLET AT TOE OF FILL SLOPE
OR INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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

LIST OF REFERENCES

1. *2016 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2015 International Building Code*, prepared by California Building Standards Commission, dated July 1, 2016.

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

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

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

5. Bryant, W. A., and E. W. Hart, 2007, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps,* California Division of Mines and Geology Special Publication 42, interim revision.

6. California Department of Water Resources, Water Data Library. http://www.water.ca.gov/waterdatalibrary.

7. California Division of Mines and Geology, *Landslide Hazards in the Southern Part of the San Diego Metropolitan Area, Point Loma Quadrangle (Plate D), San Diego County, California*, dated 1995.

8. California Emergency Management Agency (CEMA, CGS and USC), *Tsunami Inundation Map for Emergency Planning, Point Loma Quadrangle,* dated June 1, 2009.

9. California Geological Survey, 2003, *State of California Earthquake Fault Zones, Point Loma Quadrangle*, dated May 1, 2003.

10. California Geological Survey, 2002, *Guidelines for Evaluating the Hazard of Surface Fault Rupture*, CGS Note 49.

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

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

LIST OF REFERENCES (Continued)

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

14. City of San Diego 2011, *Guidelines for Geotechnical Reports*.

15. *City of San Diego Seismic Safety Study, Geologic Hazards and Faults*, 2008, Map Sheets 13 and 17.

16. County of San Diego, *San Diego County Multi Jurisdiction Hazard Mitigation Plan, San Diego, California* – Final Draft, July, 2010.

17. Geocon Incorporated, 1993, *Geotechnical Engineering Investigation for Mercado Apartments, San Diego, California,* dated January 1993, (Project No. 04749-31-02).

18. Geocon Incorporated, 1997, *Limited Geotechnical Investigation, Southwest Marine Parking Lot, Main Street And Sampson Street, San Diego, California*, dated December 1997 (Project No. 06023-22-01).

19. Geocon Incorporated, 2000, *Fault Hazard Study, Kelco Plant, 2025 Harbor Drive, San Diego, California,* dated August 2000 (Project No. 06155-22-06).

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

21. Kennedy, M. P., and S. H. Clarke, 1999, *Age of Faulting in San Diego Bay in the Vicinity of the Coronado Bridge,* California Division of Mines and Geology, Open File Report 97-10A.

22. Kennedy, M. P., and S. H. Clarke, 2001, *Late Quaternary Faulting in San Diego Bay and Hazard to the Coronado Bridge*, in California Geology, July/August 2001.

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

24. Munsell® Soil Color Charts," GretagMacbeth, New Windsor, NY, 1998.

25. Legg, M. R., J. C. Borrero, and C. E. Synolakis (2002), *Evaluation of Tsunami Risk to Southern California Coastal Cities*, 2002 NEHRP Professional Fellowship Report, dated January.

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

LIST OF REFERENCES (Concluded)

27. Omega Engineering, *Conceptual Grading Plans, U STOR IT Self Storage, 2209 National Avenue, San Diego, California*, dated October 4, 2017 (Project No. 117138).

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

29. SCS Engineers, *Phase I Environmental Site Assessment, 3350 El Cajon Boulevard, San Diego, California*, dated February 8, 2017 (Project No. 01217029.00).

30. Treiman, Jerome A., *The Rose Canyon Fault Zone Southern California*, California Division of Mines and Geology Publication, 1993.

31. Treiman, J. A, 2002, *Silver Strand Fault, Coronado Fault, Spanish Bight Fault, San Diego Fault, and Downtown Graben Fault, Southern Rose Canyon Fault Zone, San Diego, California*, dated July 17, 2002, California Division of Mines and Geology Fault Evaluation Report FER-245.

32. Treiman, J. A., 2003, *Silver Strand Fault, Coronado Fault, Spanish Bight Fault, San Diego Fault, and Downtown Graben Fault, Southern Rose Canyon Fault Zone, San Diego, California*, dated April 22, 2003, California Division of Mines and Geology Fault Evaluation Report FER-245, Supplement No. 1.

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

34. United States Geological Survey computer program, *U.S. Seismic Design Maps*, http://earthquake.usgs.gov/designmaps/us/application.php.

35. United States Geological Survey, 1934, Point Loma Quadrangle, California, 6.0-Minute Quadrangle, 1:2,400, based on 1923 Topographic Survey.

36. United States Geological Survey, 1942, Point Loma, California 7½-Minute Quadrangle, 1:2,400.

37. URS, 2004, *San Diego County Multi-Jurisdictional Hazard Mitigation Plan, San Diego County, California*, dated March 15, (URS Project No. 27653042.00500).

Priority Development Project (PDP)
Storm Water Quality Management Plan (SWQMP)

National Ave Self-Storage

Permit Application Number:

Drawings Number:

☐ **Check if electing for offsite alternative compliance**

Engineer of Work:

REGISTERED PROFESSIONAL ENGINEER
PATRIC T. DE BOER
No. C 83583
EXP. 3/31/19
CIVIL
STATE OF CALIFORNIA

Patric De Boer RCE 83583
Provide Wet Signature and Stamp Above Line

Prepared For:

U-STOR-IT (Barrio Logan) LLC
2209 National Avenue
San Diego, CA 92113
(847) 346-8776

Prepared By:

OMEGA
ENGINEERING CONSULTANTS

Omega Engineering Consultants
4343 Viewridge Ave. Suite B
San Diego, CA 92123
(858)-634-8620

Date:
July 23, 2018

Approved by: City of San Diego Date

The City of SAN DIEGO

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Project Name: U-STOR-IT (Barrio Logan) LLC

Table of Contents

- Attachment 3: Structural BMP Maintenance Plan
    - Maintenance Agreement (Form DS-3247) (when applicable)
- Attachment 4: Copy of Plan Sheets Showing Permanent Storm Water BMPs
- Attachment 5: Project's Drainage Report
- Attachment 6: Project's Geotechnical and Groundwater Investigation Report

Acronyms

APN	Assessor's Parcel Number
ASBS	Area of Special Biological Significance
BMP	Best Management Practice
CEQA	California Environmental Quality Act
CGP	Construction General Permit
DCV	Design Capture Volume
DMA	Drainage Management Areas
ESA	Environmentally Sensitive Area
GLU	Geomorphic Landscape Unit
GW	Ground Water
HMP	Hydromodification Management Plan
HSG	Hydrologic Soil Group
HU	Harvest and Use
INF	Infiltration
LID	Low Impact Development
LUP	Linear Underground/Overhead Projects
MS4	Municipal Separate Storm Sewer System
N/A	Not Applicable
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
PDP	Priority Development Project
PE	Professional Engineer
POC	Pollutant of Concern
SC	Source Control
SD	Site Design
SDRWQCB	San Diego Regional Water Quality Control Board
SIC	Standard Industrial Classification
SWPPP	Stormwater Pollutant Protection Plan
SWQMP	Storm Water Quality Management Plan
TMDL	Total Maximum Daily Load
WMAA	Watershed Management Area Analysis
WPCP	Water Pollution Control Program
WQIP	Water Quality Improvement Plan

Certification Page

Project Name:
Permit Application

I hereby declare that I am the Engineer in Responsible Charge of design of storm water BMPs for this project, and that I have exercised responsible charge over the design of the project as defined in Section 6703 of the Business and Professions Code, and that the design is consistent with the requirements of the Storm Water Standards, which is based on the requirements of SDRWQCB Order No. R9-2013-0001 as amended by R9-2015-0001 and R9-2015-0100 (MS4 Permit).

I have read and understand that the City Engineer has adopted minimum requirements for managing urban runoff, including storm water, from land development activities, as described in the Storm Water Standards. I certify that this PDP SWQMP has been completed to the best of my ability and accurately reflects the project being proposed and the applicable source control and site design BMPs proposed to minimize the potentially negative impacts of this project's land development activities on water quality. I understand and acknowledge that the plan check review of this PDP SWQMP by the City Engineer is confined to a review and does not relieve me, as the Engineer in Responsible Charge of design of storm water BMPs for this project, of my responsibilities for project design.

Patric de Boer
____
Engineer of Work's Signature

83583 ______________ 3/31/19 ______________

PE# Expiration Date

Patric De Boer
____
Print Name

Omega Engineering Consultants
____
Company

July 23, 2018
____
Date

REGISTERED PROFESSIONAL ENGINEER
PATRIC T. DE BOER
No. C 83583
CIVIL
STATE OF CALIFORNIA

Engineer's Stamp

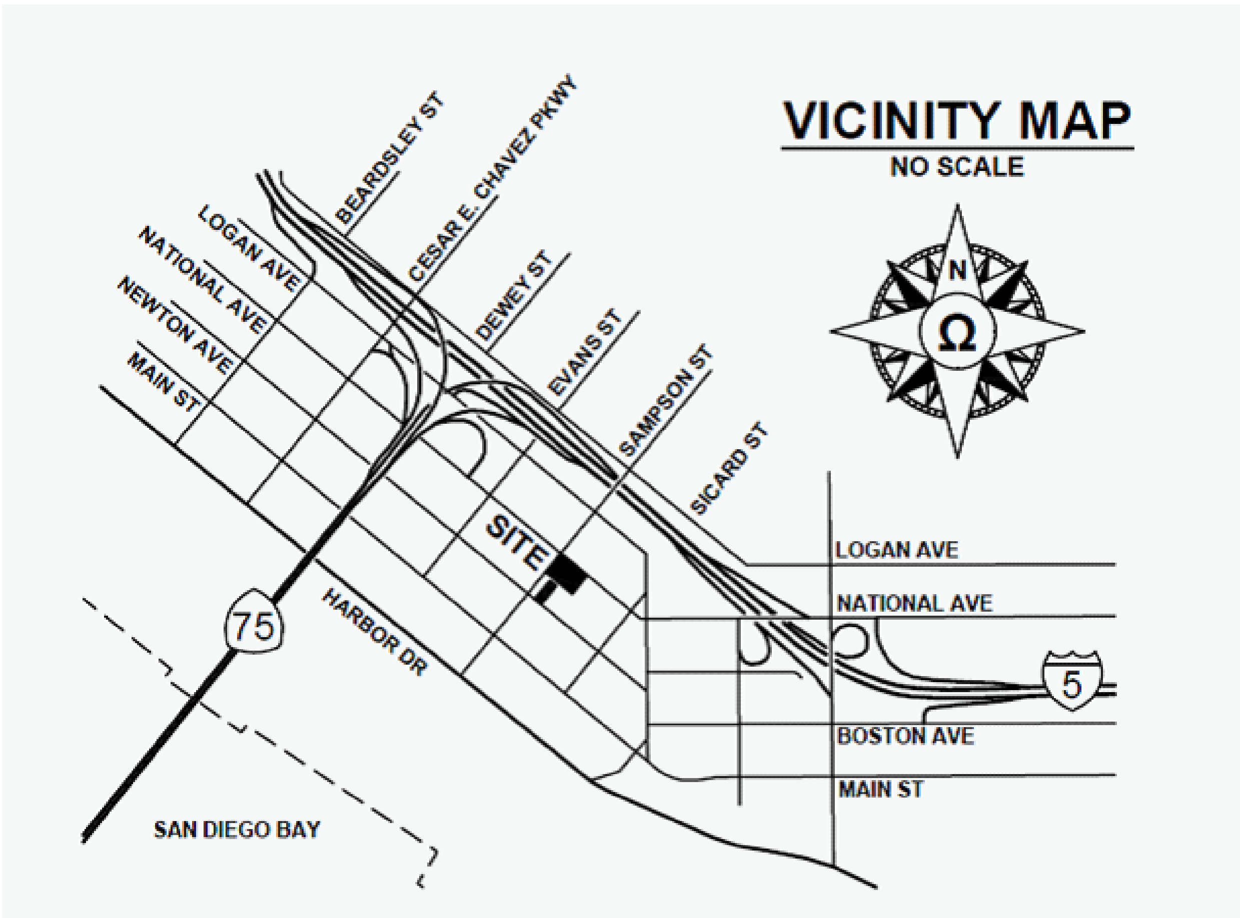
Submittal Record

Use this Table to keep a record of submittals of this PDP SWQMP. Each time the PDP SWQMP is re-submitted, provide the date and status of the project. In last column indicate changes that have been made or indicate if response to plancheck comments is included. When applicable, insert response to plancheck comments.

Submittal Number	Date	Project Status	Changes
1	10/27/17	☑ Preliminary Design/Planning/CEQA ☐ Final Design	**Initial Submittal**
2	07/23/18	☐ Preliminary Design/Planning/CEQA ☑ Final Design	First Ministerial Submittal
3		☐ Preliminary Design/Planning/CEQA ☐ Final Design	
4		☐ Preliminary Design/Planning/CEQA ☐ Final Design	

Project Vicinity Map

Project Name: U-STOR-IT (Barrio Logan) LLC
Permit Application



City of San Diego Form DS-560 Storm Water Requirements Applicability Checklist

Attach DS-560 form.

City of San Diego
Development Services
1222 First Ave., MS-302
San Diego, CA 92101
(619) 446-5000

Storm Water Requirements Applicability Checklist

FORM
DS-560
October 2016

Project Address: 2209 National Avenue, San Diego CA, 92113	Project Number *(for City Use Only)*:

SECTION 1. Construction Storm Water BMP Requirements:

All construction sites are required to implement construction BMPs in accordance with the performance standards in the Storm Water Standards Manual. Some sites are additionally required to obtain coverage under the State Construction General Permit (CGP)[1], which is administered by the State Water Resources Control Board.

For all projects complete PART A: If project is required to submit a SWPPP or WPCP, continue to PART B.

PART A: Determine Construction Phase Storm Water Requirements.

1. Is the project subject to California's statewide General NPDES permit for Storm Water Discharges Associated with Construction Activities, also known as the State Construction General Permit (CGP)? (Typically projects with land disturbance greater than or equal to 1 acre.)

☐ Yes; SWPPP required, skip questions 2-4 ☒ No; next question

2. Does the project propose construction or demolition activity, including but not limited to, clearing, grading, grubbing, excavation, or any other activity resulting in ground disturbance and contact with storm water runoff?

☒ Yes; WPCP required, skip 3-4 ☐ No; next question

3. Does the project propose routine maintenance to maintain original line and grade, hydraulic capacity, or original purpose of the facility? (Projects such as pipeline/utility replacement)

☐ Yes; WPCP required, skip 4 ☒ No; next question

4. Does the project only include the following Permit types listed below?

- Electrical Permit, Fire Alarm Permit, Fire Sprinkler Permit, Plumbing Permit, Sign Permit, Mechanical Permit, Spa Permit.
- Individual Right of Way Permits that exclusively include only ONE of the following activities: water service, sewer lateral, or utility service.
- Right of Way Permits with a project footprint less than 150 linear feet that exclusively include only ONE of the following activities: curb ramp, sidewalk and driveway apron replacement, pot holing, curb and gutter replacement, and retaining wall encroachments.

☐ Yes; no document required

Check one of the boxes below, and continue to PART B:

☐ If you checked "Yes" for question 1,
a SWPPP is REQUIRED. Continue to PART B

☒ If you checked "No" for question 1, and checked "Yes" for question 2 or 3,
a WPCP is REQUIRED. If the project proposes less than 5,000 square feet of ground disturbance AND has less than a 5-foot elevation change over the entire project area, a Minor WPCP may be required instead. **Continue to PART B.**

☐ If you checked "No" for all questions 1-3, and checked "Yes" for question 4
PART B **does not apply and no document is required. Continue to Section 2.**

1. More information on the City's construction BMP requirements as well as CGP requirements can be found at: www.sandiego.gov/stormwater/regulations/index.shtml

PART B: Determine Construction Site Priority

This prioritization must be completed within this form, noted on the plans, and included in the SWPPP or WPCP. The city reserves the right to adjust the priority of projects both before and after construction. Construction projects are assigned an inspection frequency based on if the project has a "high threat to water quality." The City has aligned the local definition of "high threat to water quality" to the risk determination approach of the State Construction General Permit (CGP). The CGP determines risk level based on project specific sediment risk and receiving water risk. Additional inspection is required for projects within the Areas of Special Biological Significance (ASBS) watershed. **NOTE:** The construction priority does **NOT** change construction BMP requirements that apply to projects; rather, it determines the frequency of inspections that will be conducted by city staff.

Complete PART B and continued to Section 2

1. ☐ **ASBS**
   a. Projects located in the ASBS watershed.

2. ☐ **High Priority**
   a. Projects 1 acre or more determined to be Risk Level 2 or Risk Level 3 per the Construction General Permit and not located in the ASBS watershed.
   b. Projects 1 acre or more determined to be LUP Type 2 or LUP Type 3 per the Construction General Permit and not located in the ASBS watershed.

3. ☐ **Medium Priority**
   a. Projects 1 acre or more but not subject to an ASBS or high priority designation.
   b. Projects determined to be Risk Level 1 or LUP Type 1 per the Construction General Permit and not located in the ASBS watershed.

4. ☒ **Low Priority**
   a. Projects requiring a Water Pollution Control Plan but not subject to ASBS, high, or medium priority designation.

SECTION 2. Permanent Storm Water BMP Requirements.

Additional information for determining the requirements is found in the Storm Water Standards Manual.

PART C: Determine if Not Subject to Permanent Storm Water Requirements.

Projects that are considered maintenance, or otherwise not categorized as "new development projects" or "redevelopment projects" according to the Storm Water Standards Manual are not subject to Permanent Storm Water BMPs.

If "yes" is checked for any number in Part C, proceed to Part F and check "Not Subject to Permanent Storm Water BMP Requirements".

If "no" is checked for all of the numbers in Part C continue to Part D.

1.	Does the project only include interior remodels and/or is the project entirely within an existing enclosed structure and does not have the potential to contact storm water?	☐ Yes ☒ No
2.	Does the project only include the construction of overhead or underground utilities without creating new impervious surfaces?	☐ Yes ☒ No
3.	Does the project fall under routine maintenance? Examples include, but are not limited to: roof or exterior structure surface replacement, resurfacing or reconfiguring surface parking lots or existing roadways without expanding the impervious footprint, and routine replacement of damaged pavement (grinding, overlay, and pothole repair).	☐ Yes ☒ No

PART D: PDP Exempt Requirements.

PDP Exempt projects are required to implement site design and source control BMPs.

If "yes" was checked for any questions in Part D, continue to Part F and check the box labeled "PDP Exempt."

If "no" was checked for all questions in Part D, continue to Part E.

1. **Does the project ONLY include new or retrofit sidewalks, bicycle lanes, or trails that:**
   - **Are designed and constructed to direct storm water runoff to adjacent vegetated areas, or other non-erodible permeable areas? Or;**
   - **Are designed and constructed to be hydraulically disconnected from paved streets and roads? Or;**
   - **Are designed and constructed with permeable pavements or surfaces in accordance with the Green Streets guidance in the City's Storm Water Standards manual?**

   ☐ Yes; PDP exempt requirements apply ☒ No; next question

2. Does the project ONLY include retrofitting or redeveloping existing paved alleys, streets or roads designed and constructed in accordance with the Green Streets guidance in the City's Storm Water Standards Manual?

   ☐ Yes; PDP exempt requirements apply ☒ No; project not exempt.

PART E: Determine if Project is a Priority Development Project (PDP).

Projects that match one of the definitions below are subject to additional requirements including preparation of a Storm Water Quality Management Plan (SWQMP).

If "yes" is checked for any number in PART E, continue to PART F and check the box labeled "Priority Development Project".

If "no" is checked for every number in PART E, continue to PART F and check the box labeled "Standard Development Project".

1. **New Development that creates 10,000 square feet or more of impervious surfaces collectively over the project site.** This includes commercial, industrial, residential, mixed-use, and public development projects on public or private land. ☐ Yes ☒ No
2. **Redevelopment project that creates and/or replaces 5,000 square feet or more of impervious surfaces on an existing site of 10,000 square feet or more of impervious surfaces**. This includes commercial, industrial, residential, mixed-use, and public development projects on public or private land. ☒ Yes ☐ No
3. **New development or redevelopment of a restaurant.** Facilities that sell prepared foods and drinks for consumption, including stationary lunch counters and refreshment stands selling prepared foods and drinks for immediate consumption (SIC 5812), and where the land development creates and/or replace 5,000 square feet or more of impervious surface. ☐ Yes ☒ No
4. **New development or redevelopment on a hillside.** The project creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site) and where the development will grade on any natural slope that is twenty-five percent or greater. ☐ Yes ☒ No
5. **New development or redevelopment of a parking lot that creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site).** ☒ Yes ☐ No
6. **New development or redevelopment of streets, roads, highways, freeways, and driveways.** The project creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site). ☐ Yes ☒ No

7. **New development or redevelopment discharging directly to an Environmentally Sensitive Area.** The project creates and/or replaces 2,500 square feet of impervious surface (collectively over project site), and discharges directly to an Environmentally Sensitive Area (ESA). "Discharging directly to" includes flow that is conveyed overland a distance of 200 feet or less from the project to the ESA, or conveyed in a pipe or open channel any distance as an isolated flow from the project to the ESA (i.e. not commingled with flows from adjacent lands). ☐ Yes ☒ No

8. **New development or redevelopment projects of a retail gasoline outlet (RGO) that create and/or replaces 5,000 square feet of impervious surface.** The development project meets the following criteria: (a) 5,000 square feet or more or (b) has a projected Average Daily Traffic (ADT) of 100 or more vehicles per day. ☐ Yes ☒ No

9. **New development or redevelopment projects of an automotive repair shops that creates and/or replaces 5,000 square feet or more of impervious surfaces.** Development projects categorized in any one of Standard Industrial Classification (SIC) codes 5013, 5014, 5541, 7532-7534, or 7536-7539. ☐ Yes ☒ No

10. **Other Pollutant Generating Project.** The project is not covered in the categories above, results in the disturbance of one or more acres of land and is expected to generate pollutants post construction, such as fertilizers and pesticides. This does not include projects creating less than 5,000 sf of impervious surface and where added landscaping does not require regular use of pesticides and fertilizers, such as slope stabilization using native plants. Calculation of the square footage of impervious surface need not include linear pathways that are for infrequent vehicle use, such as emergency maintenance access or bicycle pedestrian use, if they are built with pervious surfaces of if they sheet flow to surrounding pervious surfaces. ☐ Yes ☒ No

PART F: Select the appropriate category based on the outcomes of PART C through PART E.

1.	The project is **NOT SUBJECT TO PERMANENT STORM WATER REQUIREMENTS**.	☐
2.	The project is a **STANDARD DEVELOPMENT PROJECT**. Site design and source control BMP requirements apply. See the Storm Water Standards Manual for guidance.	☐
3.	The project is **PDP EXEMPT**. Site design and source control BMP requirements apply. See the Storm Water Standards Manual for guidance.	☐
4.	The project is a **PRIORITY DEVELOPMENT PROJECT**. Site design, source control, and structural pollutant control BMP requirements apply. See the Storm Water Standards Manual for guidance on determining if project requires a hydromodification plan management	☒

Jonathan Teas — Staff Engineer

Name of Owner or Agent *(Please Print)* — Title

Jonathan teas — 06/27/2018

Signature — Date

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Applicability of Permanent, Post-Construction Storm Water BMP Requirements			Form I-1
Project Identification			
Project Name: U-STOR-IT (Barrio Logan) LLC			
Permit Application Number:			Date: 07/23/18
Determination of Requirements			
The purpose of this form is to identify permanent, post-construction requirements that apply to the project. This form serves as a short summary of applicable requirements, in some cases referencing separate forms that will serve as the backup for the determination of requirements. Answer each step below, starting with **Step 1** and progressing through each step until reaching "Stop". Refer to the manual sections and/or separate forms referenced in each step below.			
Step	**Answer**	**Progression**	
Step 1: Is the project a "development project"? See Section 1.3 of the manual (Part 1 of Storm Water Standards) for guidance.	☑ Yes	Go to **Step 2**.	
	☐ No	**Stop**. Permanent BMP requirements do not apply. No SWQMP will be required. Provide discussion below.	
Discussion / justification if the project is not a "development project" (e.g., the project includes *only* interior remodels within an existing building): N/A			
Step 2: Is the project a Standard Project, PDP, or PDP Exempt? To answer this item, see Section 1.4 of the manual in its entirety for guidance AND complete Form DS-560, Storm Water Requirements Applicability Checklist.	☐ Standard Project	**Stop.** Standard Project requirements apply	
	☑ PDP	PDP requirements apply, including PDP SWQMP. Go to **Step 3**.	
	☐ PDP Exempt	**Stop.** Standard Project requirements apply. Provide discussion and list any additional requirements below.	
Discussion / justification, and additional requirements for exceptions to PDP definitions, if applicable: N/A			

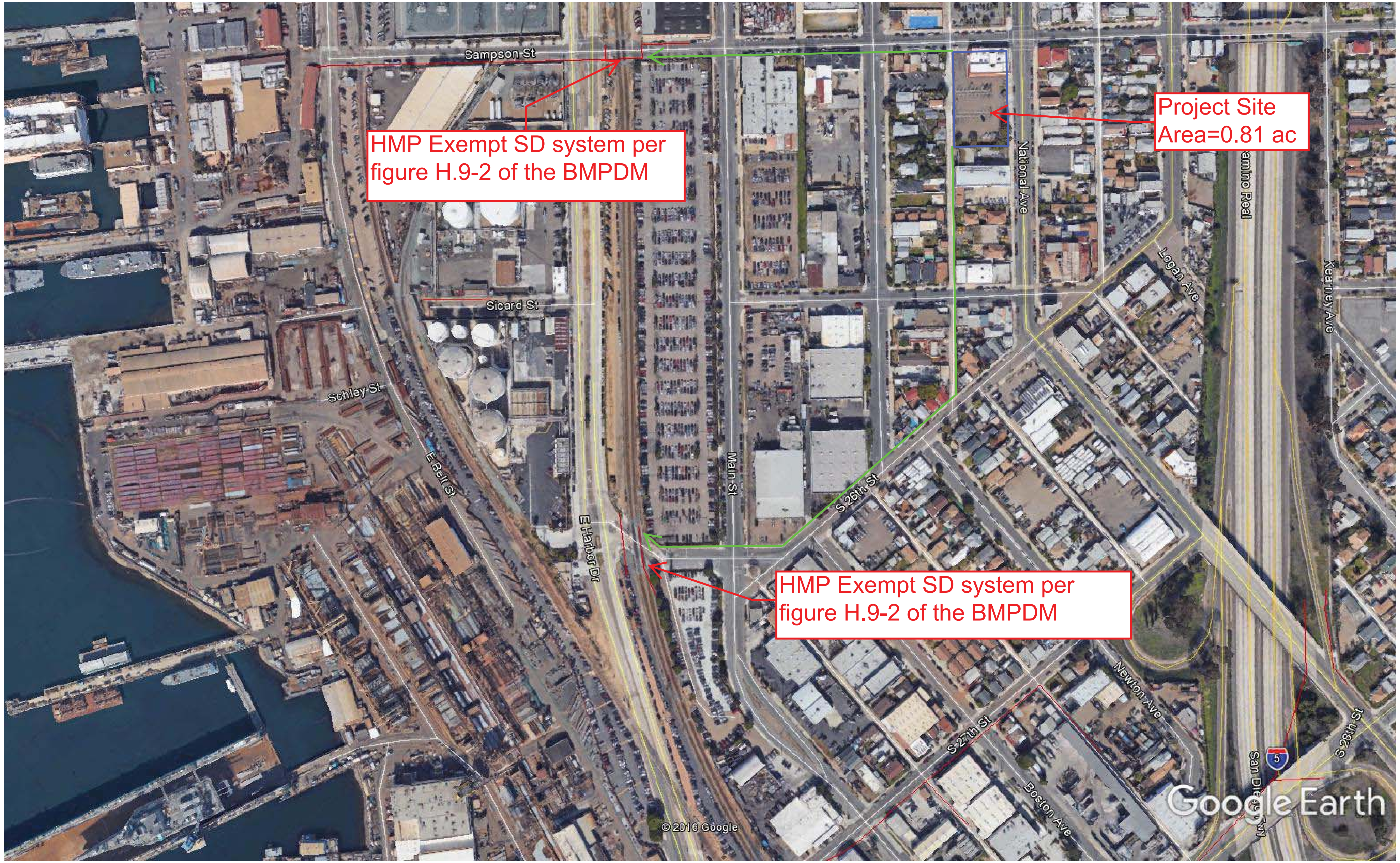
Project Name: U-STOR-IT (Barrio Logan) LLC

Form I-1 Page 2 of 2		
Step	**Answer**	**Progression**
Step 3. Is the project subject to earlier PDP requirements due to a prior lawful approval? See Section 1.10 of the manual (Part 1 of Storm Water Standards) for guidance.	☐ Yes	Consult the City Engineer to determine requirements. Provide discussion and identify requirements below. Go to **Step 4**.
	☑ No	BMP Design Manual PDP requirements apply. Go to **Step 4**.
Discussion / justification of prior lawful approval, and identify requirements (<u>not required if prior lawful approval does not apply</u>):		
Step 4. Do hydromodification control requirements apply? See Section 1.6 of the manual (Part 1 of Storm Water Standards) for guidance.	☐ Yes	PDP structural BMPs required for pollutant control (Chapter 5) and hydromodification control (Chapter 6). Go to **Step 5**.
	☑ No	**Stop**. PDP structural BMPs required for pollutant control (Chapter 5) only. Provide brief discussion of exemption to hydromodification control below.
Discussion / justification if hydromodification control requirements do <u>not</u> apply: The proposed project discharges from the site to an alley, and then storm water is carried via curb and gutter to the public storm drain system which carries the storm water directly to the San Diego Bay, a hydromodification exempt water body. See HMP Exemption Exhibit.		
Step 5. Does protection of critical coarse sediment yield areas apply? See Section 6.2 of the manual (Part 1 of Storm Water Standards) for guidance.	☐ Yes	Management measures required for protection of critical coarse sediment yield areas (Chapter 6.2). **Stop**.
	☑ No	Management measures not required for protection of critical coarse sediment yield areas. Provide brief discussion below. **Stop**.
Discussion / justification if protection of critical coarse sediment yield areas does <u>not</u> apply: This Section does not apply because hydromodification management requirements do not apply. However, the site is not located downstream of a Critical Coarse Sediment Yield Area.		

HMP Exemption Exhibit

Attach a HMP Exemption Exhibit that shows direct storm water runoff discharge from the project site to HMP exempt area. Include project area, applicable underground storm drain line and/or concrete lined channels, outfall information and exempt waterbody.
Reference applicable drawing number(s).

Exhibit must be provided on 11"x17" or larger paper.



Approximate Site Boundary

Anticipated Site Drainage Routes

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Project Name: U-STOR-IT (Barrio Logan) LLC

Site Information Checklist For PDPs	Form I-3B
Project Summary Information	
Project Name	U-STOR-IT (Barrio Logan) LLC
Project Address	2209 National Avenue San Diego, CA 92113
Assessor's Parcel Number(s) (APN(s))	538-690-29-00, 538-690-34-00, and 538-690-37-00
Permit Application Number	
Project Watershed	Select One: ☐ San Dieguito River ☐ Penasquitos ☐ Mission Bay ☐ San Diego River ☑ San Diego Bay ☐ Tijuana River
Hydrologic subarea name with Numeric Identifier up to two decimal places (9XX.XX)	908.22 Chollas Hydrologic Sub-Area
Project Area (total area of Assessor's Parcel(s) associated with the project or total area of the right-of-way)	35,130 s.f. (0.806 a Acres (35,131 Square Feet)
Area to be disturbed by the project (Project Footprint)	0.81 Acres (35,131 Square Feet)
Project Proposed Impervious Area (subset of Project Footprint)	0.78 Acres (34,078 Square Feet)
Project Proposed Pervious Area (subset of Project Footprint)	0.03 Acres (1,053 Square Feet)
Note: Proposed Impervious Area + Proposed Pervious Area = Area to be Disturbed by the Project. This may be less than the Project Area.	
The proposed increase or decrease in impervious area in the proposed condition as compared to the pre-project condition	-1 %

Form I-3B Page 2 of 11
Description of Existing Site Condition and Drainage Patterns
Current Status of the Site (select all that apply): ☑Existing development ☐Previously graded but not built out ☐Agricultural or other non-impervious use ☐Vacant, undeveloped/natural Description / Additional Information: The existing site is 86% impervious, and consists of the following. An existing bank building located at the northerly corner of the site, the banks parking lot which contains some landscaping, and driveways. Water is conveyed at 4%-5% slopes via surface run-off to the public alley along the south side of the site. Then storm water is conveyed via curb and gutter to the public storm drain system, up to the discharge point being the San Diego Bay.
Existing Land Cover Includes (select all that apply): ☑Vegetative Cover ☐Non-Vegetated Pervious Areas ☑Impervious Areas Description / Additional Information: The existing development land cover consists of: Adjacent sidewalk along the frontage of the site at Sampson Street, National Avenue, a southerly alley, AC paving, PCC paving, some landscaping, and the building rooftop.
Underlying Soil belongs to Hydrologic Soil Group (select all that apply): ☐NRCS Type A ☐NRCS Type B ☐NRCS Type C ☑NRCS Type D
Approximate Depth to Groundwater: ☐Groundwater Depth < 5 feet ☐5 feet < Groundwater Depth < 10 feet ☐10 feet < Groundwater Depth < 20 feet ☑Groundwater Depth > 20 feet
Existing Natural Hydrologic Features (select all that apply): ☐Watercourses ☐Seeps ☐Springs ☐Wetlands ☑None Description / Additional Information: N/A

Form I-3B Page 3 of 11
Description of Existing Site Topography and Drainage
How is storm water runoff conveyed from the site? At a minimum, this description should answer: 1. Whether existing drainage conveyance is natural or urban; 2. If runoff from offsite is conveyed through the site? If yes, quantification of all offsite drainage areas, design flows, and locations where offsite flows enter the project site and summarize how such flows are conveyed through the site; 3. Provide details regarding existing project site drainage conveyance network, including storm drains, concrete channels, swales, detention facilities, storm water treatment facilities, and natural and constructed channels; 4. Identify all discharge locations from the existing project along with a summary of the conveyance system size and capacity for each of the discharge locations. Provide summary of the pre-project drainage areas and design flows to each of the existing runoff discharge locations.
Descriptions/Additional Information
1. The existing drainage conveyance is urban and consists of surface flow along the existing parking lot and rooftop. 2. No offsite run-on is expected to enter the site. 3. The existing site drains to the southwest via surface run-off to the adjacent alley where surface flows to the discharge points and then either Sampson street or 26th 4. The storm water discharges from the site to the southerly corner of the site (discharge point 1) and the westerly corner of the site (discharge point 2). After leaving the site, storm water is conveyed via curb and gutter to the public storm drain system at curb inlets along 26th Street and Sampson Street respectively. Discharge point 1 receives 2.82 CFS, and discharge point 2 receives 1.38 CFS from the site for the 100 year, 6hr storm. See drainage report within appendix 5 for more information.

Form I-3B Page 4 of 11
Description of Proposed Site Development and Drainage Patterns
Project Description / Proposed Land Use and/or Activities: The project proposes to demolish and remove the existing structures and hardscape and construct a 5-story self-storage facility. The proposed improvements include the self-storage building, a drive way, and the addition of two biofiltration area basins located along the south westerly boundary of the site. Off-site street improvements are also proposed, including: A driveway, and replacement of adjacent sidewalk at the frontages of the site along Sampson Street, National Avenue, and the adjacent alley.
List/describe proposed impervious features of the project (e.g., buildings, roadways, parking lots, courtyards, athletic courts, other impervious features): The impervious features of the site will include the building roof which will take up the majority of the site.
List/describe proposed pervious features of the project (e.g., landscape areas): The pervious features of the site will be limited to the proposed biofiltration basins.
Does the project include grading and changes to site topography? ☑ Yes ☐ No Description / Additional Information: Project will include grading to meet the proposed design intent, but the grading will be done in a manner to minimize earthwork by following the general grade of the existing conditions where possible to do so.

Form I-3B Page 5 of 11
Does the project include changes to site drainage (e.g., installation of new storm water conveyance systems)? ☑ Yes ☐ No If yes, provide details regarding the proposed project site drainage conveyance network, including storm drains, concrete channels, swales, detention facilities, storm water treatment facilities, natural and constructed channels, and the method for conveying offsite flows through or around the proposed project site. Identify all discharge locations from the proposed project site along with a summary of the conveyance system size and capacity for each of the discharge locations. Provide a summary of pre and post-project drainage areas and design flows to each of the runoff discharge locations. Reference the drainage study for detailed calculations. Description / Additional Information: The project will be constructed with a storm drain system that will route all runoff from the building roof to the proposed biofiltration basins. The biofiltration basins will drain directly to two points of discharge, which are located along the adjacent alley, and then the public storm drain system conveys flow via curb and gutter down the block. Biofiltration areas will be used to treat run-off for pollution control requirements. See the drainage report within attachment 5, for 100 year storm peak discharge analysis at both discharge points.

Project Name: U-STOR-IT (Barrio Logan) LLC

Form I-3B Page 6 of 11
Identify whether any of the following features, activities, and/or pollutant source areas will be present (select all that apply): ☑Onsite storm drain inlets ☑Interior floor drains and elevator shaft sump pumps ☐Interior parking garages ☐Need for future indoor & structural pest control ☑Landscape/outdoor pesticide use ☐Pools, spas, ponds, decorative fountains, and other water features ☐Food service ☑Refuse areas ☐Industrial processes ☐Outdoor storage of equipment or materials ☐Vehicle and equipment cleaning ☐Vehicle/equipment repair and maintenance ☐Fuel dispensing areas ☐Loading docks ☑Fire sprinkler test water ☐Miscellaneous drain or wash water ☐Plazas, sidewalks, and parking lots Description/Additional Information: N/A

Project Name: U-STOR-IT (Barrio Logan) LLC

Form I-3B Page 7 of 11
Identification and Narrative of Receiving Water
Narrative describing flow path from discharge location(s), through urban storm conveyance system, to receiving creeks, rivers, and lagoons and ultimate discharge location to Pacific Ocean (or bay, lagoon, lake or reservoir, as applicable)
The storm water discharges from the site to the southerly corner of the site (discharge point 1) and the westerly corner of the site (discharge point 2) discharging to the surface of an adjacent alley along the south westerly boundary. Runoff discharging from discharge point 1, then drains south easterly along the adjacent gutter in an alley, and then is carried southerly along South 26th street, where it enters the public storm drain system just before E. Harbor Drive. Runoff discharging from discharge point 2 then drains westerly along the adjacent gutter in an alley, and then is carried south westerly along Street Sampson Street entering the public storm drain system near the main street cross-street. See site DMA map for further details.
Provide a summary of all beneficial uses of receiving waters downstream of the project discharge locations
San Diego Bay: BIOL, COMM, EST, IND, MAR, MIGR, NAV, RARE, REC1, RECE2, SHELL, AND WILD
Identify all ASBS (areas of special biological significance) receiving waters downstream of the project discharge locations
Approximately 0.4 miles from discharge to public storm drain system, to the San Diego Bay receiving water.
Provide distance from project outfall location to impaired or sensitive receiving waters
Approximately 0.4 miles from discharge to public storm drain system, to the San Diego Bay receiving water.
Summarize information regarding the proximity of the permanent, post-construction storm water BMPs to the City's Multi-Habitat Planning Area and environmentally sensitive lands
Permanent biofiltration BMP's are proposed for the site. The site discharge point does not lie upstream of any MHPA areas identified by the City of San Diego General plan conservation element.

Form I-3B Page 8 of 11		
Identification of Receiving Water Pollutants of Concern		
List any 303(d) impaired water bodies within the path of storm water from the project site to the Pacific Ocean (or bay, lagoon, lake or reservoir, as applicable), identify the pollutant(s)/stressor(s) causing impairment, and identify any TMDLs and/or Highest Priority Pollutants from the WQIP for the impaired water bodies:		
303(d) Impaired Water Body (Refer to Appendix K)	Pollutant(s)/Stressor(s) (Refer to Appendix K)	TMDLs/WQIP Highest Priority Pollutant (Refer to Table 1-4 in Chapter 1)
San Diego Bay	Sediment Toxicity	TMDL Estimated Completion 2008
San Diego Bay	Benthic Community Effects	TMDL Estimated Completion 2019
San Diego Bay	Polychlorinated Biphenyls	TMDL Estimated Completion 2013
San Diego Bay	Indicator bacteria	WQIP Highest Priority Pollutant
San Diego Bay	Dissolved Copper	WQIP Highest Priority Pollutant
San Diego Bay	Lead	WQIP Highest Priority Pollutant
San Diego Bay	Zinc	WQIP Highest Priority Pollutant
San Diego Bay	Copper	TMDL Estimated Completion 2019

Identification of Project Site Pollutants*			
*Identification of project site pollutants is only required if flow-thru treatment BMPs are implemented onsite in lieu of retention or biofiltration BMPs (note the project must also participate in an alternative compliance program unless prior lawful approval to meet earlier PDP requirements is demonstrated) Identify pollutants anticipated from the project site based on all proposed use(s) of the site (see Appendix B.6):			
Pollutant	Not Applicable to the Project Site	Anticipated from the Project Site	Also a Receiving Water Pollutant of Concern
Sediment	☐	☐	☐
Nutrients	☐	☐	☐
Heavy Metals	☐	☐	☐
Organic Compounds	☐	☐	☐
Trash & Debris	☐	☐	☐
Oxygen Demanding Substances	☐	☐	☐
Oil & Grease	☐	☐	☐
Bacteria & Viruses	☐	☐	☐
Pesticides	☐	☐	☐

Form I-3B Page 9 of 11

Hydromodification Management Requirements

Do hydromodification management requirements apply (see Section 1.6)?

☐ Yes, hydromodification management flow control structural BMPs required.

☐ No, the project will discharge runoff directly to existing underground storm drains discharging directly to water storage reservoirs, lakes, enclosed embayments, or the Pacific Ocean.

☐ No, the project will discharge runoff directly to conveyance channels whose bed and bank are concrete-lined all the way from the point of discharge to water storage reservoirs, lakes, enclosed embayments, or the Pacific Ocean.

☑ No, the project will discharge runoff directly to an area identified as appropriate for an exemption by the WMAA for the watershed in which the project resides.

Description / Additional Information (to be provided if a 'No' answer has been selected above):

The proposed project discharges via hardened conveyance to the underground storm drain systems that carry the storm water to the San Diego Bay, a hydromodification exempt water body. See hydrmodofication exemption exhibit.

Note: If "No" answer has been selected the SWQMP must include an exhibit that shows the storm water conveyance system from the project site to an exempt water body. The exhibit should include details about the conveyance system and the outfall to the exempt water body.

Critical Coarse Sediment Yield Areas*
*This Section only required if hydromodification management requirements apply

Based on Section 6.2 and Appendix H does CCSYA exist on the project footprint or in the upstream area draining through the project footprint?

☐ Yes

☐ No

Discussion / Additional Information:

N/A

Form I-3B Page 10 of 11
Flow Control for Post-Project Runoff* ***This Section only required if hydromodification management requirements apply**
List and describe point(s) of compliance (POCs) for flow control for hydromodification management (see Section 6.3.1). For each POC, provide a POC identification name or number correlating to the project's HMP Exhibit and a receiving channel identification name or number correlating to the project's HMP Exhibit. N/A
Has a geomorphic assessment been performed for the receiving channel(s)? ☑No, the low flow threshold is $0.1Q_2$ (default low flow threshold) ☐Yes, the result is the low flow threshold is $0.1Q_2$ ☐Yes, the result is the low flow threshold is $0.3Q_2$ ☐Yes, the result is the low flow threshold is $0.5Q_2$ If a geomorphic assessment has been performed, provide title, date, and preparer: N/A
Discussion / Additional Information: (optional) N/A

Form I-3B Page 11 of 11
Other Site Requirements and Constraints
When applicable, list other site requirements or constraints that will influence storm water management design, such as zoning requirements including setbacks and open space, or local codes governing minimum street width, sidewalk construction, allowable pavement types, and drainage requirements. The building takes up the majority of the site, thus we have designed the storm water conveyance system to capture and treat storm water from the roof. The storm water discharge has been designed to be similiar to the previous/existing condition. This was required in order to match the flow discharge rates and locations of the existing site.
Optional Additional Information or Continuation of Previous Sections As Needed
This space provided for additional information or continuation of information from previous sections as needed. N/A

Project Name: U-STOR-IT (Barrio Logan) LLC

Source Control BMP Checklist for PDPs			Form I-4B
Source Control BMPs			
All development projects must implement source control BMPs where applicable and feasible. See Chapter 4 and Appendix E of the BMP Design Manual (Part 1 of the Storm Water Standards) for information to implement source control BMPs shown in this checklist. Answer each category below pursuant to the following. • "Yes" means the project will implement the source control BMP as described in Chapter 4 and/or Appendix E of the BMP Design Manual. Discussion / justification is not required. • "No" means the BMP is applicable to the project but it is not feasible to implement. Discussion / justification must be provided. • "N/A" means the BMP is not applicable at the project site because the project does not include the feature that is addressed by the BMP (e.g., the project has no outdoor materials storage areas). Discussion / justification may be provided.			
Source Control Requirement	**Applied?**		
4.2.1 Prevention of Illicit Discharges into the MS4	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.2.1 not implemented:			
4.2.2 Storm Drain Stenciling or Signage	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.2.2 not implemented:			
4.2.3 Protect Outdoor Materials Storage Areas from Rainfall, Run-On, Runoff, and Wind Dispersal	☐ Yes	☑ No	☐ N/A
Discussion / justification if 4.2.3 not implemented: No outdoor material storage areas on the sites.			
4.2.4 Protect Materials Stored in Outdoor Work Areas from Rainfall, Run-On, Runoff, and Wind Dispersal	☐ Yes	☑ No	☐ N/A
Discussion / justification if 4.2.4 not implemented: No outdoor material storage areas on the sites			
4.2.5 Protect Trash Storage Areas from Rainfall, Run-On, Runoff, and Wind Dispersal	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.2.5 not implemented:			

Form I-4B Page 2 of 2			
Source Control Requirement	**Applied?**		
4.2.6 Additional BMPs Based on Potential Sources of Runoff Pollutants (must answer for each source listed below)			
On-site storm drain inlets	☑ Yes	☐ No	☐ N/A
Interior floor drains and elevator shaft sump pumps	☑ Yes	☐ No	☐ N/A
Interior parking garages	☑ Yes	☐ No	☐ N/A
Need for future indoor & structural pest control	☑ Yes	☐ No	☐ N/A
Landscape/Outdoor Pesticide Use	☑ Yes	☐ No	☐ N/A
Pools, spas, ponds, decorative fountains, and other water features	☐ Yes	☐ No	☑ N/A
Food service	☐ Yes	☐ No	☑ N/A
Refuse areas	☑ Yes	☐ No	☐ N/A
Industrial processes	☐ Yes	☐ No	☑ N/A
Outdoor storage of equipment or materials	☐ Yes	☐ No	☑ N/A
Vehicle/Equipment Repair and Maintenance	☐ Yes	☐ No	☑ N/A
Fuel Dispensing Areas	☐ Yes	☐ No	☑ N/A
Loading Docks	☐ Yes	☐ No	☑ N/A
Fire Sprinkler Test Water	☐ Yes	☐ No	☑ N/A
Miscellaneous Drain or Wash Water	☐ Yes	☐ No	☑ N/A
Plazas, sidewalks, and parking lots	☐ Yes	☐ No	☑ N/A
SC-6A: Large Trash Generating Facilities	☐ Yes	☐ No	☑ N/A
SC-6B: Animal Facilities	☐ Yes	☐ No	☑ N/A
SC-6C: Plant Nurseries and Garden Centers	☐ Yes	☐ No	☑ N/A
SC-6D: Automotive Facilities	☐ Yes	☐ No	☑ N/A
Discussion / justification if 4.2.6 not implemented. Clearly identify which sources of runoff pollutants are discussed. Justification must be provided for all "No" answers shown above. N/A			

Project Name: U-STOR-IT (Barrio Logan) LLC

Site Design BMP Checklist for PDPs	Form I-5B		
Site Design BMPs			
All development projects must implement site design BMPs where applicable and feasible. See Chapter 4 and Appendix E of the BMP Design Manual (Part 1 of Storm Water Standards) for information to implement site design BMPs shown in this checklist. Answer each category below pursuant to the following. • "Yes" means the project will implement the site design BMP as described in Chapter 4 and/or Appendix E of the BMP Design Manual. Discussion / justification is not required. • "No" means the BMP is applicable to the project but it is not feasible to implement. Discussion / justification must be provided. • "N/A" means the BMP is not applicable at the project site because the project does not include the feature that is addressed by the BMP (e.g., the project site has no existing natural areas to conserve). Discussion / justification may be provided. A site map with implemented site design BMPs must be included at the end of this checklist.			
Site Design Requirement	**Applied?**		
4.3.1 Maintain Natural Drainage Pathways and Hydrologic Features	☐ Yes	☐ No	☑ N/A
Discussion / justification if 4.3.1 not implemented: No natural drainage pathways remain on this previously developed site			
1-1 Are existing natural drainage pathways and hydrologic features mapped on the site map?	☐ Yes	☐ No	☑ N/A
1-2 Are trees implemented? If yes, are they shown on the site map?	☐ Yes	☐ No	☑ N/A
1-3 Implemented trees meet the design criteria in 4.3.1 Fact Sheet (e.g. soil volume, maximum credit, etc.)?	☐ Yes	☐ No	☑ N/A
1-4 Is tree credit volume calculated using Appendix B.2.2.1 and SD-1 Fact Sheet in Appendix E?	☐ Yes	☐ No	☑ N/A
4.3.2 Have natural areas, soils and vegetation been conserved?	☐ Yes	☐ No	☑ N/A
Discussion / justification if 4.3.2 not implemented:			

Project Name: U-STOR-IT (Barrio Logan) LLC

Form I-5B Page 2 of 4			
Site Design Requirement	**Applied?**		
4.3.3 Minimize Impervious Area	☐ Yes	☑ No	☐ N/A
Discussion / justification if 4.3.3 not implemented: The proposed storage facility will take up the majority of the site.			
4.3.4 Minimize Soil Compaction	☐ Yes	☑ No	☐ N/A
Discussion / justification if 4.3.4 not implemented: Project impervious surface requirements for the building, does not allow for implementation of SD-4			
4.3.5 Impervious Area Dispersion	☐ Yes	☑ No	☐ N/A
Discussion / justification if 4.3.5 not implemented: The proposed storage facility will take up the majority of the site. The biofiltration basin has been sized to handle the DCV projected by the site assuming no dispersion.			
5-1 Is the pervious area receiving runon from impervious area identified on the site map?	☐ Yes	☐ No	☑ N/A
5-2 Does the pervious area satisfy the design criteria in 4.3.5 Fact Sheet in Appendix E (e.g. maximum slope, minimum length, etc.)	☐ Yes	☐ No	☑ N/A
5-3 Is impervious area dispersion credit volume calculated using Appendix B.2.1.1 and 4.3.5 Fact Sheet in Appendix E?	☐ Yes	☐ No	☑ N/A

Form I-5B Page 3 of 4			
Site Design Requirement	**Applied?**		
4.3.6 Runoff Collection	☐ Yes	☑ No	☐ N/A
Discussion / justification if 4.3.6 not implemented: The proposed site, does not collect enough run-off to justify the collection and use of run-off.			
6a-1 Are green roofs implemented in accordance with design criteria in 4.3.6A Fact Sheet? If yes, are they shown on the site map?	☐ Yes	☐ No	☑ N/A
6a-2 Is the green roof credit volume calculated using Appendix B.2.1.2 and 4.3.6A Fact Sheet in Appendix E?	☐ Yes	☐ No	☑ N/A
6b-1 Are permeable pavements implemented in accordance with design criteria in 4.3.6B Fact Sheet? If yes, are they shown on the site map?	☐ Yes	☐ No	☑ N/A
6b-2 Is the permeable pavement credit volume calculated using Appendix B.2.1.3 and 4.3.6B Fact Sheet in Appendix E?	☐ Yes	☐ No	☑ N/A
4.3.7 Landscaping with Native or Drought Tolerant Species	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.3.7 not implemented:			
4.3.8 Harvest and Use Precipitation	☐ Yes	☑ No	☐ N/A
Discussion / justification if 4.3.8 not implemented: The proposed site as a five-story self-storage facility, will present a low demand for harvested rainwater. The low demand does not justify implementing harvesting and use of precipitation, see Attachment 1c.			
8-1 Are rain barrels implemented in accordance with design criteria in 4.3.8 Fact Sheet? If yes, are they shown on the site map?	☐ Yes	☐ No	☑ N/A
8-2 Is the rain barrel credit volume calculated using Appendix B.2.2.2 and 4.3.8 Fact Sheet in Appendix E?	☐ Yes	☐ No	☑ N/A

Project Name: U-STOR-IT (Barrio Logan) LLC

Form I-5B Page 4 of 4
Insert Site Map with all site design BMPs identified:
SEE DMA EXHIBIT ATTACHMENT 1A

Project Name: U-STOR-IT (Barrio Logan) LLC

Summary of PDP Structural BMPs	Form I-6
PDP Structural BMPs	
All PDPs must implement structural BMPs for storm water pollutant control (see Chapter 5 of the BMP Design Manual, Part 1 of Storm Water Standards). Selection of PDP structural BMPs for storm water pollutant control must be based on the selection process described in Chapter 5. PDPs subject to hydromodification management requirements must also implement structural BMPs for flow control for hydromodification management (see Chapter 6 of the BMP Design Manual). Both storm water pollutant control and flow control for hydromodification management can be achieved within the same structural BMP(s). PDP structural BMPs must be verified by the City at the completion of construction. This includes requiring the project owner or project owner's representative to certify construction of the structural BMPs (complete Form DS-563). PDP structural BMPs must be maintained into perpetuity (see Chapter 7 of the BMP Design Manual). Use this form to provide narrative description of the general strategy for structural BMP implementation at the project site in the box below. Then complete the PDP structural BMP summary information sheet (page 3 of this form) for each structural BMP within the project (copy the BMP summary information page as many times as needed to provide summary information for each individual structural BMP).	
Describe the general strategy for structural BMP implementation at the site. This information must describe how the steps for selecting and designing storm water pollutant control BMPs presented in Section 5.1 of the BMP Design Manual were followed, and the results (type of BMPs selected). For projects requiring hydromodification flow control BMPs, indicate whether pollutant control and flow control BMPs are integrated or separate. It was determined by the in-situ geotechnical infiltration tests that the site is underlain by very low infiltration hydrologic soil type D, precluding the use of partial infiltration or full infiltration biofiltration basins. We instead chose to use fully lined biofiltration basin areas to treat site runoff. The biofiltration basins (BMP-1) & (BMP-2), will serve the purpose of pollutant control. The basins will have a minimum 18” treatment soil layer over a 12” gravel layer with a perforated sub-drain. (Continue on page 2 as necessary.)	

Form I-6 Page 2 of
(Continued from page 1)

Project Name: U-STOR-IT (Barrio Logan) LLC

Form I-6 Page of (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP-1	
Construction Plan Sheet No.	
Type of Structural BMP: ☐ Retention by harvest and use (e.g. HU-1, cistern) ☐ Retention by infiltration basin (INF-1) ☐ Retention by bioretention (INF-2) ☐ Retention by permeable pavement (INF-3) ☐ Partial retention by biofiltration with partial retention (PR-1) ☑ Biofiltration (BF-1) ☐ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide BMP type/description in discussion section below) ☐ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below) ☐ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion section below) ☐ Detention pond or vault for hydromodification management ☐ Other (describe in discussion section below)	
Purpose: ☑ Pollutant control only ☐ Hydromodification control only ☐ Combined pollutant control and hydromodification control ☐ Pre-treatment/forebay for another structural BMP ☐ Other (describe in discussion section below)	
Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Andrew J. Kann Omega Engineering 858-634-8620
Who will be the final owner of this BMP?	Lawrence, Nora
Who will maintain this BMP into perpetuity?	Lawrence, Nora
What is the funding mechanism for maintenance?	U-Store It Self Storage

Project Name: U-STOR-IT (Barrio Logan) LLC

Form I-6 Page of (Copy as many as needed)
Structural BMP ID No. BMP-1
Construction Plan Sheet No.
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs):

Project Name: National Ave Self-Storage

Form I-6 Page of (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP-2	
Construction Plan Sheet No.	
Type of Structural BMP: ☐ Retention by harvest and use (e.g. HU-1, cistern) ☐ Retention by infiltration basin (INF-1) ☐ Retention by bioretention (INF-2) ☐ Retention by permeable pavement (INF-3) ☐ Partial retention by biofiltration with partial retention (PR-1) ☑ Biofiltration (BF-1) ☐ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide BMP type/description in discussion section below) ☐ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below) ☐ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion section below) ☐ Detention pond or vault for hydromodification management ☐ Other (describe in discussion section below)	
Purpose: ☑ Pollutant control only ☐ Hydromodification control only ☐ Combined pollutant control and hydromodification control ☐ Pre-treatment/forebay for another structural BMP ☐ Other (describe in discussion section below)	
Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Andrew J. Kann Omega Engineering 858-634-8620
Who will be the final owner of this BMP?	Lawrence, Nora
Who will maintain this BMP into perpetuity?	Lawrence, Nora
What is the funding mechanism for maintenance?	U-Store It Self Storage

Project Name: National Ave Self-Storage

Form I-6 Page of (Copy as many as needed)
Structural BMP ID No. BMP-2
Construction Plan Sheet No.
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs):

SD	**City of San Diego** **Development Services** 1222 First Ave., MS-501 San Diego, CA 92101	# Permanent BMP Construction **Self Certification Form**	**FORM** **DS-563** **December 2016**

Date Prepared: 07/23/2018	Project No./Drawing No.:
Project Applicant: Omega-Engineering Consultants	Phone: (858) 634-8627
Project Address: 2209 National Avenue, San Diego CA, 92113	
Project Name: U-STOR-IT (Barrio Logan) LLC	

The purpose of this form is to verify that the site improvements for the project, identified above, have been constructed in conformance with the approved Storm Water Standards Manual documents and drawings.

This form must be completed by the engineer and submitted prior to final inspection of the construction permit. Completion and submittal of this form is required for Priority Development Projects in order to comply with the City's Storm Water ordinances and applicable San Diego Regional MS4 Permit. Final inspection for occupancy and/or release of grading or public improvement bonds may be delayed if this form is not submitted and approved by the City of San Diego.

Certification:

As the professional in responsible charge for the design of the above project, I certify that I have inspected all constructed Low Impact Development (LID) site design, source control, hydromodification, and treatment control BMP's required per the Storm Water Standards Manual; and that said BMP's have been constructed in compliance with the approved plans and all applicable specifications, permits, ordinances and San Diego Regional MS4 Permit.

I understand that this BMP certification statement does not constitute an operation and maintenance verification.

Signature: ______________________________

Date of Signature: ______________________________

Printed Name: ______________________________

Title: ______________________________

Phone No. ______________________________

Engineer's Stamp

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Attachment 1
Backup For PDP Pollutant Control BMPs

This is the cover sheet for Attachment 1.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Project Name: U-STOR-IT (Barrio Logan) LLC

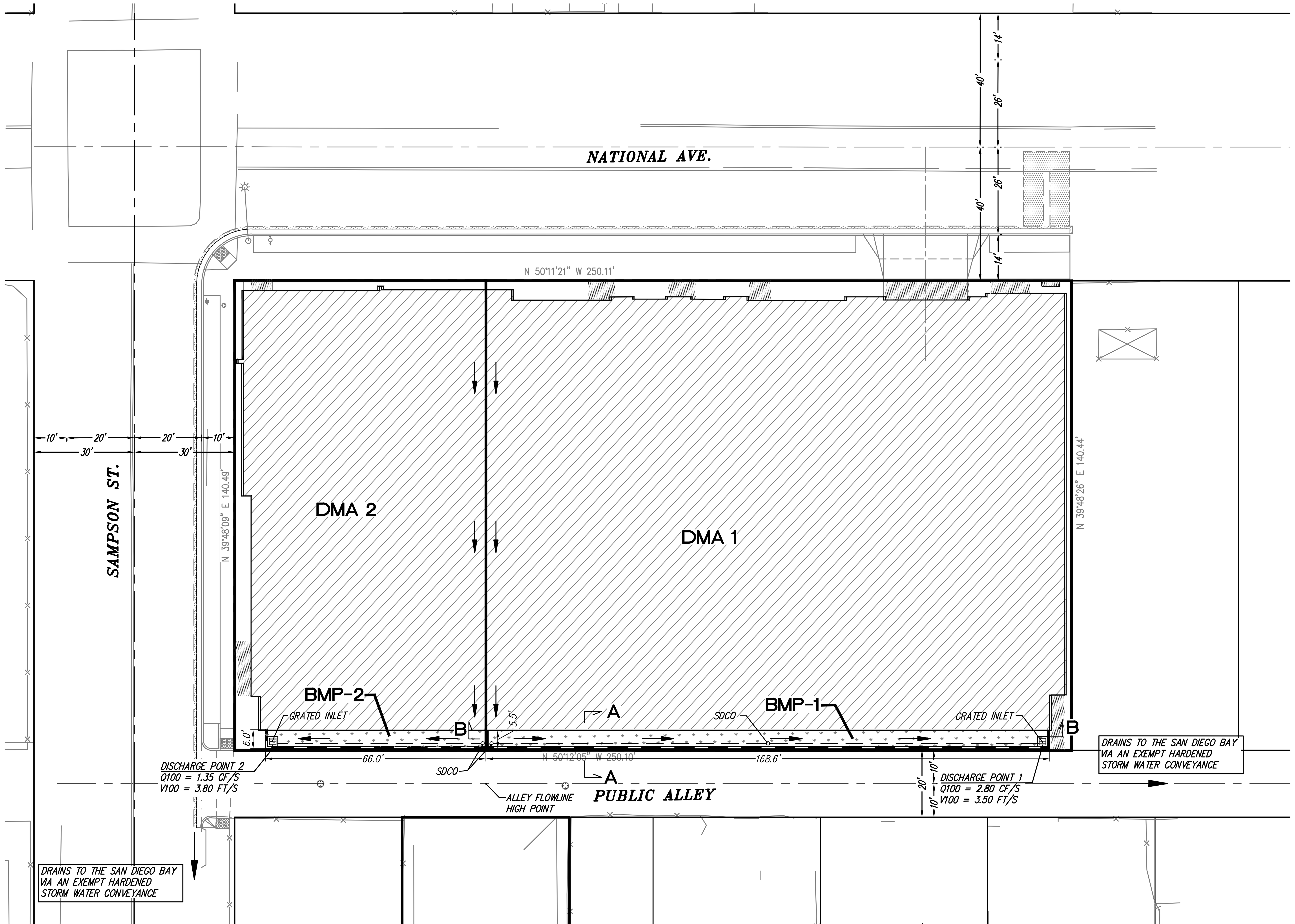
Indicate which Items are Included:

Attachment Sequence	Contents	Checklist
Attachment 1a	DMA Exhibit (Required) See DMA Exhibit Checklist.	☑ Included
Attachment 1b	Tabular Summary of DMAs Showing DMA ID matching DMA Exhibit, DMA Area, and DMA Type (Required)* *Provide table in this Attachment OR on DMA Exhibit in Attachment 1a	☑ Included on DMA Exhibit in Attachment 1a ☐ Included as Attachment 1b, separate from DMA Exhibit
Attachment 1c	Form I-7, Harvest and Use Feasibility Screening Checklist (Required unless the entire project will use infiltration BMPs) Refer to Appendix B.3-1 of the BMP Design Manual to complete Form I-7.	☑ Included ☐ Not included because the entire project will use infiltration BMPs
Attachment 1d	Infiltration Feasibility Information. Contents of Attachment 1d depend on the infiltration condition: • No Infiltration Condition: ○ Infiltration Feasibility Condition Letter (***Note: must be stamped and signed by licensed geotechnical engineer***) ○ Form I-8A (optional) ○ Form I-8B (optional) • Partial Infiltration Condition: ○ Infiltration Feasibility Condition Letter (***Note: must be stamped and signed by licensed geotechnical engineer***) ○ Form I-8A ○ Form I-8B • Full Infiltration Condition: ○ Form I-8A ○ Form I-8B ○ Worksheet C.4-3 ○ Form I-9 Refer to Appendices C and D of the BMP Design Manual for guidance.	☑ Included ☐ Not included because the entire project will use harvest and use BMPs
Attachment 1e	Pollutant Control BMP Design Worksheets / Calculations (Required) Refer to Appendices B and E of the BMP Design Manual for structural pollutant control BMP design guidelines and site design credit calculations	☑ Included

Use this checklist to ensure the required information has been included on the DMA Exhibit:

The DMA Exhibit must identify:

- [x] Underlying hydrologic soil group
- [x] Approximate depth to groundwater
- [x] Existing natural hydrologic features (watercourses, seeps, springs, wetlands)
- [x] Critical coarse sediment yield areas to be protected
- [x] Existing topography and impervious areas
- [x] Existing and proposed site drainage network and connections to drainage offsite
- [x] Proposed grading
- [x] Proposed impervious features
- [x] Proposed design features and surface treatments used to minimize imperviousness
- [x] Drainage management area (DMA) boundaries, DMA ID numbers, and DMA areas (square footage or acreage), and DMA type (i.e., drains to BMP, self-retaining, or self-mitigating)
- [ ] Potential pollutant source areas and corresponding required source controls (see Chapter 4, Appendix E.1, and Form I-3B)
- [x] Structural BMPs (identify location, type of BMP, size/detail, and include cross-section)



DMA DATA TABLE

DMA-NO.	AREA (SF)	AREA (AC)	IMPERVIOUS %	DESIGN DCV	TREATED BY
DMA-1	24,581	0.56	97%	955 CF	BMP-1
DMA-2	10,550	0.24	97%	410 CF	BMP-2
TOTAL	35,131	0.81	97%	1,365 CF	–

BMP DATA TABLE

BMP-NO.	TREATING	REQ'D FOOTPRINT	PROPOSED AREA	NOTES
BMP-1	DMA-1	649 SF	688 SF	BIOFILTRATION BASIN
BMP-2	DMA-2	279 SF	282 SF	BIOFILTRATION BASIN

*BASIN DETAILS ON THIS SHEET

BIOFILTRATION MAINTENANCE NOTES

ACCESS:
- BIOFILTRATION BASINS BMP-1 & BMP-2 CAN BE ACCESSED VIA THE NORTH SIDE OF THE ALLEY ADJACENT TO THE SOUTH SIDE OF THE BUILDING.

PERFORATED SUBDRAIN INSPECTION:
- INSPECTION OF THE PERFORATED SUBDRAIN CAN BE ACCOMPLISHED BY REMOVING THE GRATE OF THE INLET BOX, OR BY REMOVING THE CLEANOUT CAP AT THE OPPOSITE END OF THE SUB-DRAIN. IF SILTING IS OBSERVED, THE PIPE CAN BE CLEANED BY RUNNING A GARDEN HOSE THROUGH THE PIPE STARTING AT THE CLEANOUT END.

MAINTENANCE INDICATORS AND ACTIONS FOR VEGITATED BMP'S:

TYPICAL MAINENANCE INDICATOR(S) FOR VEGETATED BMP'S	MAINTENANCE ACTIONS
ACCUMULATION OF SEDIMENT, LITTER, OR DEBRIS	REMOVE AND PROPERLY DISPOSE OF ACCUMULATED MATERIALS, WITHOUT DAMAGE TO THE VEGETATION.
POOR VEGETATION ESTABLISHMENT	MOW OR TRIM AS APPROPRIATE, BUT NOT LESS THAN THE DESIGN HEIGHT OF THE VEGETATION PER ORIGINAL PLANS WHEN APPLICABLE (E.G. A VEGETATED SWALE MAY REQUIRE A MINIMUM VEGETATION HEIGHT).
ERIOSION DUE TO CONCENTRATED IRRIGATION FLOW	REPAIR/RE-SEED/RE-PLANT ERODED AREAS AND ADJUST THE IRRIGATION SYSTEM.
EROSION DUE TO CONCENTRATED STORM WATER RUNOFF FLOW	REPAIR/RE-SEED/RE-PLANT ERODED AREAS, AND MAKE APPROPRIATE CORRECTIVE MEASURES SUCH AS ADDING EROSION CONTROL BLANKETS, ADDING STONE AT FLOW ENTRY POINTS, OR MINOR RE-GRADING TO RESTORE PROPER DRAINAGE ACCORDING TO THE ORIGINAL PLAN. IF THE ISSUE IS NOT CORRECTED BY RESTORING THE BMP TO THE ORIGINAL PLAN AND GRADE, THE CITY ENGINEER SHALL BE CONTACTED PRIOR TO ANY ADDITIONAL REPAIRS OR RECONSTRUCTION.
STANDING WATER IN VEGETATED SWALES	MAKE APPROPRIATE CORRECTIVE MEASURES SUCH AS ADJUSTING IRRIGATION SYSTEM, REMOVING OBSTRUCTIONS OF DEBRIS OR INVASIVE VEGETATION, LOOSENING OR REPLACING TOP SOIL TO ALLOW FOR BETTER INFILTRATION, OR MINOR RE-GRADING FOR PROPER DRAINAGE. IF THE ISSUE IS NOT CORRECTED BY RESTORING THE BMP TO THE ORIGINAL PLAN AND GRADE, THE CITY ENGINEER SHALL BE CONTACTED PRIOR TO ANY ADDITIONAL REPAIRS OR RECONSTRUCTION.
STANDING WATER FOR LONGER THAN 96 HOURS FOLLOWING A STORM EVENT	MAKE APPROPRIATE CORRECTIVE MEASURES SUCH AS ADJUSTING IRRIGATION SYSTEM, REMOVING OBSTRUCTIONS OF DEBRIS OR INVASIVE VEGETATION, CLEARING UNDERDRAINS (WHERE APPLICABLE), OR REPAIRING/REPLACING CLOGGED OR COMPACTED SOILS.
OBSTRUCTED INLET OR OUTLET STRUCTURE	CLEAR OBSTRUCTIONS.
DAMAGE TO STRUCTURAL COMPONENTS SUCH AS WEIRS, INLET OR OUTLET STRUCTURES	REPAIR OR REPLACE AS APPLICABLE.
RECCOMENDED EQUIPMENT: DUTCH HOE, PIPE WRENCH, SCREW DRIVER, GARDEN SPADE, SHEARS, AND A RAKE	

LEGEND:

DMA LIMITS
DRAINAGE DIRECTION ARROW
DRAINAGE MANAGEMENT AREA — DMA-#
PAVEMENT AREA
ROOFTOP AREA
BIOFILTRATION AREA
LANDSCAPE

SOURCE CONTROL BMP NOTES

ALL APPLICABLE SOURCE CONTROL BMPS SHALL BE UTILIZED

A. ALL ONSITE INLETS TO BE MARKED "NO DUMPING" OR SIMILAR AND ALL OPERATIONAL PRECAUTIONS TO AVOID NON STORM WATER DISCHARGE SHALL BE FOLLOWED PER THE CITY'S BMP DESIGN MANUAL.

B. PROPOSED REFUSE AREA WILL REMAIN COVERED AND PROTECTED FROM WIND DISPERSAL. SIGNS SHALL BE PLACED WITH WORDS "DO NOT DUMP HAZARDOUS MATERIALS OR LIQUIDS HERE" OR SIMILAR. OWNER SHALL BE RESPONSIBLE TO KEEP THE AREA CLEAN OF LITTER AND SPILLS.

C. OWNER TO BE RESPONSIBLE FOR SWEEPING PLAZAS, SIDEWALKS, AND PARKING LOTS. THIS IS TO BE DONE REGULARLY AND AS NEEDED TO PREVENT ACCUMULATION OF LITTER AND DEBRIS.

D. FIRE SPRINKLER TEST WATER SHALL BE DRAINED TO THE SANITARY SEWER

GENRAL STORM WATER NOTES

1. HYDROLOGIC SOIL GROUP IS TYPE D, PER SOILS REPORT PRODUCED BY GEOCON INCORPORATED. SEE APPENDIX 6 OF THE SWQMP.
2. GROUNDWATER IS EXPECTED TO BE BETWEEN 60'–55' BELOW EXISTING GRADE ON SITE.
3. NO EXISTING NATURAL HYDROLOGIC FEATURES
4. NO CRITICAL COARSE SEDIMENT YIELD AREAS ON SITE
5. ALL APPLICABLE SOURCE CONTROL BMPS SHALL BE IMPLEMENTED
   5A. SOURCE CONTROL NOTES TO COME IN MINISTERIAL REVIEW

BIOFILTRATION LINER NOTES

BIOFILTRATION LINER TO BE A MINIMUM PER DETAIL B-B, WITH A ROLL WIDTH TO BE NO LESS THAN 12 FT, AND ROLL LENGTH TO BE NO LESS THAN 100 FT TO MINIMIZE THE REQUIRED NUMBER OF SEAMS

CONTRACTOR SHALL VERIFY SUITABILITY OF SELECTED PRODUCT WITH CIVIL ENGINEER PRIOR TO PURCHASE AND THE LINER SHALL CARRY A MINIMUM 20 YEAR WARRANTY.

BIOFILTRATION MEDIA NOTES

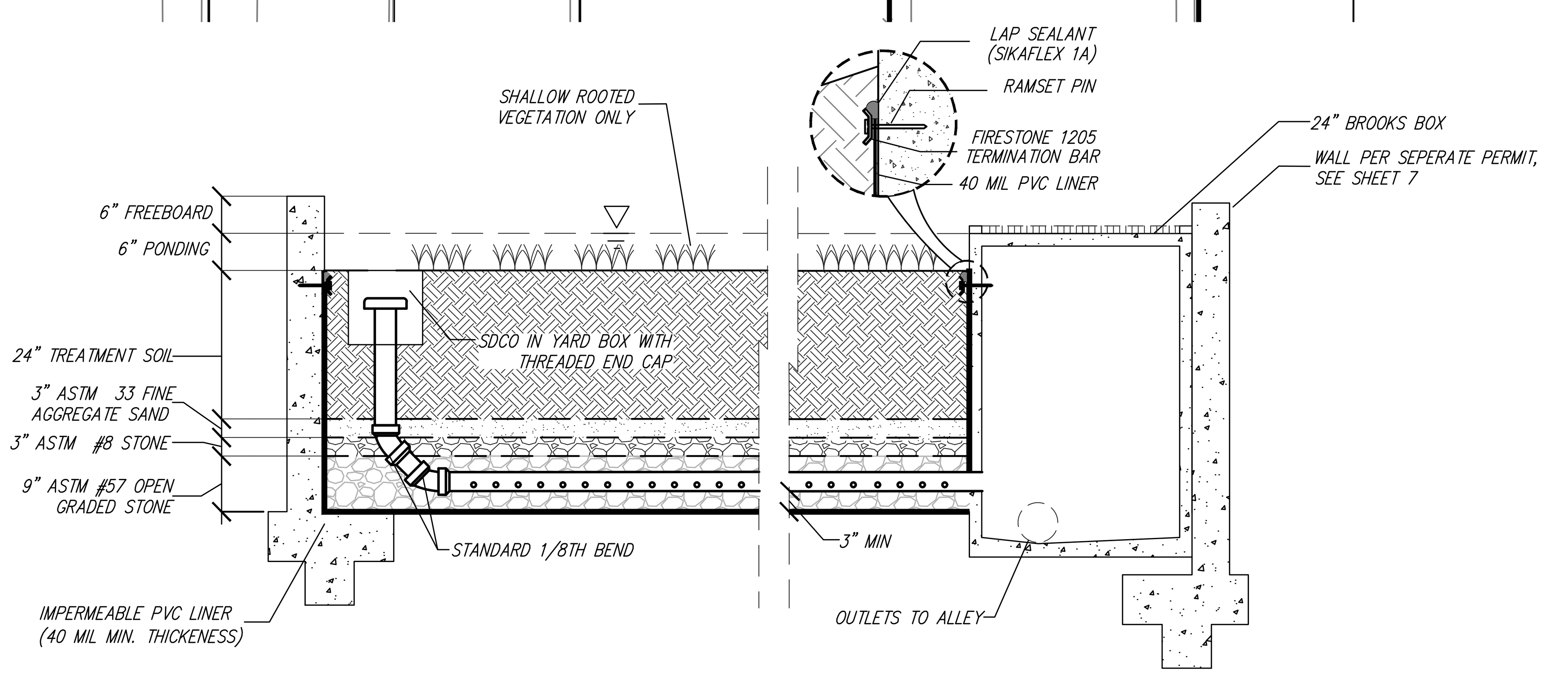
1. IN ACCORDANCE WITH BMP DESIGN MANUAL APPENDIX E ONLY SHALLOW ROOTED VEGETATION SHALL BE PLANTED WITH THE FACILITY MEDIA DEPTH AT 18".
2. ANY VEGETATION THAT IS TO BE PLANTED WITHIN THE BIOFILTRATION FACILITY THAT IS NOT DEEMED TO BE SHALLOW ROOTED, SHALL REQUIRE THE MEDIA DEPTH TO BE INCREASED TO 24" TO CONFORM WITH THE CURRENT BMP DESIGN MANUAL APPENDIX E.
3. ANY TREES PLANTED THAT ARE PROPOSED TO BE PLANTED WITHIN THE BIOFITRATION AREA WILL REQUIRE SPECIAL DETAILING AND A PLANTING MEDIA SECTION THAT IS INCREASED TO A MINIMUM OF 36" DEPTH IN ACCORDANCE WITH BMP DESIGN MANUAL APPENDIX E.

BIOFILTRATION INSPECTION SCHEDULE NOTES

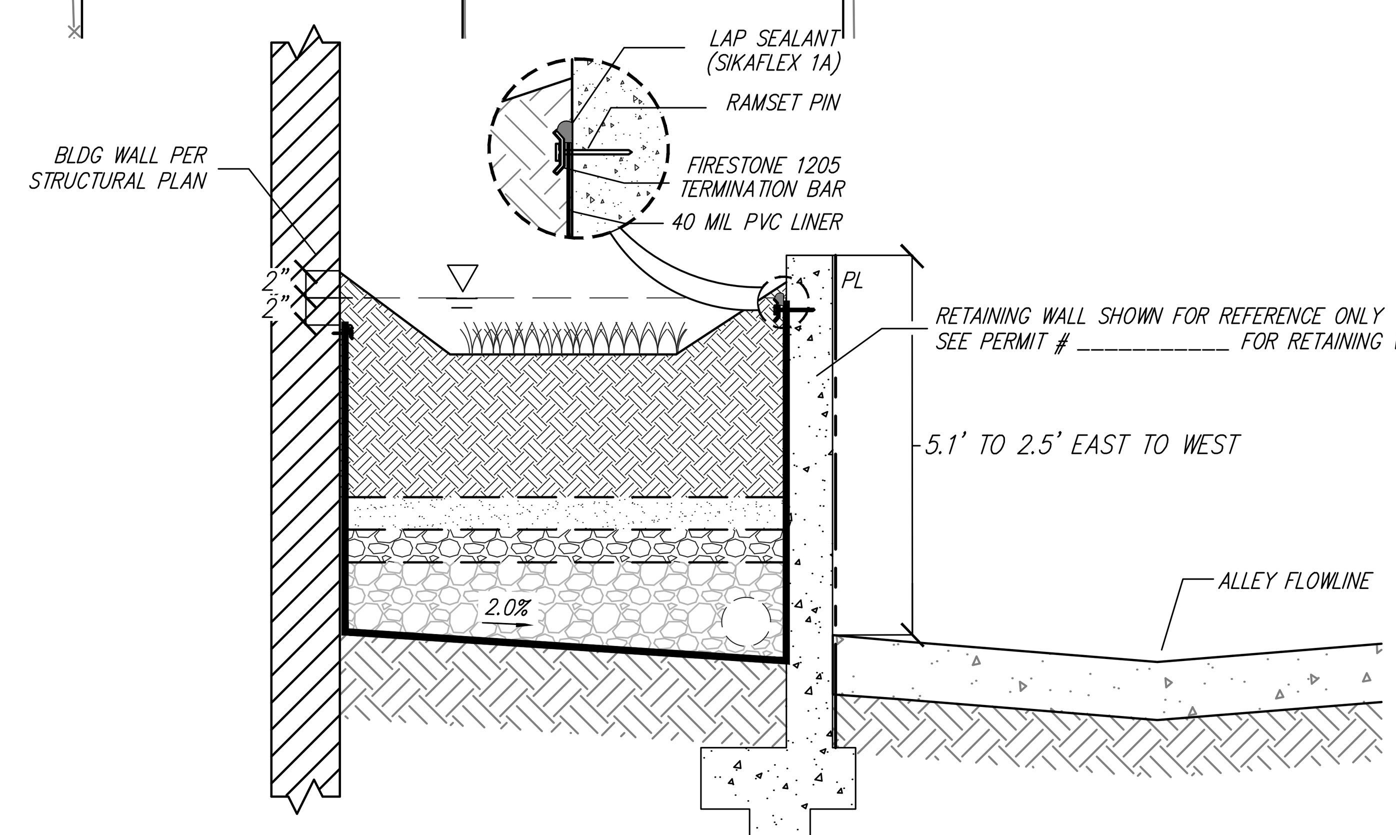
CONTRACTOR MUST CONTACT ENGINEER FOR INSPECTION(*) OF BMPS AT THE FOLLOWING STAGES OF CONSTRUCTION:

-PRIOR TO START OF CONSTRUCTION OF BIO-FILTRATION AREA
-PRIOR TO CONSTRUCTION OF OUTLET STRUCTURES
-AFTER GRADING OF BASIN AREA
-AFTER PLACEMENT OF IMPERMEABLE LINER
-AFTER PLACEMENT OF SUB-DRAIN
-AFTER THE PLACEMENT OF GRAVEL DRAINAGE LAYER
-AFTER PLACEMENT OF TREATMENT SOIL
-AFTER IRRIGATION AND LANDSCAPING ACTIVITIES

(*) SURVEY STAKES SHALL BE AVAILABLE FOR EACH INSPECTION



CROSS-SECTION 'B'-'B'
(TYP)



CROSS-SECTION 'A'-'A'
(TYP)

GRAPHICAL SCALE: 1" = 20'
0 10 20 40 60

UNDERGROUND SERVICE ALERT

CALL BEFORE YOU DIG

SECTION 4216 & 4217 OF THE GOVERNMENT CODE REQUIRES A DIG ALERT IDENTIFICAITON NUMBER BE ISSUED BEFORE A "PERMIT TO EXCAVATE" WILL BE VALID. FOR YOUR DIG ALERT I.D. NUMBER CALL UNDERGROUND SERVICE ALERT TOLL FREE @ 1-800-422-4133 TWO (2) WORKING DAYS BEFORE YOU DIG. WEB ADDRESS: WWW.DIGALERT.ORG

REGISTERED PROFESSIONAL ENGINEER — ANDREW J. KANN — No. 50940 — Exp. 9-30- — CIVIL — STATE OF CALIFORNIA

ANDREW J. KANN R.C.E. 50940 DATE

OMEGA ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVE. SUITE B
SAN DIEGO, CA 92123
PH:(858) 634-8620 FAX:(858)-634-8627

PRIVATE CONTRACT

BMP PLAN FOR:

U-STORE IT

LOTS 39 TO 48 OF BLOCK 126 OF MAP NO. 379

CITY OF SAN DIEGO, CALIFORNIA
DEVELOPMENT SERVICES DEPARTMENT
SHEET 8 OF 10 SHEETS

PROJECT NO. ______
V.T.M. –

FOR CITY ENGINEER			DATE	
DESCRIPTION	BY	APPROVED	DATE	FILMED
ORIGINAL	OEC			
AS-BUILTS				

NAD83 COORDINATES
194-1726
LAMBERT COORDINATES

CONTRACTOR ______ DATE STARTED ______
INSPECTOR ______ DATE COMPLETED ______

-08 -D

FILENAME: P:\DWG OMEGA\0400 NATIONAL AVE SELF STORAGE\ACAD\GRADING & IMPROVEMENT\GP08-DMA MAP.DWG DATE: 7/26/2018 12:36:23 PM

Project Name: U-STOR-IT (Barrio Logan) LLC

Tabular Summary of DMAs							Worksheet B-1		
DMA Unique Identifier	Area (acres)	Impervious Area (acres)	% Imp	HSG	Area Weighted Runoff Coefficient	DCV (cubic feet)	Treated By (BMP ID)	Pollutant Control Type	Drains to (POC ID)
DMA-1	0.56	0.54	97	D	0.88	955	BMP-1	BIOFILTRATION	1
DMA-2	0.24	0.24	97	D	0.88	410	BMP-2	BIOFILTRATION	2
Summary of DMA Information (Must match project description and SWQMP Narrative)									
No. of DMAs	Total DMA Area (acres)	Total Impervious Area (acres)	% Imp		Area Weighted Runoff Coefficient	Total DCV (cubic feet)	Total Area Treated (acres)		No. of POCs
2	0.81	0.78	97		0.82	1,365	0.81		2

Where: DMA = Drainage Management Area; Imp = Imperviousness; HSG = Hydrologic Soil Group; DCV= Design Capture Volume; BMP = Best Management Practice; POC = Point of Compliance; ID = identifier; No. = Number

Worksheet B.3-1: Harvest and Use Feasibility Screening

Harvest and Use Feasibility Screening	Worksheet B.3-1
1. Is there a demand for harvested water (check all that apply) at the project site that is reliably present during the wet season? ☒ Toilet and urinal flushing ☐ Landscape irrigation ☐ Other:____________________	
2. If there is a demand; estimate the anticipated average wet season demand over a period of 36 hours. Guidance for planning level demand calculations for toilet/urinal flushing and landscape irrigation is provided in Section B.3.2. [Provide a summary of calculations here] Residential: (2) * 9.3 gallons per day * 1.5 days per 36 hours Demand = 27.9 Gal/Day Landscaping: 390 Gal/(Ac*36 hours). 972 SF Low water demand landscaping for biofiltration basins. Demand = 8.70 gal/36 hours Total Demand (Gal): 36.6 Gal/36 hours 3.7 Gal/41 CF Total Demand (CF): 3.3	
3. Calculate the DCV using worksheet B-2.1. [Provide a results here] DCV = 1372 (cubic feet)	

3a. Is the 36-hour demand greater than or equal to the DCV?	3b. Is the 36-hour demand greater than 0.25DCV but less than the full DCV?	3c. Is the 36-hour demand less than 0.25DCV?
Yes / No ⇨ ⇩	Yes / No ⇨ ⇩	Yes ⇩
Harvest and use appears to be feasible. Conduct more detailed evaluation and sizing calculations to confirm that DCV can be used at an adequate rate to meet drawdown criteria.	Harvest and use may be feasible. Conduct more detailed evaluation and sizing calculations to determine feasibility. Harvest and use may only be able to be used for a portion of the site, or (optionally) the storage may need to be upsized to meet long term capture targets while draining in longer than 36 hours.	Harvest and use is considered to be infeasible.

Note: 36-hour demand calculations are for feasibility analysis only, once the feasibility analysis is complete the applicant may be allowed to use a different drawdown time provided they meet the 80 percent of average annual (long term) runoff volume performance standard.

(INFILTRATION FEASIBILITY CONDITION LETTER OR UPDATED FORM I-8 FORM TO BE SUBMITTED WITH NEXT SUBMITTAL)

Categorization of Infiltration Feasibility Condition		Form I-8	
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?** Note that it is not necessary to investigate each and every criterion in the worksheet if infiltration is precluded. Instead a letter of justification from a geotechnical professional familiar with the local conditions substantiating any geotechnical issues will be required.			
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 must be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: **Geocon Inc. encountered field infiltration rates of:** **P-1: 0.002 inches/hour (0.001 with a FOS of 2.0)** **P-2: 0.013 inches/hour (0.007 with a FOS of 2.0)** **P-3: 0.024 inches/hour (0.012 with a FOS of 2.0)** **P-4: 0.002 inches/hour (0.001 with a FOS of 2.0)** **These tests results in an average of about 0.010 inches/hour (0.005 with a FOS of 2.0).** **The results of the infiltration tests indicate rates of less than 0.5 inches per hour (including the factor of safety); therefore, infiltration is not considered feasible.** 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 must be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: **Undocumented fill and Old Paralic Deposits underlie the property. Water that would be allowed to infiltrate would migrate laterally outside of the property limits to the existing right-of-ways (located to the south) and toward existing and proposed structures (located to the north and west). The Old Paralic Deposits possess hydroconsolidation potential ranging from 0.1 to 3 percent. We expect about 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation is up to about 4¼ inches. Therefore, infiltration in regards the geologic hazards would be considered infeasible. Liners and subdrains should be incorporated into the design and construction of the planned storm water devices to prevent saturation and potential hydroconsolidation of the soil supporting the existing or proposed development.** 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">Form I-8 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 must 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:

Geocon Inc. did not encounter groundwater during the drilling operation at the property to the maximum depth of 50 feet or an elevation of 10 feet MSL. Groundwater is anticipated to be present at an elevation of 0 to 5 feet MSL. The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. Therefore, infiltration due to groundwater elevations would be considered feasible above an elevation of 15 feet MSL.

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 must 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 full infiltration would cause water balance issues including change of ephemeral streams or discharge of contaminated water 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>NO 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 Agency/Jurisdictions to substantiate findings

Form I-8 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 must be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

Geocon Inc. encountered field infiltration rates of:
P-1: 0.002 inches/hour (0.001 with a FOS of 2.0)
P-2: 0.013 inches/hour (0.007 with a FOS of 2.0)
P-3: 0.024 inches/hour (0.012 with a FOS of 2.0)
P-4: 0.002 inches/hour (0.001 with a FOS of 2.0)
These tests results in an average of about 0.010 inches/hour (0.005 with a FOS of 2.0).
The results of the infiltration tests indicate rates of less than 0.5 inches per hour (including the factor of safety); therefore, infiltration is not considered 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.

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 must be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X

Provide basis:

Undocumented fill and Old Paralic Deposits underlie the property. Water that would be allowed to infiltrate would migrate laterally outside of the property limits to the existing right-of-ways (located to the south) and toward existing and proposed structures (located to the north and west). The Old Paralic Deposits possess hydroconsolidation potential ranging from 0.1 to 3 percent. We expect about 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation is up to about 4¼ inches. Therefore, infiltration in regards the geologic hazards would be considered infeasible. Liners and subdrains should be incorporated into the design and construction of the planned storm water devices to prevent saturation and potential hydroconsolidation of the soil supporting the existing or proposed development.

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.

Form I-8 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 must be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

Geocon Inc. did not encounter groundwater during the drilling operation at the property to the maximum depth of 50 feet or an elevation of 10 feet MSL. Groundwater is anticipated to be present at an elevation of 0 to 5 feet MSL. The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. Therefore, infiltration due to groundwater elevations would be considered feasible above an elevation of 15 feet MSL.

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
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question must be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

Infiltration of stormwater would not be anticipated to violate downstream water rights.

Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	NO INFILTRATION

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

Worksheet B.5-1: Sizing Method for Pollutant Removal Criteria BMP-1

Sizing Method for Pollutant Removal Criteria		Worksheet B.5-1	
1	Area draining to the BMP	24,581	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.88	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	955	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	6	inches
6	Media thickness [18 inches minimum], also add mulch layer and washed ASTM 33 fine aggregate sand thickness to this line for sizing calculations	24	inches
7	Aggregate storage (also add ASTM No 8 stone) above underdrain invert (12 inches typical) – use 0 inches if the aggregate is not over the entire bottom surface area	12	inches
8	Aggregate storage below underdrain invert (3 inches minimum) – use 0 inches if the aggregate is not over the entire bottom surface area	3	inches
9	Freely drained pore storage of the media	0.2	in/in
10	Porosity of aggregate storage	0.4	in/in
11	Media filtration rate to be used for sizing (maximum filtration rate of 5 in/hr. with no outlet control; if the filtration rate is controlled by the outlet use the outlet controlled rate (includes infiltration into the soil and flow rate through the outlet structure) which will be less than 5 in/hr.)	5	in/hr.
Baseline Calculations			
12	Allowable routing time for sizing	6	hours
13	Depth filtered during storm [Line 11 x Line 12]	30	inches
14	Depth of Detention Storage [Line 5 + (Line 6 x Line 9) + (Line 7 x Line 10) + (Line 8 x Line 10)]	16.8	inches
15	Total Depth Treated [Line 13 + Line 14]	46.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	1,433	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	367	sq. ft.
Option 2 - Store 0.75 of remaining DCV in pores and ponding			
18	Required Storage (surface + pores) Volume [0.75 x Line 4]	716	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	511	sq. ft.
Footprint of the BMP			
20	BMP Footprint Sizing Factor (Default 0.03 or an alternative minimum footprint sizing factor from Line 11 in Worksheet B.5-4)	0.03	
21	Minimum BMP Footprint [Line 1 x Line 2 x Line 20]	649	sq. ft.
22	Footprint of the BMP = Maximum (Minimum (Line 17, Line 19), Line 21)	649	sq. ft.
23	Provided BMP Footprint	688	sq. ft.
24	Is Line 23 ≥ Line 22? If Yes, then footprint criterion is met. If No, increase the footprint of the BMP.	☒ Yes	☐ No

Worksheet B.5-1: Sizing Method for Pollutant Removal Criteria BMP-2

Sizing Method for Pollutant Removal Criteria		Worksheet B.5-1	
1	Area draining to the BMP	10,550	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.88	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	410	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	6	inches
6	Media thickness [18 inches minimum], also add mulch layer and washed ASTM 33 fine aggregate sand thickness to this line for sizing calculations	24	inches
7	Aggregate storage (also add ASTM No 8 stone) above underdrain invert (12 inches typical) – use 0 inches if the aggregate is not over the entire bottom surface area	12	inches
8	Aggregate storage below underdrain invert (3 inches minimum) – use 0 inches if the aggregate is not over the entire bottom surface area	3	inches
9	Freely drained pore storage of the media	0.2	in/in
10	Porosity of aggregate storage	0.4	in/in
11	Media filtration rate to be used for sizing (maximum filtration rate of 5 in/hr. with no outlet control; if the filtration rate is controlled by the outlet use the outlet controlled rate (includes infiltration into the soil and flow rate through the outlet structure) which will be less than 5 in/hr.)	5	in/hr.
Baseline Calculations			
12	Allowable routing time for sizing	6	hours
13	Depth filtered during storm [Line 11 x Line 12]	30	inches
14	Depth of Detention Storage [Line 5 + (Line 6 x Line 9) + (Line 7 x Line 10) + (Line 8 x Line 10)]	16.8	inches
15	Total Depth Treated [Line 13 + Line 14]	46.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	615	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	158	sq. ft.
Option 2 - Store 0.75 of remaining DCV in pores and ponding			
18	Required Storage (surface + pores) Volume [0.75 x Line 4]	308	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	220	sq. ft.
Footprint of the BMP			
20	BMP Footprint Sizing Factor (Default 0.03 or an alternative minimum footprint sizing factor from Line 11 in Worksheet B.5-4)	0.03	
21	Minimum BMP Footprint [Line 1 x Line 2 x Line 20]	279	sq. ft.
22	Footprint of the BMP = Maximum (Minimum (Line 17, Line 19), Line 21)	279	sq. ft.
23	Provided BMP Footprint	282	sq. ft.
24	Is Line 23 ≥ Line 22? If Yes, then footprint criterion is met. If No, increase the footprint of the BMP.	☒ Yes	☐ No

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Attachment 2
Backup for PDP Hydromodification Control Measures

This is the cover sheet for Attachment 2.

☑ Mark this box if this attachment is empty because the project is exempt from PDP hydromodification management requirements.

Project Name: U-STOR-IT (Barrio Logan) LLC

Indicate which Items are Included:

Attachment Sequence	Contents	Checklist
Attachment 2a	Hydromodification Management Exhibit (Required)	☐ Included See Hydromodification Management Exhibit Checklist.
Attachment 2b	Management of Critical Coarse Sediment Yield Areas (WMAA Exhibit is required, additional analyses are optional) See Section 6.2 of the BMP Design Manual.	☐ Exhibit showing project drainage boundaries marked on WMAA Critical Coarse Sediment Yield Area Map (Required) Optional analyses for Critical Coarse Sediment Yield Area Determination ☐ 6.2.1 Verification of Geomorphic Landscape Units Onsite ☐ 6.2.2 Downstream Systems Sensitivity to Coarse Sediment ☐ 6.2.3 Optional Additional Analysis of Potential Critical Coarse Sediment Yield Areas Onsite
Attachment 2c	Geomorphic Assessment of Receiving Channels (Optional) See Section 6.3.4 of the BMP Design Manual.	☐ Not Performed ☐ Included ☐ Submitted as separate stand-alone document
Attachment 2d	Flow Control Facility Design and Structural BMP Drawdown Calculations (Required) Overflow Design Summary for each structural BMP See Chapter 6 and Appendix G of the BMP Design Manual	☐ Included ☐ Submitted as separate stand-alone document

Project Name: U-STOR-IT (Barrio Logan) LLC

Use this checklist to ensure the required information has been included on the Hydromodification Management Exhibit:

The Hydromodification Management Exhibit must identify:

- ☐ Underlying hydrologic soil group
- ☐ Approximate depth to groundwater
- ☐ Existing natural hydrologic features (watercourses, seeps, springs, wetlands)
- ☐ Critical coarse sediment yield areas to be protected OR provide a separate map showing that the project site is outside of any critical coarse sediment yield areas
- ☐ Existing topography
- ☐ Existing and proposed site drainage network and connections to drainage offsite
- ☐ Proposed grading
- ☐ Proposed impervious features
- ☐ Proposed design features and surface treatments used to minimize imperviousness
- ☐ Point(s) of Compliance (POC) for Hydromodification Management
  Existing and proposed drainage boundary and drainage area to each POC (when necessary, create separate exhibits for pre-development and post-project conditions)
- ☐ Structural BMPs for hydromodification management (identify location, type of BMP, and size/detail).

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Attachment 3
Structural BMP Maintenance Information

This is the cover sheet for Attachment 3.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

Indicate which Items are Included:

Attachment Sequence	Contents	Checklist
Attachment 3	Maintenance Agreement (Form DS-3247) (when applicable)	☑ Included ☐ Not applicable

Project Name: U-STOR-IT (Barrio Logan) LLC

Use this checklist to ensure the required information has been included in the Structural BMP Maintenance Information Attachment:

Attachment 3: For private entity operation and maintenance, Attachment 3 must include a Storm Water Management and Discharge Control Maintenance Agreement (Form DS-3247). The following information must be included in the exhibits attached to the maintenance agreement:

- ☑ Vicinity map
- ☑ Site design BMPs for which DCV reduction is claimed for meeting the pollutant control obligations.
- ☑ BMP and HMP location and dimensions
- ☑ BMP and HMP specifications/cross section/model
- ☑ Maintenance recommendations and frequency
- N/A ☐ LID features such as (permeable paver and LS location, dim, SF).

The City of SAN DIEGO

RECORDING REQUESTED BY:
THE CITY OF SAN DIEGO AND
WHEN RECORDED MAIL TO:

National Ave. Self Storage

2209 National Avenue

San Diego CA, 92113

(THIS SPACE IS FOR RECORDER'S USE ONLY)

STORM WATER MANAGEMENT AND DISCHARGE CONTROL MAINTENANCE AGREEMENT

APPROVAL NUMBER:	ASSESSORS PARCEL NUMBER:	PROJECT NUMBER:
______________	538-690-29,34,37	______________

This agreement is made by and between the City of San Diego, a municipal corporation [City] and ________________

____,

the owner or duly authorized representative of the owner [Property Owner] of property located at

2209 National Avenue, San Diego, CA 92113

(PROPERTY ADDRESS)

and more particularly described as: Lots 3, 4, and 39 - 48, in block 126 according to map thereof No. 379 Filed in the county of San Diego, State of California, on Ocotober 30th 1886.

(LEGAL DESCRIPTION OF PROPERTY)

in the City of San Diego, County of San Diego, State of California.

Property Owner is required pursuant to the City of San Diego Municipal Code, Chapter 4, Article 3, Division 3, Chapter 14, Article 2, Division 2, and the Land Development Manual, Storm Water Standards to enter into a Storm Water Management and Discharge Control Maintenance Agreement [Maintenance Agreement] for the installation and maintenance of Permanent Storm Water Best Management Practices [Permanent Storm Water BMP's] prior to the issuance of construction permits. The Maintenance Agreement is intended to ensure the establishment and maintenance of Permanent Storm Water BMP's onsite, as described in the attached exhibit(s), the project's Storm Water Quality Management Plan [SWQMP] and Grading and/or Improvement Plan Drawing No(s), or Building Plan Project No(s): ______________________.

Property Owner wishes to obtain a building or engineering permit according to the Grading and/or Improvement Plan Drawing No(s) or Building Plan Project No(s): ______________________.

Continued on Page 2

NOW, THEREFORE, the parties agree as follows:

1. Property Owner shall have prepared, or if qualified, shall prepare an Operation and Maintenance Procedure [OMP] for Permanent Storm Water BMP's, satisfactory to the City, according to the attached exhibit(s), consistent with the Grading and/or Improvement Plan Drawing No(s), or Building Plan Project No(s): ___________.

2. Property Owner shall install, maintain and repair or replace all Permanent Storm Water BMP's within their property, according to the OMP guidelines as described in the attached exhibit(s), the project's SWQMP and Grading and/or Improvement Plan Drawing No(s), or Building Plan Project No(s) ____________.

3. Property Owner shall maintain operation and maintenance records for at least five (5) years. These records shall be made available to the City for inspection upon request at any time.

This Maintenance Agreement shall commence upon execution of this document by all parties named hereon, and shall run with the land.

Executed by the City of San Diego and by Property Owner in San Diego, California.

See Attached Exhibit(s): To Be Provided

____
(Owner Signature)

____
(Print Name and Title)

____
(Company/Organization Name)

06/27/2018
(Date)

THE CITY OF SAN DIEGO

APPROVED:

____
(City Control Engineer Signature)

____
(Print Name)

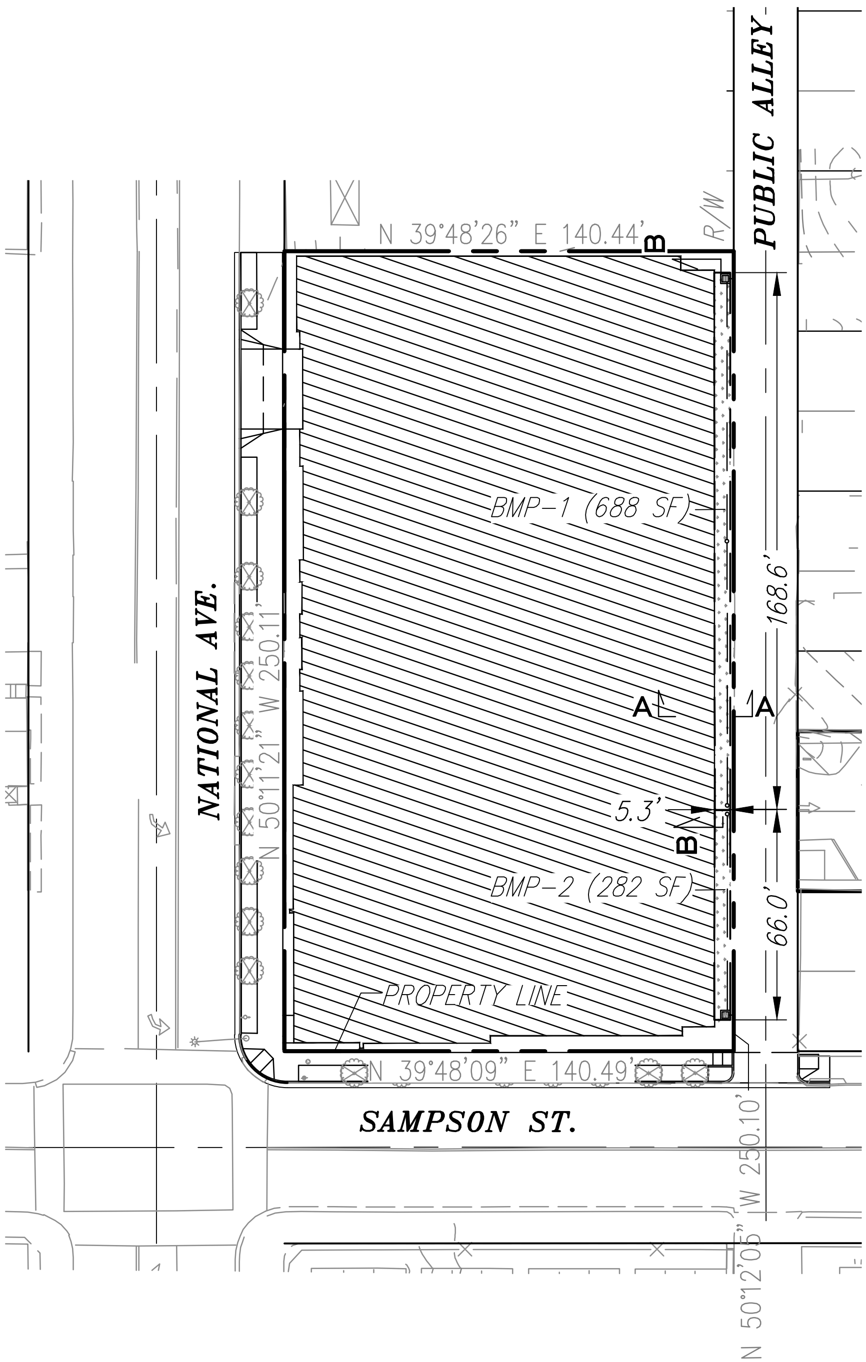
____
(Date)

NOTE: ALL SIGNATURES MUST INCLUDE NOTARY ACKNOWLEDGMENTS PER CIVIL CODE SEC. 1180 ET.SEQ.

Reset Button Page 2

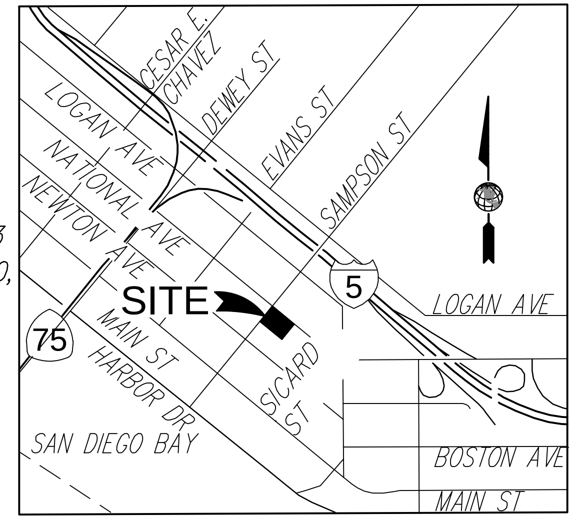
SWMDCMA

EXHIBIT "A"



SITE ADDRESS AND APN:

2209 NATIONAL AVE SAN DIEGO, CA 92113
APN'S: 538-690-29-00, 538-690-34-00, AND 538-690-37-00



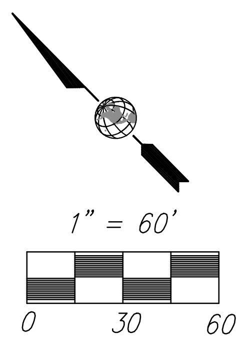
LEGEND

ROOF AREA

BIOFILTRATION BASIN (970 SF)

LEGAL DESCRIPTION

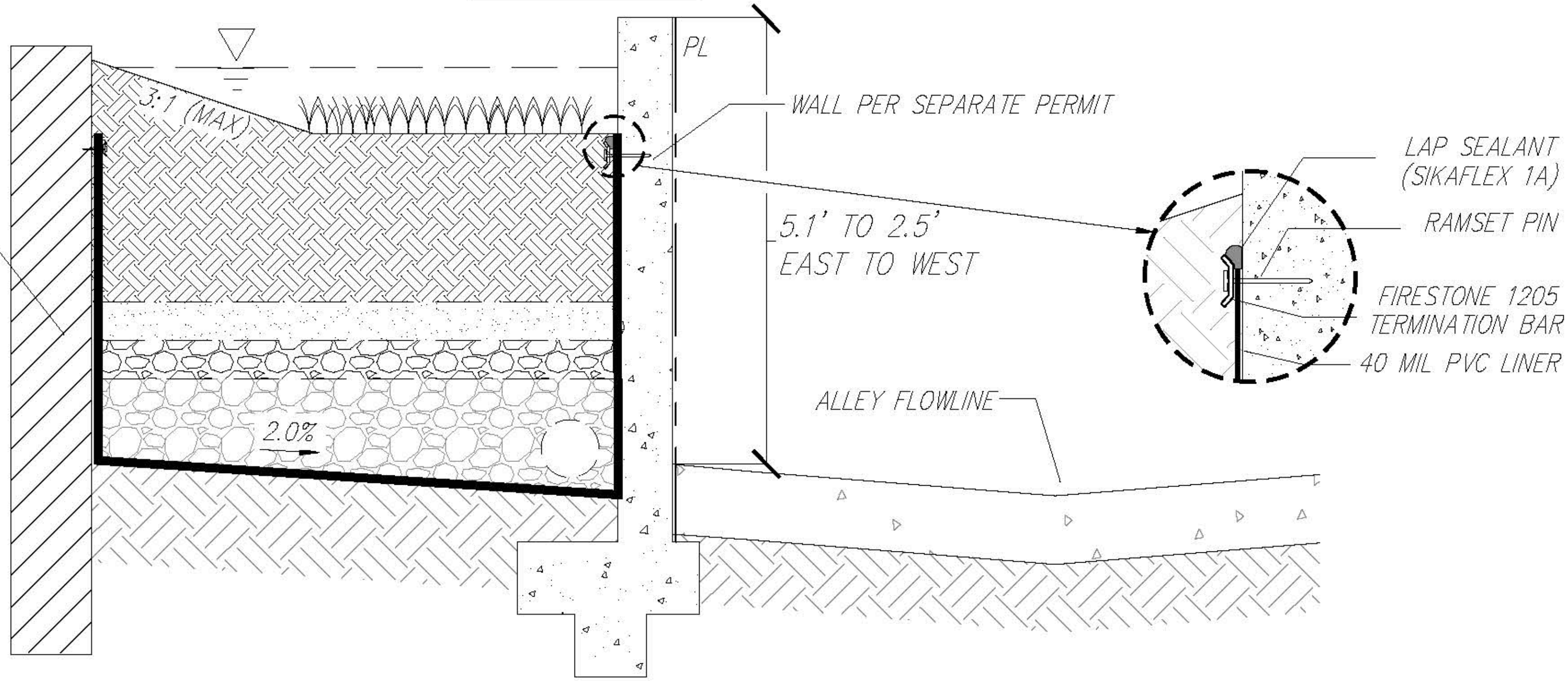
LOTS 3 AND 4 AND LOTS 39 THROUGH 48, IN BLOCK 126, OF THE SAN DIEGO LAND AND TOWN COMPANY'S ADDITION, IN THE CITY OF SAN DIEGO, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, ACCORDING TO MAP THEREOF NO. 379, FILED IN THE OFFICE OF THE COUNTY RECORD OF SAN DIEGO COUNTY, OCTOBER 30, 1886.



OMEGA
ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVE. SUITE B
SAN DIEGO, CA 92123
PH:(858) 634-8620

SWMDCMA

EXHIBIT "B"



BIOFILTRATION SECTION A–A

NOT TO SCALE

BIOFILTRATION CROSS SECTION AND OUTLET CONTROL FACILITY

BIOFILTRATION AREA & GRAVEL STORAGE FACILITY
NOT TO SCALE

INSPECTION & MAINTENANCE

BIOFILTRATION FACILITY SHALL BE MAINTAINED AT A MINIMUM OF ONCE EVERY THREE MONTHS. MAINTENANCE SHALL INCLUDE THE REMOVAL OF ALL TRASH OR DEBRIS, AND TRIMMING/PROPER DISPOSAL OF VEGETATION. THE MAINTENANCE SHALL ALSO INCLUDE INSURING THAT THE SOIL IS NOT EXCESSIVELY COMPACTED. IF SO, LOCALIZED SCARIFICATION OR REMOVAL AND REINSTALLATION OF THE TOP LAYER OF PLANTING SOIL MAY BE REQUIRED. SHOULD EXCESSIVE PONDING BE OBSERVED DURING LIGHT RAINFALL EVENTS (LESS THAN 0.60") LOCALIZED SOIL MITIGATION MEASURES MAY BE REQUIRED. PONDING UNDER LARGER RAINFALL EVENTS IS EXPECTED. IF PONDING REMAINS 48 HOURS AFTER ANY RAINFALL EVENT, SCARIFICATION OF THE SURFACE LAYER MAY BE NECESSARY. THE BIORETENTION AREA SHALL BE INSPECTED AFTER EACH RAINFALL EVENT.

THE GRAVEL STORAGE FACILITY SHALL BE INSPECTED ON A YEARLY BASIS BY REMOVING ONE OF THE CLEANOUT GRATES AND CHECKING FOR THE PRESENCE OF PONDING MORE THAN 2 DAYS AFTER RAINFALL EVENTS.

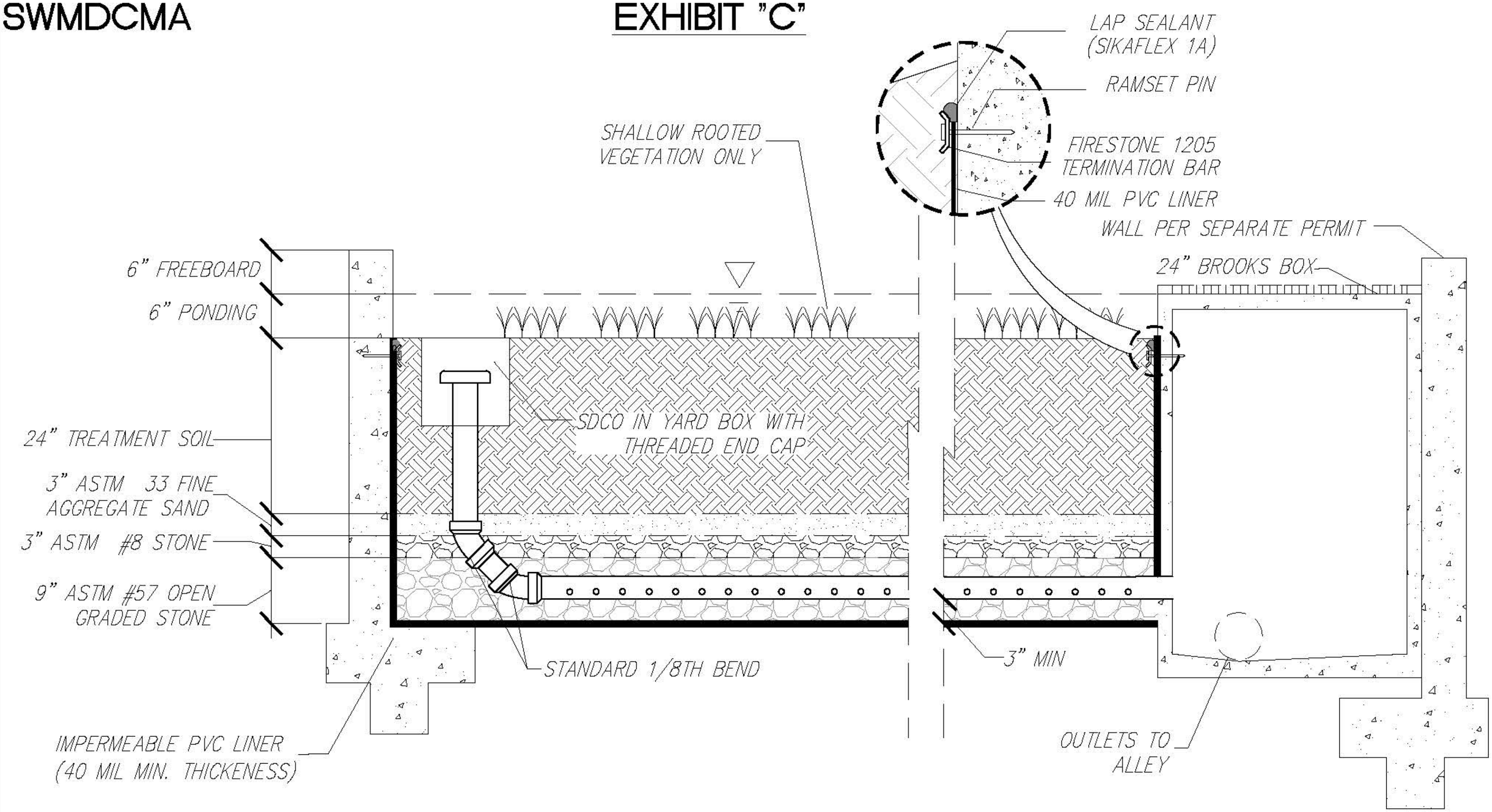
SOIL & ROCK NOTE

TREATMENT SOIL SHALL BE: 85–88% WASHED SAND, 8–12% FINES (SILT & CLAY) 3–5% ORGANIC MATTER. LONG TERM INFILTRATION RATE OF 5.0 IN/HR ROCK COURSE SHALL BE CLASS II PERMEABLE (40% POROSITY)

OMEGA
ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVE. SUITE B
SAN DIEGO, CA 92123
PH:(858) 634-8620

SWMDCMA

EXHIBIT "C"



CROSS-SECTION 'B'-'B'

(TYP)

OMEGA
ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVE. SUITE B
SAN DIEGO, CA 92123
PH:(858) 634-8620

SWMDCMA

EXHIBIT "D"

SITE DESIGN, SOURCE CONTROL AND POLLUTANT CONTROL BMP OPERATION + MAINTENANCE PROCEDURE

STORM WATER MANAGEMENT AND DISCHARGE CONTROL MAINTENANCE AGREEMENT APPROVAL NO.:

O&M RESPONSIBLE PARTY DESIGNEE: PROPERTY OWNER: LAWRENCE NORA

BMP DESCRIPTION	INSPECTION FREQUENCY	MAINTENANCE FREQUENCY	MAINTENANCE METHOD	QUANTITY	INCLUDED IN O&M MANUAL				SHEET NUMBER(S)
SITE DESIGN ELEMENTS	MONTHLY	AS NEEDED		N/A	X	YES		NO	8
DESCRIPTION: SD-7									
SOURCE CONTROL ELEMENTS		AS NEEDED		N/A	X	YES		NO	8
DESCRIPTION: SC-1,2,5, & 6									
POLLUTANT CONTROL BMP(S)	AFTER RAIN, OR			2	X	YES		NO	8
DESCRIPTION: BMP-1 & BMP-2	MAX 3 MONTHS								
HMP FACILITY (IF SEPARATE)						YES	X	NO	N/A
DESCRIPTION: N/A									
HMP EXEMPT YES									

ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVE. SUITE B
SAN DIEGO, CA 92123
PH:(858) 634-8620

Attachment 4
Copy of Plan Sheets Showing Permanent Storm Water BMPs

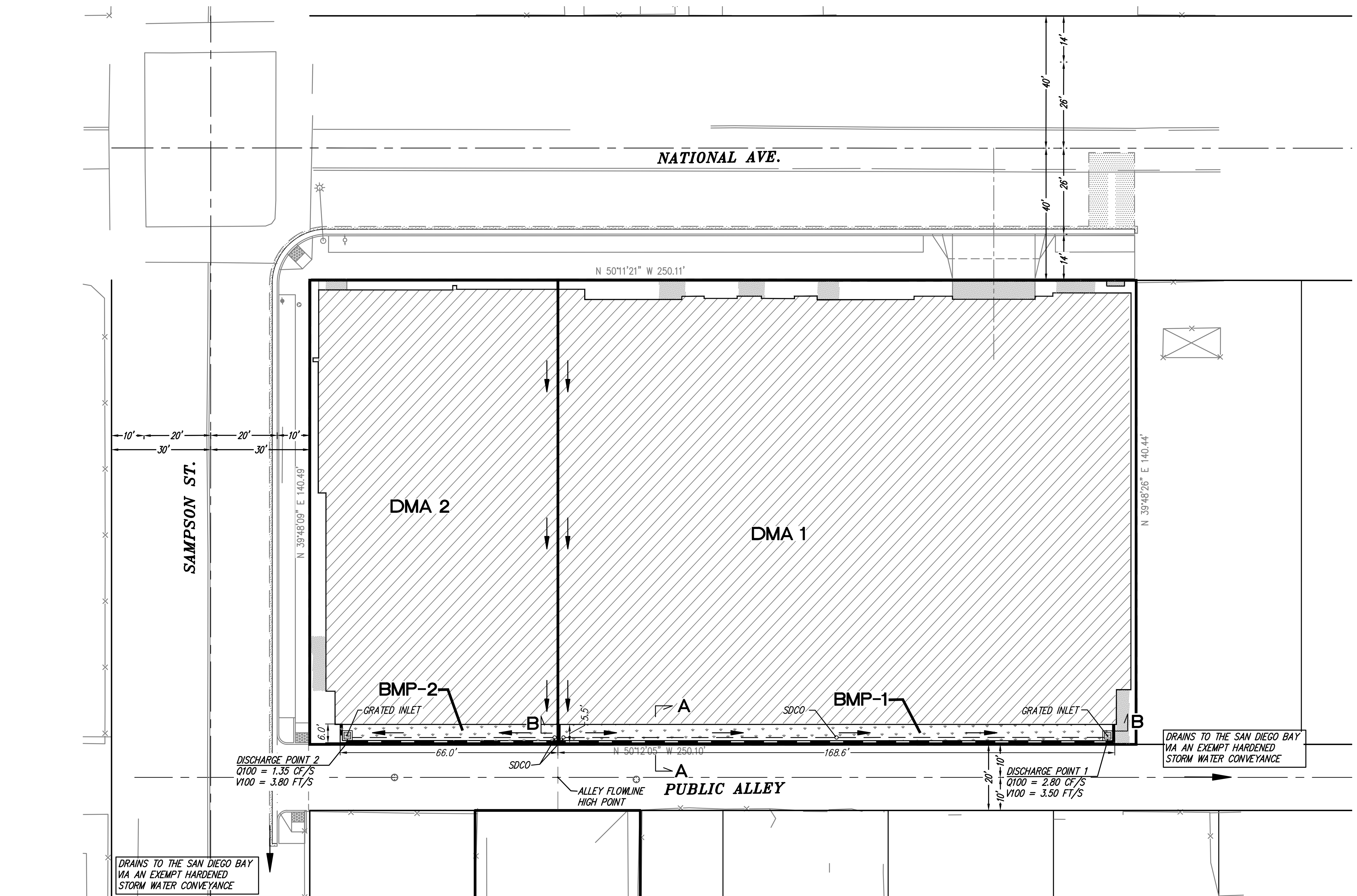
This is the cover sheet for Attachment 4.

Project Name: U-STOR-IT (Barrio Logan) LLC

Use this checklist to ensure the required information has been included on the plans:

The plans must identify:

- ☑ Structural BMP(s) with ID numbers matching Form I-6 Summary of PDP Structural BMPs
- ☑ The grading and drainage design shown on the plans must be consistent with the delineation of DMAs shown on the DMA exhibit
- ☑ Details and specifications for construction of structural BMP(s)
- ☐ Signage indicating the location and boundary of structural BMP(s) as required by the City Engineer
- ☑ How to access the structural BMP(s) to inspect and perform maintenance
- ☑ Features that are provided to facilitate inspection (e.g., observation ports, cleanouts, silt posts, or other features that allow the inspector to view necessary components of the structural BMP and compare to maintenance thresholds)
- N/A ☐ Manufacturer and part number for proprietary parts of structural BMP(s) when applicable
- ☑ Maintenance thresholds specific to the structural BMP(s), with a location-specific frame of reference (e.g., level of accumulated materials that triggers removal of the materials, to be identified based on viewing marks on silt posts or measured with a survey rod with respect to a fixed benchmark within the BMP)
- ☑ Recommended equipment to perform maintenance
- N/A ☐ When applicable, necessary special training or certification requirements for inspection and maintenance personnel such as confined space entry or hazardous waste management
- ☐ Include landscaping plan sheets showing vegetation requirements for vegetated structural BMP(s)
- ☑ All BMPs must be fully dimensioned on the plans
- N/A ☐ When proprietary BMPs are used, site specific cross section with outflow, inflow and model number shall be provided. Broucher photocopies are not allowed.



DMA DATA TABLE					
DMA-NO.	AREA (SF)	AREA (AC)	IMPERVIOUS %	DESIGN DCV	TREATED BY
DMA-1	24,581	0.56	97%	955 CF	BMP-1
DMA-2	10,550	0.24	97%	410 CF	BMP-2
TOTAL	35,131	0.81	97%	1,365 CF	–

BMP DATA TABLE				
BMP-NO.	TREATING	REQ'D FOOTPRINT	PROPOSED AREA	NOTES
BMP-1	DMA-1	649 SF	688 SF	BIOFILTRATION BASIN
BMP-2	DMA-2	279 SF	282 SF	BIOFILTRATION BASIN

*BASIN DETAILS ON THIS SHEET

BIOFILTRATION MAINTENANCE NOTES

ACCESS:

- BIOFILTRATION BASINS BMP-1 & BMP-2 CAN BE ACCESSED VIA THE NORTH SIDE OF THE ALLEY ADJACENT TO THE SOUTH SIDE OF THE BUILDING.

PERFORATED SUBDRAIN INSPECTION:

- INSPECTION OF THE PERFORATED SUBDRAIN CAN BE ACCOMPLISHED BY REMOVING THE GRATE OF THE INLET BOX, OR BY REMOVING THE CLEANOUT CAP AT THE OPPOSITE END OF THE SUB-DRAIN. IF SILTING IS OBSERVED, THE PIPE CAN BE CLEANED BY RUNNING A GARDEN HOSE THROUGH THE PIPE STARTING AT THE CLEANOUT END.

MAINTENANCE INDICATORS AND ACTIONS FOR VEGITATED BMP'S:

TYPICAL MAINENANCE INDICATOR(S) FOR VEGETATED BMP'S	MAINTENANCE ACTIONS
ACCUMULATION OF SEDIMENT, LITTER, OR DEBRIS	REMOVE AND PROPERLY DISPOSE OF ACCUMULATED MATERIALS, WITHOUT DAMAGE TO THE VEGETATION.
POOR VEGETATION ESTABLISHMENT	MOW OR TRIM AS APPROPRIATE, BUT NOT LESS THAN THE DESIGN HEIGHT OF THE VEGETATION PER ORIGINAL PLANS WHEN APPLICABLE (E.G. A VEGETATED SWALE MAY REQUIRE A MINIMUM VEGETATION HEIGHT).
ERIOSION DUE TO CONCENTRATED IRRIGATION FLOW	REPAIR/RE-SEED/RE-PLANT ERODED AREAS AND ADJUST THE IRRIGATION SYSTEM.
EROSION DUE TO CONCENTRATED STORM WATER RUNOFF FLOW	REPAIR/RE-SEED/RE-PLANT ERODED AREAS, AND MAKE APPROPRIATE CORRECTIVE MEASURES SUCH AS ADDING EROSION CONTROL BLANKETS, ADDING STONE AT FLOW ENTRY POINTS, OR MINOR RE-GRADING TO RESTORE PROPER DRAINAGE ACCORDING TO THE ORIGINAL PLAN. IF THE ISSUE IS NOT CORRECTED BY RESTORING THE BMP TO THE ORIGINAL PLAN AND GRADE, THE CITY ENGINEER SHALL BE CONTACTED PRIOR TO ANY ADDITIONAL REPAIRS OR RECONSTRUCTION.
STANDING WATER IN VEGETATED SWALES	MAKE APPROPRIATE CORRECTIVE MEASURES SUCH AS ADJUSTING IRRIGATION SYSTEM, REMOVING OBSTRUCTIONS OF DEBRIS OR INVASIVE VEGETATION, LOOSENING OR REPLACING TOP SOIL TO ALLOW FOR BETTER INFILTRATION, OR MINOR RE-GRADING FOR PROPER DRAINAGE. IF THE ISSUE IS NOT CORRECTED BY RESTORING THE BMP TO THE ORIGINAL PLAN AND GRADE, THE CITY ENGINEER SHALL BE CONTACTED PRIOR TO ANY ADDITIONAL REPAIRS OR RECONSTRUCTION.
STANDING WATER FOR LONGER THAN 96 HOURS FOLLOWING A STORM EVENT	MAKE APPROPRIATE CORRECTIVE MEASURES SUCH AS ADJUSTING IRRIGATION SYSTEM, REMOVING OBSTRUCTIONS OF DEBRIS OR INVASIVE VEGETATION, CLEARING UNDERDRAINS (WHERE APPLICABLE), OR REPAIRING/REPLACING CLOGGED OR COMPACTED SOILS.
OBSTRUCTED INLET OR OUTLET STRUCTURE	CLEAR OBSTRUCTIONS.
DAMAGE TO STRUCTURAL COMPONENTS SUCH AS WEIRS, INLET OR OUTLET STRUCTURES	REPAIR OR REPLACE AS APPLICABLE.
RECCOMENDED EQUIPMENT: DUTCH HOE, PIPE WRENCH, SCREW DRIVER, GARDEN SPADE, SHEARS, AND A RAKE	

LEGEND:

- DMA LIMITS
- DRAINAGE DIRECTION ARROW
- DRAINAGE MANAGEMENT AREA — DMA-#
- PAVEMENT AREA
- ROOFTOP AREA
- BIOFILTRATION AREA
- LANDSCAPE

SOURCE CONTROL BMP NOTES

ALL APPLICABLE SOURCE CONTROL BMPS SHALL BE UTILIZED

A. ALL ONSITE INLETS TO BE MARKED "NO DUMPING" OR SIMILAR AND ALL OPERATIONAL PRECAUTIONS TO AVOID NON STORM WATER DISCHARGE SHALL BE FOLLOWED PER THE CITY'S BMP DESIGN MANUAL.

B. PROPOSED REFUSE AREA WILL REMAIN COVERED AND PROTECTED FROM WIND DISPERSAL. SIGNS SHALL BE PLACED WITH WORDS "DO NOT DUMP HAZARDOUS MATERIALS OR LIQUIDS HERE" OR SIMILAR. OWNER SHALL BE RESPONSIBLE TO KEEP THE AREA CLEAN OF LITTER AND SPILLS.

C. OWNER TO BE RESPONSIBLE FOR SWEEPING PLAZAS, SIDEWALKS, AND PARKING LOTS. THIS IS TO BE DONE REGULARLY AND AS NEEDED TO PREVENT ACCUMULATION OF LITTER AND DEBRIS.

D. FIRE SPRINKLER TEST WATER SHALL BE DRAINED TO THE SANITARY SEWER

GENRAL STORM WATER NOTES

1. HYDROLOGIC SOIL GROUP IS TYPE D, PER SOILS REPORT PRODUCED BY GEOCON INCORPORATED. SEE APPENDIX 6 OF THE SWQMP.
2. GROUNDWATER IS EXPECTED TO BE BETWEEN 60'-55' BELOW EXISTING GRADE ON SITE.
3. NO EXISTING NATURAL HYDROLOGIC FEATURES
4. NO CRITICAL COARSE SEDIMENT YIELD AREAS ON SITE
5. ALL APPLICABLE SOURCE CONTROL BMPS SHALL BE IMPLEMENTED
   5A. SOURCE CONTROL NOTES TO COME IN MINISTERIAL REVIEW

BIOFILTRATION LINER NOTES

BIOFILTRATION LINER TO BE A MINIMUM PER DETAIL B-B, WITH A ROLL WIDTH TO BE NO LESS THAN 12 FT, AND ROLL LENGTH TO BE NO LESS THAN 100 FT TO MINIMIZE THE REQUIRED NUMBER OF SEAMS

CONTRACTOR SHALL VERIFY SUITABILITY OF SELECTED PRODUCT WITH CIVIL ENGINEER PRIOR TO PURCHASE AND THE LINER SHALL CARRY A MINIMUM 20 YEAR WARRANTY.

BIOFILTRATION MEDIA NOTES

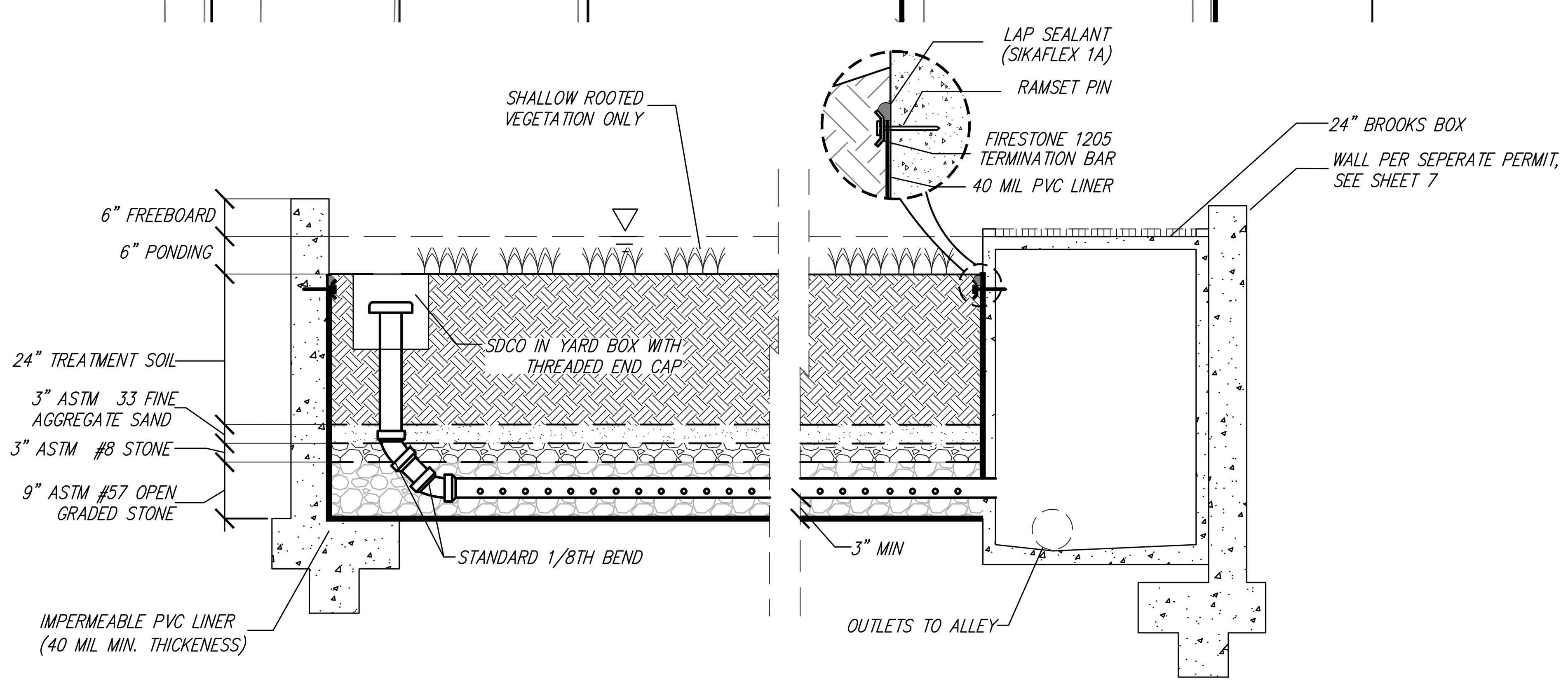
1. IN ACCORDANCE WITH BMP DESIGN MANUAL APPENDIX E ONLY SHALLOW ROOTED VEGETATION SHALL BE PLANTED WITH THE FACILITY MEDIA DEPTH AT 18".
2. ANY VEGETATION THAT IS TO BE PLANTED WITHIN THE BIOFILTRATION FACILITY THAT IS NOT DEEMED TO BE SHALLOW ROOTED, SHALL REQUIRE THE MEDIA DEPTH TO BE INCREASED TO 24" TO CONFORM WITH THE CURRENT BMP DESIGN MANUAL APPENDIX E.
3. ANY TREES PLANTED THAT ARE PROPOSED TO BE PLANTED WITHIN THE BIOFITRATION AREA WILL REQUIRE SPECIAL DETAILING AND A PLANTING MEDIA SECTION THAT IS INCREASED TO A MINIMUM OF 36" DEPTH IN ACCORDANCE WITH BMP DESIGN MANUAL APPENDIX E.

BIOFILTRATION INSPECTION SCHEDULE NOTES

CONTRACTOR MUST CONTACT ENGINEER FOR INSPECTION(*) OF BMPS AT THE FOLLOWING STAGES OF CONSTRUCTION:

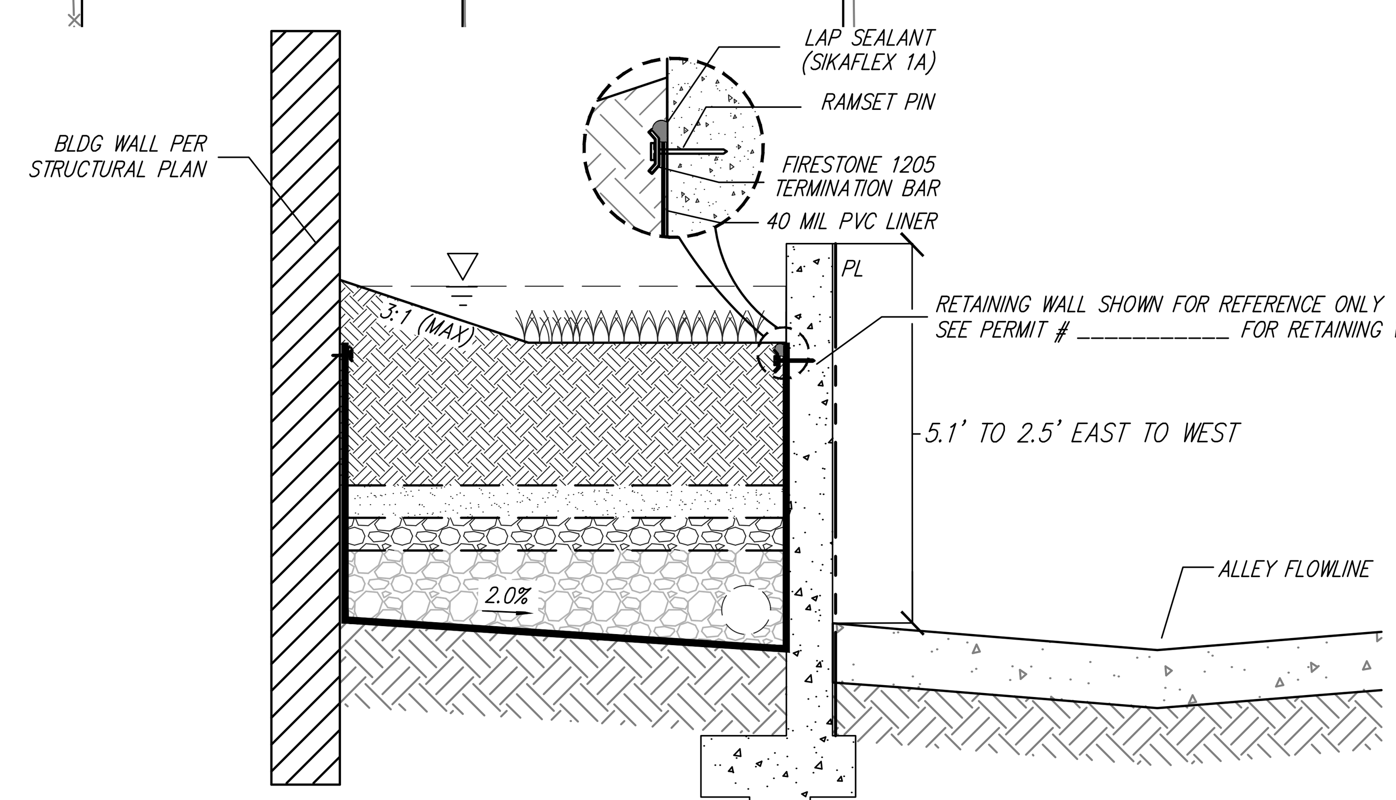
-PRIOR TO START OF CONSTRUCTION OF BIO-FILTRATION AREA
-PRIOR TO CONSTRUCTION OF OUTLET STRUCTURES
-AFTER GRADING OF BASIN AREA
-AFTER PLACEMENT OF IMPERMEABLE LINER
-AFTER PLACEMENT OF SUB-DRAIN
-AFTER THE PLACEMENT OF GRAVEL DRAINAGE LAYER
-AFTER PLACEMENT OF TREATMENT SOIL
-AFTER IRRIGATION AND LANDSCAPING ACTIVITIES

(*) SURVEY STAKES SHALL BE AVAILABLE FOR EACH INSPECTION



CROSS-SECTION 'B'-'B'

(TYP)



CROSS-SECTION 'A'-'A'

(TYP)

GRAPHICAL SCALE: 1" = 20'
0 10 20 40 60

UNDERGROUND SERVICE ALERT

SECTION 4216 & 4217 OF THE GOVERNMENT CODE REQUIRES A DIG ALERT IDENTIFICAITON NUMBER BE ISSUED BEFORE A "PERMIT TO EXCAVATE" WILL BE VALID. FOR YOUR DIG ALERT I.D. NUMBER CALL UNDERGROUND SERVICE ALERT TOLL FREE @ 1-800-422-4133 TWO (2) WORKING DAYS BEFORE YOU DIG. WEB ADDRESS: WWW.DIGALERT.ORG

REGISTERED PROFESSIONAL ENGINEER
ANDREW J. KANN
No. 50940
Exp. 9-30-
CIVIL
STATE OF CALIFORNIA

ANDREW J. KANN R.C.E. 50940 DATE

OMEGA
ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVE. SUITE B
SAN DIEGO, CA 92123
PH:(858) 634-8620 FAX:(858)-634-8627

PRIVATE CONTRACT

BMP PLAN FOR:

U-STORE IT

LOTS 39 TO 48 OF BLOCK 126 OF MAP NO. 379

CITY OF SAN DIEGO, CALIFORNIA
DEVELOPMENT SERVICES DEPARTMENT
SHEET 8 OF 10 SHEETS

PROJECT NO. ______

FOR CITY ENGINEER ______ DATE ______

V.T.M. –

DESCRIPTION	BY	APPROVED	DATE	FILMED
ORIGINAL	OEC			
AS-BUILTS				

NAD83 COORDINATES

194-1726
LAMBERT COORDINATES

CONTRACTOR ______ DATE STARTED ______
INSPECTOR ______ DATE COMPLETED ______

-08 -D

Attachment 5
Drainage Report

Attach project's drainage report. Refer to Drainage Design Manual to determine the reporting requirements.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

U-STOR-IT (Barrio Logan) LLC
Preliminary Drainage Study

2209 National Avenue,
San Diego, CA 92113

Date Prepared:
July 23rd, 2018

Prepared for:
U-STOR-IT Barrio Logan LLC

Prepared By:
Omega Engineering Consultants
4340 Viewridge Ave, Suite B
San Diego, CA 92123
Ph: (858) 634-8620

Declaration of Responsible Charge:
I hereby declare that I am the engineer of work for this project, that I have exercised responsible charge over the design of the project as defined in section 6703 of the business and professions code, and that the design is consistent with current standards. I understand that the check of the project drawings and specifications by the City of San Diego is confined to a review only and does not relieve me, as an engineer of work, of my responsibilities for project design.

Patric T. de Boer RCE 83583
Registration Expires 3-31-2019

Table of Contents

Appendices

Site & Project Description

This Hydrology and Hydraulics report has been prepared as part of the redevelopment plan for the site at 2209 National Avenue into a self-storage facility. The project will remove the existing structure and paving on the site, and construct a self-storage structure. The site discharges to the public storm drain system at two locations to an adjacent alley which carries the storm water to Sampson Street or 26th Avenue. Then storm water flows via curb and gutter to the public storm drain system and then, directly to the San Diego Bay. See Figure 2 for the existing drainage limits. See Figure 3 for the proposed drainage limits.

Methodology

This drainage report has been prepared in accordance with current City of San Diego regulations and procedures, with the exception of the drainage basin weighted C values. These were calculated according to The County of San Diego Hydrology Manual. All of the proposed conduits and conveyances have been designed to intercept and convey the 100-year storm. The Modified Rational Method was used to compute the anticipated runoff. See the attached calculations for particulars. The following references have been used in preparation of this report:

(1) Handbook of Hydraulics, E.F. Brater & H.W. King, 6th Ed., 1976.
(2) Modern Sewer Design, American Iron & Steel Institute, 1st Ed., 1980.
(3) County of San Diego Hydrology Manual, 2003

Culvert Design and Analysis
The storm drain culverts were sized using the K' values from King's Handbook Appendix 7-14, (Appendix 7.0 of this report). The following formula was used:

Q= (K'/n)*d^(8/3)*s^(0.5)
K'= Discharge Factor
d= Diameter of Conduit (ft)
n= Manning's Coefficient
Q= Runoff Discharge (cfs)
s= Pipe Slope (ft/ft)

Rational Method
Q=CIA
Where:
Q= peak discharge, in cubic feet per second (cfs)
C= runoff coefficient, proportion of the rainfall that runs off the surface (no units)
= 0.90*(% impervious)+Cp*(1-% Impervious)) page 5, County Hydrology Manual
I = average rainfall intensity for a duration equal to the Tc for the area, (in/hr)
= $7.44*P6*Tc^{-0.645}$
A = drainage area contributing to the design location, in acres
Cp= Pervious Coefficient Runoff Value, County of San Diego Hydrology Manual minimum of 0.35
$Tc = 1.8*(1.1-C)*(Tc)^{0.5}*S^{0.33}$
Where:
S= Slope of drainage course*

Existing Conditions

The site location consists of an existing bank building located at the northerly corner of the site, and the banks associated parking and driveways. Water is conveyed at 4%-5% slopes via surface run-off to the adjacent alley, and discharges at points 1 and 2 entering the public storm drain via curb and gutter at 26th Street and Sampson Street. The public storm drain discharges at the San Diego Bay.

Proposed Conditions

The project proposes to demolish and remove the existing structure and hardscape, and construct a self-storage facility. The proposed improvements include the storage facility building, and a driveway. Two biofiltration basins will be constructed alongside the south westerly frontage of the site. The biofiltration basins will drain to the adjacent alley, at discharge points 1 and 2, before entering the public storm drain via a curb inlet at 26th street and Sampson Street. See the Storm Water Quality Management Plan (SWQMP) for details.

Existing Runoff Analysis

The existing site was modeled as two sub-basins, EX-1 & EX-2. Basin EX-1 contains the majority of the parking lot on the site, all of the site landscaping, and discharge point 1 located at the southeasterly corner of the site. EX-2 contains the entire bank building, some impervious surfaces, and discharge point 2 which is located at the southwesterly corner of the site. See Figure 2 for more information. As the existing surface conditions varied for each sub-basin, run-off coefficients were found using a weighted average with soils having a run-off coefficient of 0.35, and drive pavement/roofs having a run-off coefficient of 0.9. Runoff flow rates were determined using the rational method, which is summarized in the Methodology section of this report.

Below is a summary of the basin input data and resulting Q's:

Basin #	Area (ac)	C	Slope (%)	Q_{100} (cfs)
EX-1	0.56	0.79	4.0	2.82
EX-2	0.24	0.90	5.0	1.38

See the attached calculations for details.

Proposed Runoff Analysis

The proposed site was modeled as two sub-basins, A-1 and A-2. Basin A-1contains the majority of the building, a biofiltration basin, and discharge poin1 located at the southeasterly corner of the site. Basin A-2 contains the remaining portion of the building, a separate biofiltration basin, and another biofiltration basin located at the southwesterly corner of the site. See Figure 3 for details. As the proposed surface conditions varied for each sub-basin, run-off coefficients were found using a weighted average with soils having a run-off coefficient of 0.35, and drive pavement/roofs having a run-off coefficient of 0.9. Runoff flow rates were determined using the rational method, which is summarized in the Methodology section of this report.

Below is a summary of the basin input data and resulting Q's:

Basin #	Area (ac)	C	Slope (%)	Q_{100} (cfs)
A-1	0.56	0.88	1.5%	2.80
A-2	0.24	0.88	1.5%	1.35

See the attached calculations for details.

Results and Conclusions

The redevelopment of the site shall result in a decrease of 0.02 CFS for the 100 year storm event for discharge point 1, and a decrease of 0.03 CFS for discharge point 2.

It is the opinion of Omega Engineering Consultants that the project will not cause adverse effects to the downstream facilities or receiving waters. A separate Storm Water Quality Management Plan (SWQMP) has been prepared to discuss the water quality impacts for the proposed development.

2209 National Avenue San Dieg CA, 92113 10/10/2017

HYDROLOGY AND HYDRAULICS CALCS (Table No. 1)

BASIN	AREA (SF)	AREA (AC)	% Imp	"C" Value
EX-1	24,581	0.56	80%	0.79
EX-2	10,550	0.24	100%	0.90
PROP TOTAL	**35,131**	**0.81**		
A-1*	24,581	0.56	97%	0.88
A-2	10,550	0.24	97%	0.88
PROP TOTAL	**35,131**	**0.81**		

Basin Confluence	Symbol
-	-
-	-
-	-
-	-

(A) "CP#1" Confluence Point Number 1

(B) C value for bare ground is 0.35 (Table 3-1 County Hydrology Manual)
C value for impervious surfaces is 0.9
Basins with mixed surface type use a weighted average
of these 2 values. (impervious % x 0.9)+(pervious % x 0.35)

2209 National Avenue San Dieg CA, 92113
HYDROLOGY AND HYDRAULICS CALCS (Table No. 2)

10/10/2017

Sub-Basin	AREA Ac.	"C"	CA	L (ft) Travel	H (ft) (elev)	S(%) (avg.)	Tc min.	T tot mins	I in/hr	Q cfs	Q tot cfs	NOTES 85th % storm
EX-1	0.56	0.79	0.45	141.00	5.60	4.0	5.00	5.00	0.20	0.09	0.09	
								5.00	**0.20**		**0.09**	
						Existing Discharge Pt. 1 =					**0.09**	CFS
EX-2	0.24	0.90	0.22	141.00	7.00	5.0	5.00	5.00	0.20	0.04	0.04	
								5.00	**0.20**		**0.04**	
						Existing Discharge Pt. 2 =					**0.04**	CFS
A-1	0.56	0.88	0.50	312.00	4.68	1.50	6.01	6.01	0.20	0.10	0.10	
								6.01	**0.20**		**0.10**	
						Prposed Discharge Pt. 1 =					**0.10**	CFS
A-2	0.24	0.88	0.21	212.00	3.18	1.50	5.00	5.00	0.20	0.04	0.04	
								5.00	**0.20**		**0.04**	
						Prposed Discharge Pt. 2 =					**0.04**	CFS

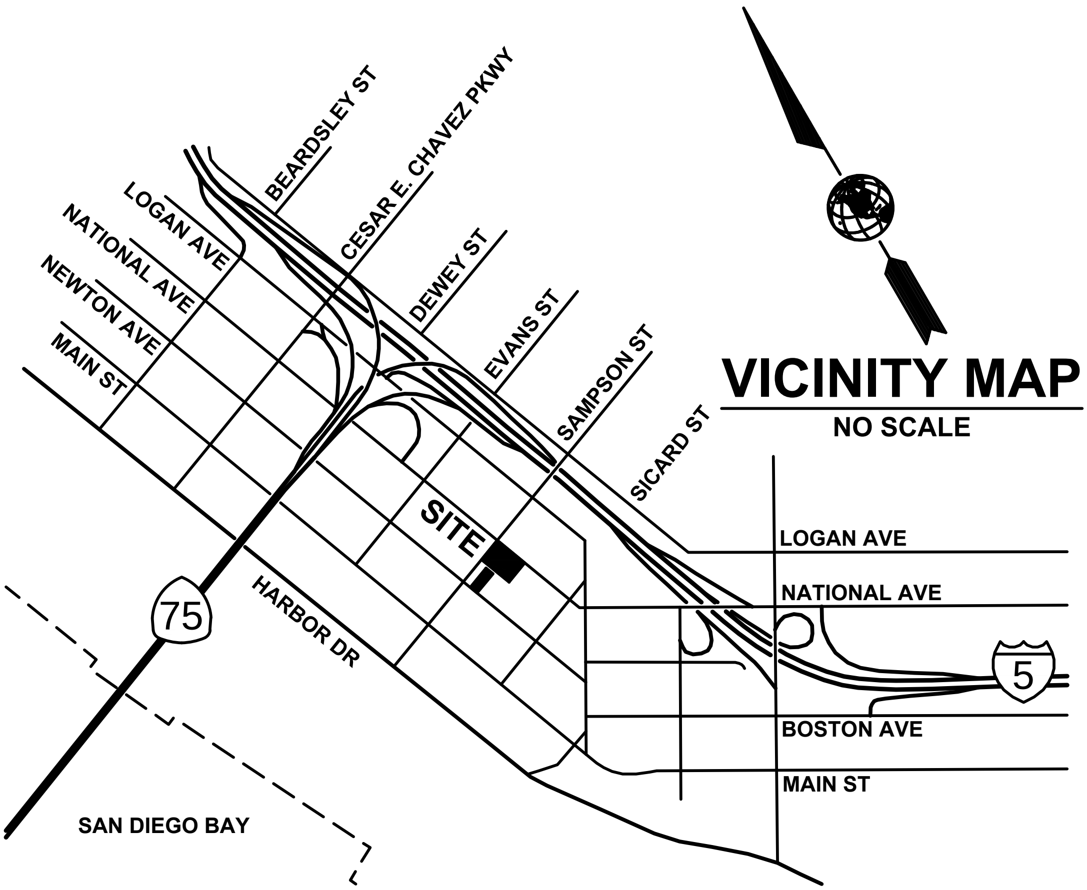
2209 National Avenue San Dieg CA, 92113

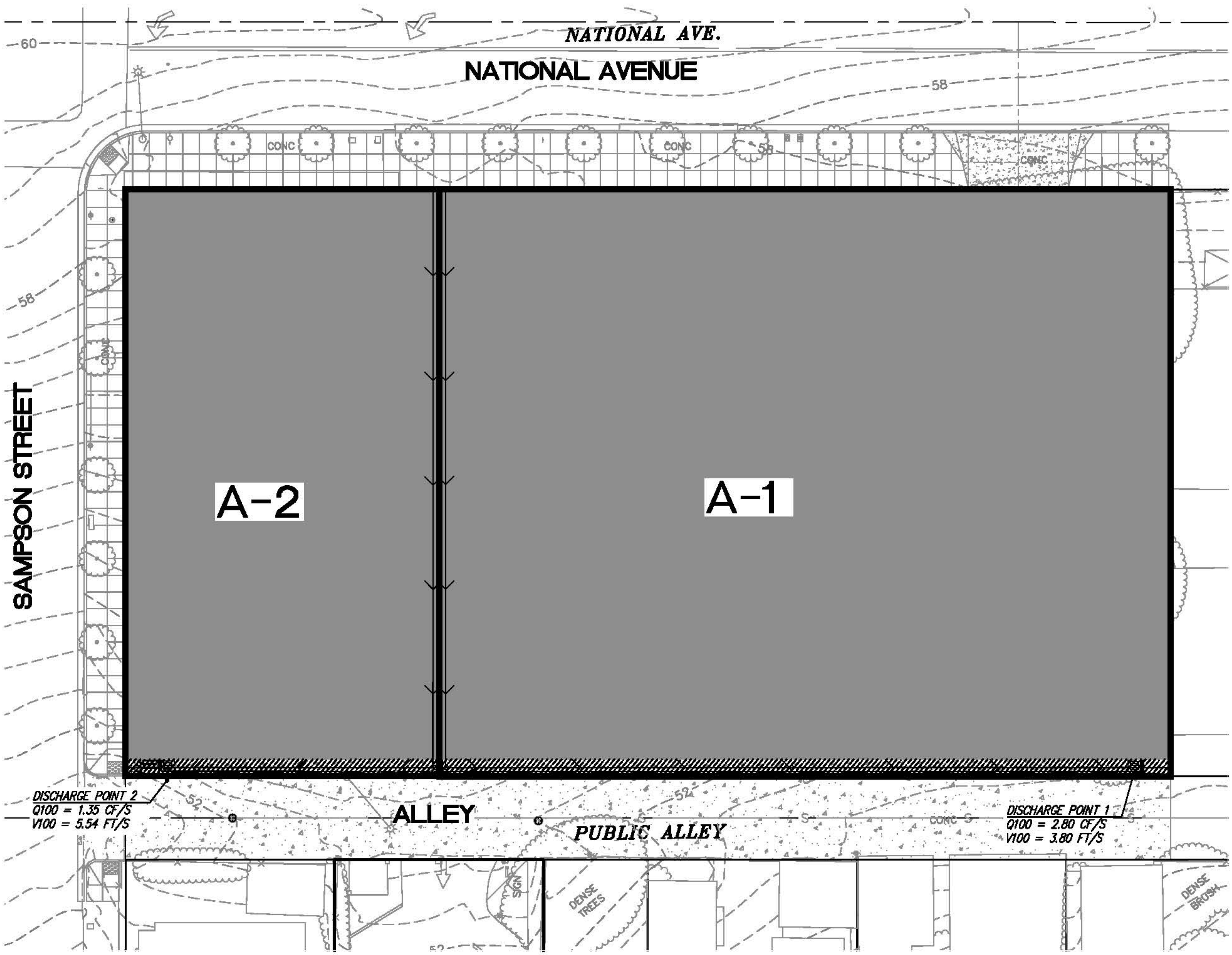
HYDROLOGY AND HYDRAULICS CALCS (Table No. 3)

10/10/2017

Sub-Basin	AREA Ac.	"C"	CA	L (ft) Travel	H (ft) (elev)	S(%) (avg.)	Tc min.	T tot mins	I in/hr	Q cfs	Q tot cfs		NOTES 100 year storm
													P(6)= 2.40
EX-1	0.56	0.79	0.45	141.00	5.60	4.0	5.00	5.00	6.32	2.82	2.82		
											2.82		
							Existing Discharge Pt. 1 =				**2.82**	**CFS**	
EX-2	0.24	0.90	0.22	141.00	7.00	5.0	5.00	5.00	6.32	1.38	1.38		
											1.38		
							Existing Discharge Pt. 2 =				**1.38**	**CFS**	
A-1	0.56	0.88	0.50	312.00	4.68	1.50	6.01	6.01	5.61	2.80	2.80		
											2.80		
							Proposed Discharge Pt. 1=				**2.80**	**CFS**	
A-2	0.24	0.88	0.21	212.00	3.18	1.50	5.00	5.00	6.32	1.35	1.35		
											1.35		
							Proposed Discharge Pt. 2=				**1.35**	**CFS**	

FIGURE 1


VICINITY MAP
NO SCALE
BEARDSLEY ST
CESAR E. CHAVEZ PKWY
DEWEY ST
EVANS ST
SAMPSON ST
SICARD ST
LOGAN AVE
NATIONAL AVE
NEWTON AVE
MAIN ST
SITE
HARBOR DR
75
SAN DIEGO BAY
LOGAN AVE
NATIONAL AVE
5
BOSTON AVE
MAIN ST



LEGEND:

AREA LIMITS

FLOW PATH ARROW INDICATES DIRECTION

BASIN NUMBER — EX-#

BUILDING AREA

BIOFILTRATION AREA

BASIN DATA					
BASIN	AREA	IMPERVIOUS	IMPERVIOUS %	Q_{100}(CFS)	V_{100}(FT/S)
A-1	24,581 SF	23,881 SF	97%	2.80	3.50
A-2	10,550 SF	10,250 SF	97%	1.35	3.80

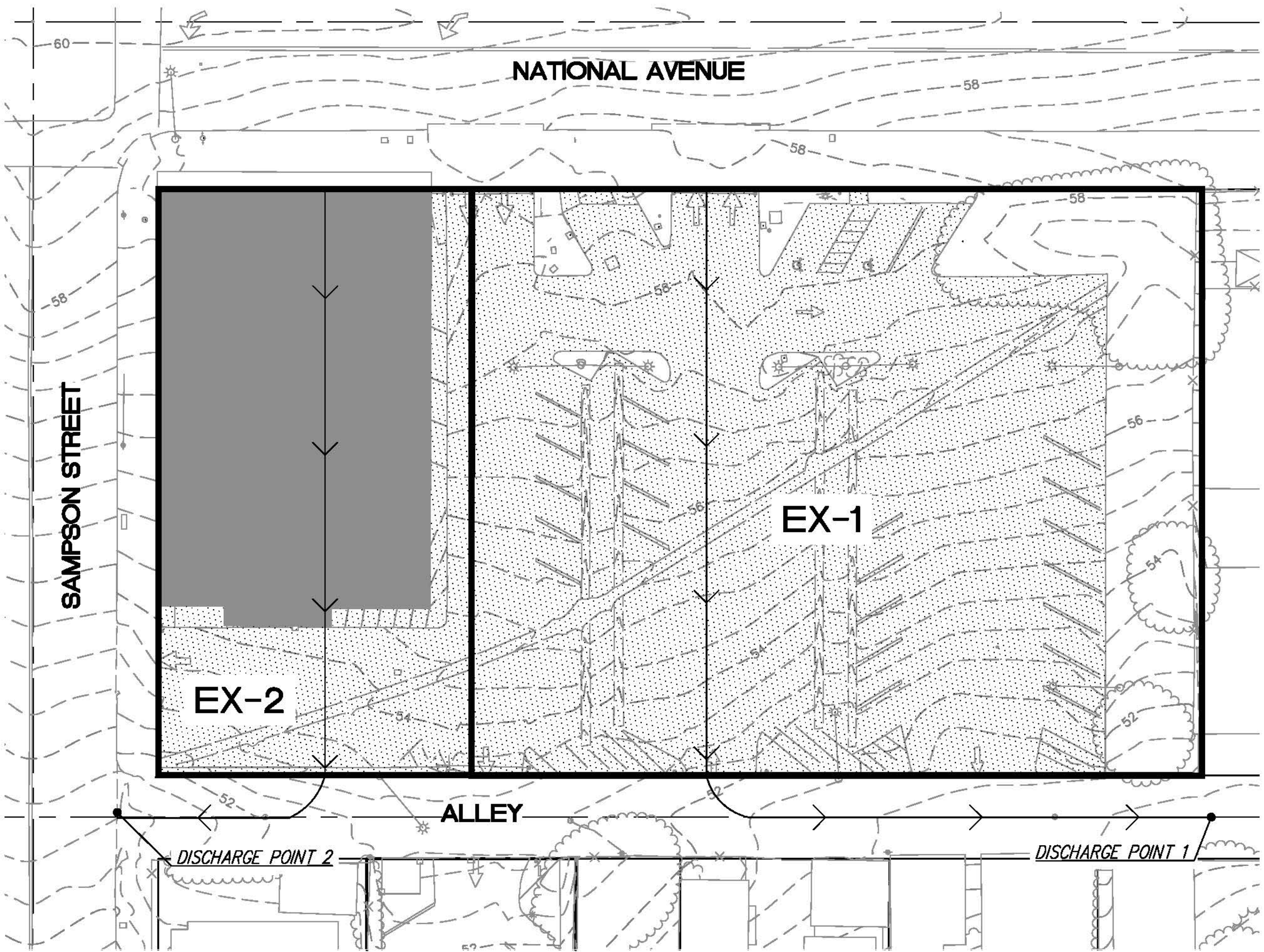
GRAPHICAL SCALE: 1" = 30'

0 15 30 60

FIGURE 3
NATIONAL AVE SELF STORAGE
PROPOSED HYDROLOGY

OMEGA

OMEGA ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA 92123
PH:(858) 634-8620 FAX:(858) 634-8627



LEGEND:

AREA LIMITS

FLOW PATH ARROW INDICATES DIRECTION

BASIN NUMBER — EX-#

BUILDING AREA

PAVEMENT AREA

LANDSCAPE AREA

BASIN DATA					
BASIN	AREA	IMPERVIOUS	IMPERVIOUS %	Q_{100}(CFS)	V_{100}(FT/S)
EX-1	24,581 SF	19,770 SF	80 %	2.82	3.52
EX-2	10,550 SF	10,550 SF	100 %	1.38	3.82

GRAPHICAL SCALE: 1" = 30'

0 15 30 60

FIGURE 2
NATIONAL AVE SELF STORAGE
EXISTING HYDROLOGY

OMEGA

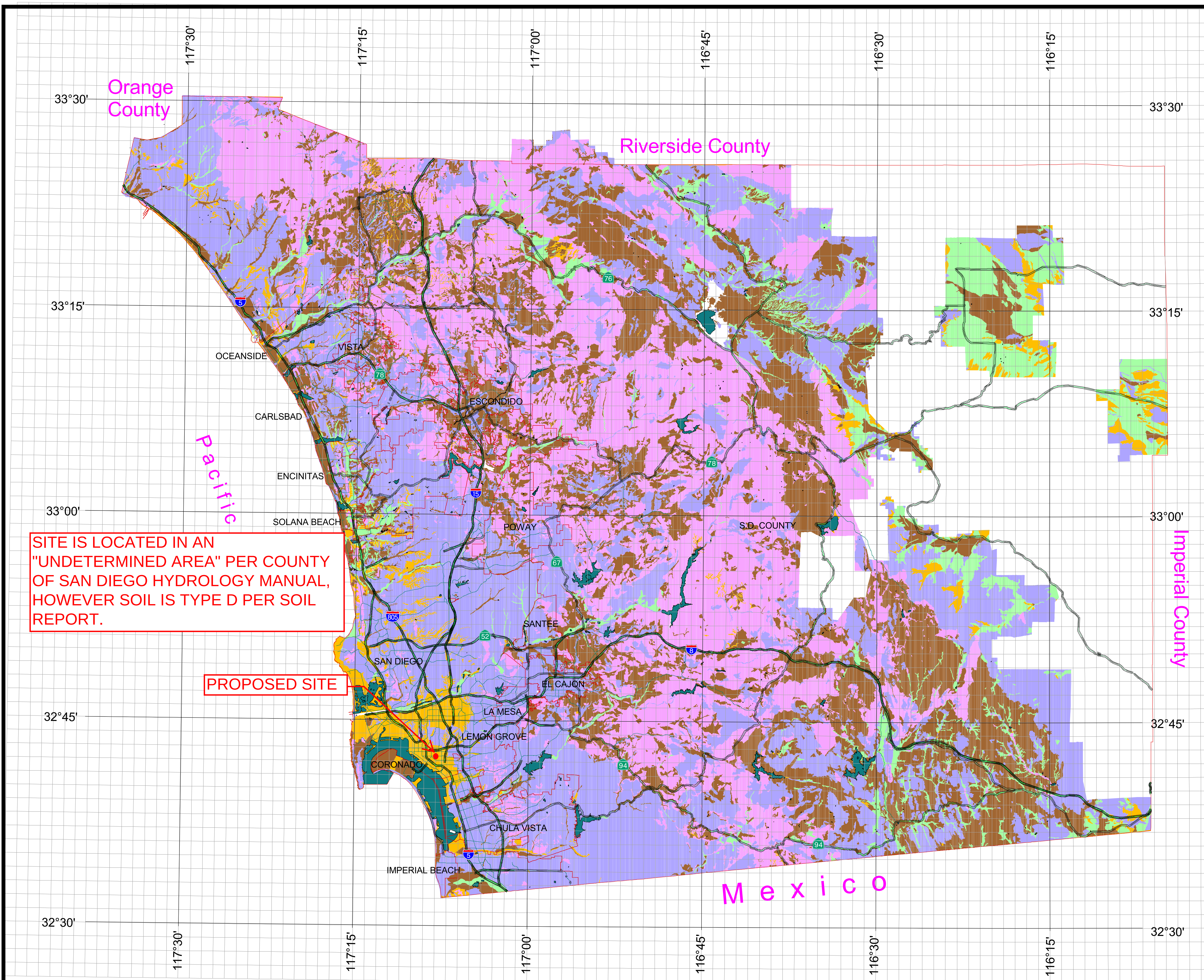
OMEGA ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA 92123
PH:(858) 634-8620 FAX:(858) 634-8627

Appendices

Appendix 1

County of San Diego Hydrology Manual

Soil Hydrologic Groups



Legend

Soil Groups

- Group A
- Group B
- Group C
- Group D
- Undetermined
- Data Unavailable

DPW GIS

Department of Public Works
Geographic Information Services

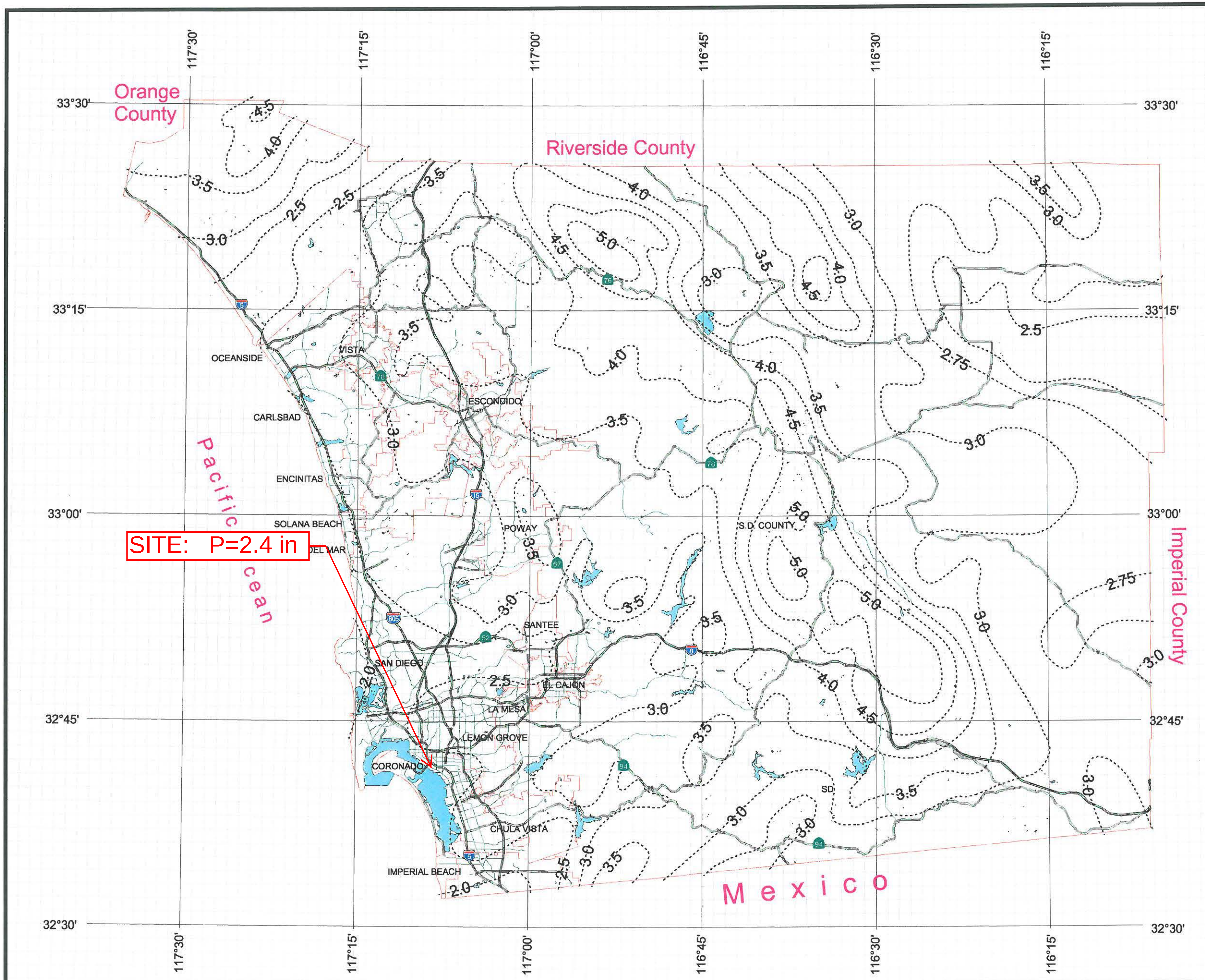
SanGIS

We Have San Diego Covered!

N W E S

3 0 3 Miles

Appendix 2



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours

Isopluvial (inches)

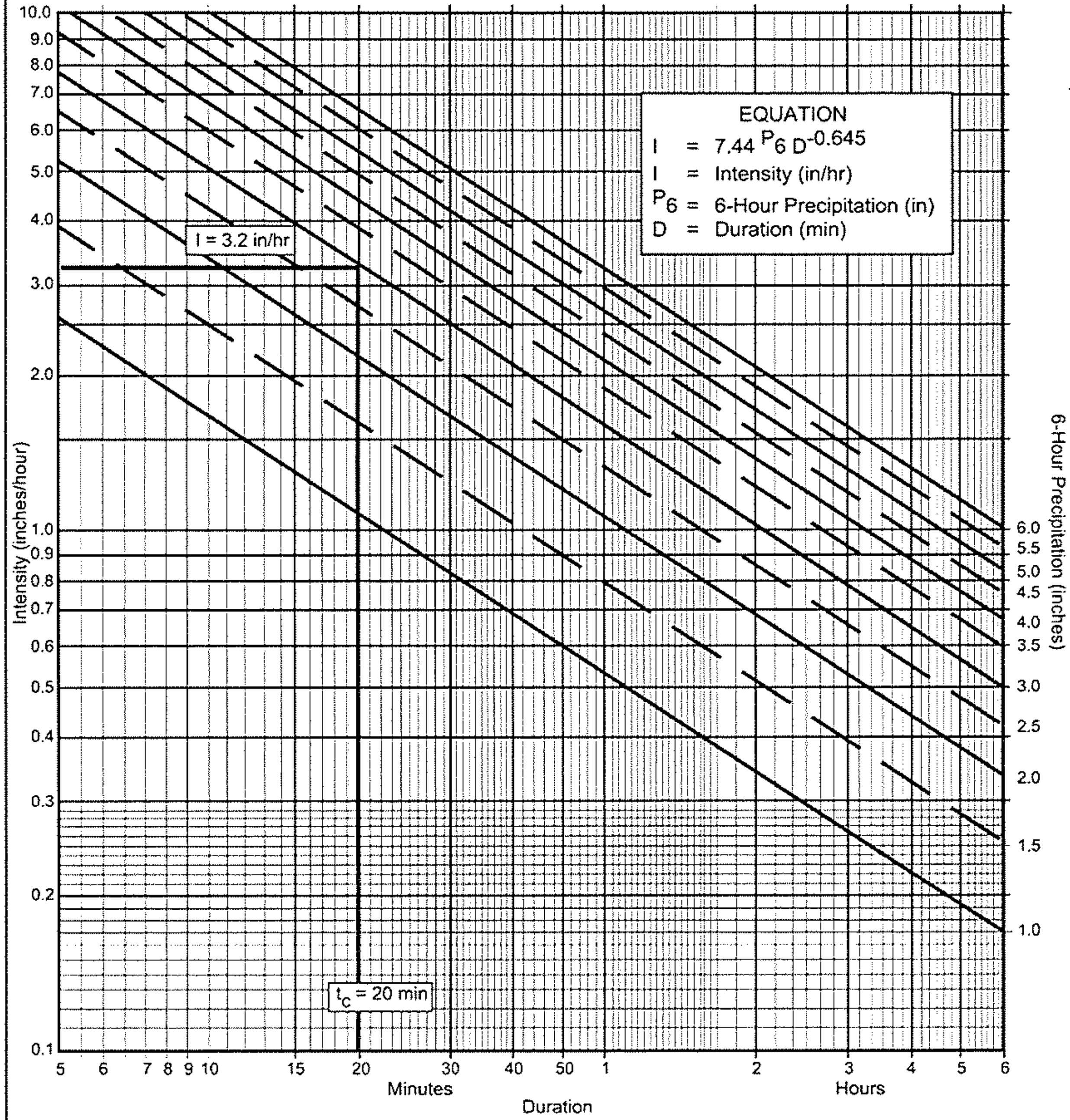
DPW GIS
Department of Public Works Geographic Information Services

SanGIS
We Have San Diego Covered!

N
W E
S

3 0 3 Miles

Appendix 3



Directions for Application:

(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicape to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application For

(a) Selected frequency ______ year

(b) P_6 = ______ in., P_{24} = ______, $\frac{P_6}{P_{24}}$ = ______ %(2)

(c) Adjusted $P_6^{(2)}$ = ______ in.

(d) t_x = ______ min.

(e) I = ______ in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

P6 / Duration	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

Intensity-Duration Design Chart - Example

Appendix 4

Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS

Land Use		Runoff Coefficient "C"				
			Soil Type			
NRCS Elements	County Elements	% IMPER.	A	B	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.30	0.35
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10	0.27	0.32	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.80	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).

DU/A = dwelling units per acre

NRCS = National Resources Conservation Service

Appendix 5

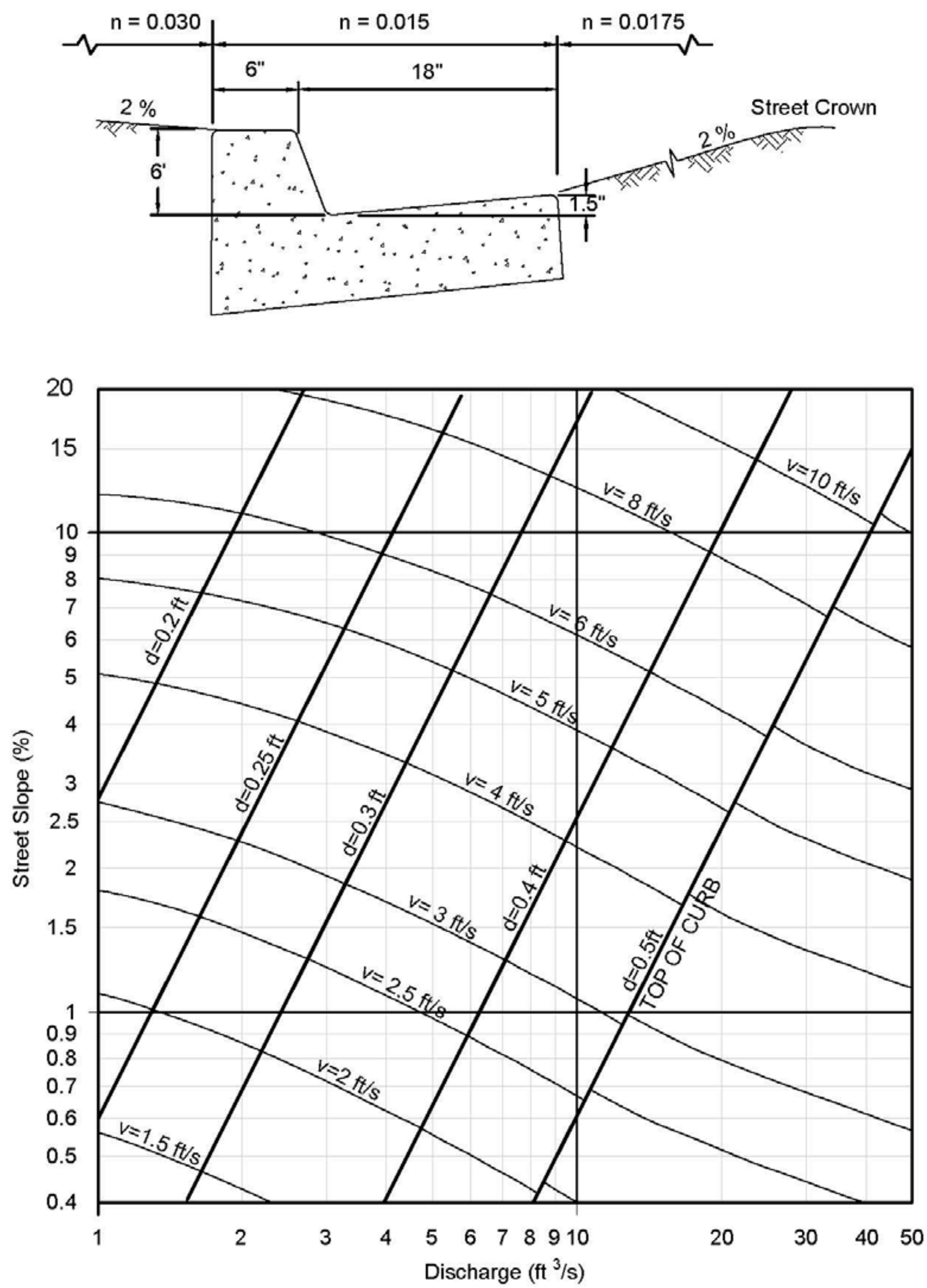


Figure 3-2: Gutter and Roadway Discharge-Velocity Chart (6" Curb)

Attachment 6
Geotechnical and Groundwater Investigation Report

Attach project's geotechnical and groundwater investigation report. Refer to Appendix C.4 to determine the reporting requirements.

THIS PAGE INTENTIONALLY LEFT BLANK FOR DOUBLE-SIDED PRINTING

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL AND FAULT INVESTIGATION

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

PREPARED FOR

U STOR IT (BARRIO LOGAN)
SAN DIEGO, CALIFORNIA

DECEMBER 5, 2017
PROJECT NO. G2093-52-01

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G2093-52-01
December 5, 2017

U STOR IT (Barrio Logan)
402 West Broadway, Suite 810
San Diego, California 92101

Attention: Mr. Lawrence Nora

Subject: GEOTECHNICAL AND FAULT INVESTIGATION
2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

Dear Mr. Nora:

In accordance with your request and our Proposal No. LG-17040, dated February 3, 2017, we herein submit the results of our geotechnical and fault rupture hazard investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards to assist in the design of the proposed building and improvements. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The site is considered suitable for the proposed building and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project.

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

Very truly yours,

GEOCON INCORPORATED

Matthew R. Love
RCE 84154

Shawn Foy Weedon
GE 2714

Rupert S. Adams
CEG 2561

MRL:SFW:RSA:ejc

(e-mail) Addressee

REGISTERED PROFESSIONAL ENGINEER
MATTHEW R. LOVE
C84154
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
SHAWN WEEDON
No. 2714
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
RUPERT S. ADAMS
NO. 2561
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

TABLE OF CONTENTS

TABLE OF CONTENTS (Concluded)

GEOTECHNICAL AND FAULT INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical and fault investigation for the proposed new self-storage facility in the Barrio Logan area of San Diego, California as shown on the Vicinity Map, Figure 1. The purpose of this geotechnical and fault investigation is to evaluate the surface and subsurface soil conditions, general site geology, and to identify geotechnical constraints that may impact the planned improvements to the property. In addition, this report provides 2016 CBC seismic design criteria; grading recommendations; shoring and tie-back recommendations; shallow foundation and concrete slab-on-grade recommendations; mat foundation recommendations; retaining wall and lateral load recommendations; and discussions regarding the local geologic hazards including faulting and seismic shaking.

This report is limited to the area proposed for the construction of the new development and associated improvements as shown on the Geologic Map, Figure 2. We used the Conceptual Grading Plan prepared by Omega Engineering (2017) as the base for the Geologic Map. Figure 3 present a geologic cross-section for the conditions encountered during our field investigation.

The scope of this investigation included reviewing readily available published and unpublished geologic literature, including available fault investigation reports for nearby sites (see List of References); performing engineering analyses; and preparing this geotechnical investigation report. We also drilled six geotechnical borings to a maximum depth of 50 feet (see Appendix A), excavated a fault trench across the site to a maximum depth of 9 feet (see Figure 4), performed four infiltration tests, sampled soil and performed laboratory testing. Appendix A presents the exploratory boring and trench logs. The results of the laboratory tests are presented in Appendix B and on the boring logs in Appendix A. Appendix C presents the results of the storm water management investigation.

Our geotechnical Borings B-3 and B-4 and associated infiltration tests P-1 and P-2 are located in the existing parking lot to the south of the property. This area was previously planned for additional site parking and potential stormwater management by the design team; however, this area is now not a part of the project. We have included the boring logs and infiltration test results from these borings in the report for informational purposes only. The locations of the offsite borings are shown on the Geologic Map, Figure 2.

2. SITE AND PROJECT DESCRIPTION

The property is located south of National Avenue and east of Sampson Street in the Barrio Logan area of San Diego, California. The rectangular property consists of a vacant commercial bank structure on the northwest corner of the property and the remainder consists of surface asphalt concrete parking. The northern bank property is relatively flat at an elevation of about 50 to 60 feet above Mean Sea Level (MSL).

We understand the planned development consists of a 3-story self-storage facility over 2 subterranean levels on the northern portion of the property. We expect the proposed structure would likely be supported on conventional shallow foundation systems founded in Old Paralic Deposits at a proposed pad elevation of 38 feet MSL. We understand that bio-filtration devices will be constructed on the southern portion of the property and will be lined to prevent infiltration into subgrade materials.

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

3. GEOLOGIC SETTING

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

The site is located on the western portion of the coastal plain. Marine sedimentary units make up the geologic sequence encountered on the site and consist of Pleistocene age Old Paralic Deposits Unit 6 (Qop_6; formerly known as the Bay Point Formation) underlain by Pliocene age San Diego Formation sediments. Old Paralic Deposits mapped as Unit 6 were deposited roughly 120k years ago and are synonymous with the Nestor Terrace. The Old Paralic Deposits represent deposition in a brackish water estuarine and near shore terrestrial environment (Kennedy, 1999), and consist of fine to coarse grained sand with varying amounts of silts, clays and gravel. The San Diego Formation located below the Old Paralic Deposits is in excess of 100 feet thick, but was not encountered during our investigation. The geologic conditions in the vicinity of the site are shown on the Regional Geologic Map, Figure 5.

The regional geology in the area is predominately controlled by the active Rose Canyon Fault Zone (RCFZ) which transitions from a strike slip fault to the north of the site to several faults that have oblique movements of both strike slip and normal faulting to the west and east. The San Diego Bay was created as a down dropped block within this fault zone. The zone extends to the south and branches into three segments, Spanish Bight, Coronado, and Silver Strand Faults. There are two active fault zones in downtown area of San Diego that have been included in state-designated Alquist-Priolo Earthquake Fault Zones: 1) near First Street and in the vicinity of 15th and 16th Streets and 2) the Downtown Graben (California Geological Survey, 2003). The graben appears to widen to the south towards San Diego Bay. The active fault mapped just east of 16th Street is possibly associated with the eastern limits of the graben. The western limit is roughly mapped along 12th Street. The Regional Fault Map, Figure 6, shows the faults in the downtown San Diego area.

4. SOIL AND GEOLOGIC CONDITIONS

Our field investigation indicates the site is underlain by one surficial soil type consisting of undocumented fill and one geologic units consisting of the Pleistocene age Old Paralic Deposits (map symbol Qop6). The boring logs (Appendix A) and Geologic Map (Figure 2) show the occurrence, distribution, and description of each unit encountered during our field investigation. The Geologic Cross-Section and Trench Log (Figures 3 and 4, respectively), presents a profile view of the underlying geologic conditions. The surficial soil and geologic units are described herein in order of increasing age.

4.1 Undocumented Fill (Qudf)

We encountered isolated pockets of undocumented fill associated with the previous site improvements within our geotechnical borings and fault trench. The fill thickness generally ranges from 6 inches to 3 feet, where encountered. The fill generally consists of medium dense and stiff, reddish brown to brown, clayey sand and clay with some gravel and deleterious materials. The existing fill is considered unsuitable for support of the proposed building structure. We expect the fill materials will be removed within the planned building areas during excavations to achieve finish grade elevations for the subterranean levels. Existing fill exposed at subgrade elevation for proposed adjacent street improvements should be processed, moisture conditioned as necessary and properly compacted. The existing fill material can be reused as properly compacted fill if relatively free from vegetation, debris, and contaminants.

4.2 Old Paralic Deposits (Qop_6)

Quaternary-age Old Paralic Deposits Unit 6 (formerly called the Bay Point Formation) underlies the existing fill soil. The upper 25 feet of the Old Paralic Deposits consists of a moderately cemented, medium dense to very dense, yellowish brown to reddish brown, silty and clayey sand with some gravel. The Old Paralic Deposit materials underlying the upper materials consists of an olive gray to gray brown, stiff to very stiff, sandy silt and clay. These materials were encountered to the maximum depth explored of 51½ feet. The Old Paralic Deposits possess a “very low” to “low” expansion potential (expansion index of 50 or less). Old Paralic Deposits are considered suitable for direct support of structural loads.

5. GROUNDWATER

We did not encounter groundwater in our geotechnical borings to the maximum depth explored of 51½ feet or an elevation of roughly 10 feet above MSL. It is typical to see groundwater from 0 to 5 feet above MSL in the downtown area. Based on a proposed finish floor elevation of about 38½ feet MSL, we do not expect groundwater to be encountered during construction of the proposed development. It is possible that perched seepage layers may be encountered during excavation and drilling operations due to adjacent irrigation and drainage practices. It is not uncommon for perched groundwater conditions to develop where none previously existed. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheets 13 and 17 defines the site with a *Hazard Category 13: Downtown Special Fault Zone*. Based on a review of the map (see Figure 7 - Downtown Special Fault Zone Map), a fault does not traverse the planned development area.

6.2 Faulting

By definition of California Geological Survey (CGS), an active fault is a fault that has had surface displacement in Holocene time (approximately 11,000 years). Potentially active faults are defined as faults with activities during the Pleistocene age (between 1,600,000 and 11,000 years ago). According to these definitions, Special Studies Zones mandated by the State of California (Alquist-Priolo) Geologic Hazards Zones Act was adopted. The purpose of this act is to assure that structures with human occupancy are not constructed across traces of active faults.

The site is located immediately south of the Rose Canyon Fault Zone in an area that is transitional between the predominately right-lateral slip faulting characteristic of the faults north of the downtown area and the predominately dip-slip faulting characteristic of faults making up the southern portion of the Rose Canyon Fault Zone (Treiman, 1993). South of the downtown area, the major faults that compose the southern end of the Rose Canyon Fault Zone are the Spanish Bight, Coronado, and Silver Strand Faults. The east side of this zone is represented by the La Nación Fault

(Treiman, 1993). Together, these faults define a wide and complexly faulted basin occupied by San Diego Bay and a narrow section of the continental shelf west of the Silver Strand.

Trenching by Lindvall and others (1990) on the Rose Canyon Fault in Rose Canyon several miles north of the site, by Owen Consultants (referenced by ICG, 1990) for the police station on a site southeast of the subject property, and by Kleinfelder Incorporated at a site near First Avenue and Market Street in the downtown area, have shown that Holocene soil (soil 11,000 years old or less) has been displaced by faulting within the Rose Canyon Fault Zone.

The California Geological Survey has issued a revised Alquist-Priolo Earthquake Fault Zone Map for the Point Loma Quadrangle (CGS, 2003) that includes portions of the downtown San Diego area. Fault splays associated with the Downtown Graben and the San Diego Fault are considered active by the State of California (Treiman, 2002, 2003) and Alquist-Priolo Earthquake Fault Zones have been established for these faults as shown on Figure 6 - Regional Fault Map.

A review of geologic literature and experience with the soil and geologic conditions in the general area, indicate that known active, potentially active, or inactive faults are not located at the site. The site is, however, located in close proximity to known faults. The property is not located within a State of California Earthquake Fault Zone; however, the site is located approximately 3,000 feet from the eastern active fault trace designated in downtown San Diego. The property is also located within the City of San Diego Special Studies Fault Zone (see Figure 7).

We reviewed several fault investigation reports for sites within the immediate areas. Based on our review of these documents, there is no indication of active faulting or off-fault deformation in the immediate site vicinity. We discuss the specific reports reviewed and the results in subsequent sections of this report.

6.3 Surface Fault Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the earth surface. We performed a site-specific fault rupture hazard investigation at the site that included excavation of an exploratory trench along an east-west trending transect across the site to evaluate the potential for faulting. The trench and exploration transect were oriented to specifically evaluate faults that trend N16W to N16E and 30 degrees from this anticipated trend. The results of our fault rupture hazard evaluation indicate the potential for surface fault rupture at the site is negligible due to the absence of active faults at the subject site. The details of our site-specific fault rupture hazard investigation are presented in Section 7 of this report.

6.4 Seismicity

The historic seismicity or instrumental seismic record in the San Diego area indicates that there have been numerous minor earthquakes in the San Diego Bay area, including events in 1964 and 1985 between M3 and 4+ (Treiman, 1993). Surface rupture has not been recorded with any of the seismic activity. Anderson and others (1989) indicate that the greatest peak acceleration recorded in the downtown area (at San Diego Light and Power) was 34 cm/sec^2 (0.03g) produced by an offshore earthquake in 1964 (M 5.6).

Anderson and others (1989) have also estimated recurrence times for major earthquakes that may affect the San Diego Region. By combining geologic data with their model for ground motion attenuation for each earthquake event, they have estimated the recurrence rate of various levels of peak ground acceleration in the San Diego area. The results of their work indicate that peak accelerations of 10 to 20 percent gravity (g) are expected approximately once every 100 years (Anderson and others, 1989). Higher peak accelerations will also occur but with a lower probability of occurrence or higher return period.

Lindvall and others (1991) have postulated a maximum likely slip rate of about 2 mm per year and a best estimate of about 1.5 mm per year, based on recent three-dimensional trenching on the Rose Canyon Fault in Rose Canyon several miles north of the site. They found stratigraphic evidence of at least three events during the past 8,100 years. The most recent surface rupture displaces the modern “A” horizon (topsoil), suggesting that this event probably occurred within the past 500 years.

Historically, the Rose Canyon Fault has exhibited low seismicity with respect to earthquakes in excess of magnitude 5.0 or greater. Earthquakes on the Rose Canyon Fault having a maximum magnitude of 6.5 are considered representative of the potential for seismic ground shaking within the property. The “maximum magnitude earthquake” is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework.

According to the computer program *EZ-FRISK* (Version 7.65), 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 Faults, located approximately 1.3 miles west of the site and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood/Rose Canyon Faults 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 Faults are 7.5 and 0.54g, respectively. Table 6.4.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration

(PGA) using Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008 acceleration-attenuation relationships.

TABLE 6.4.1
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	1	7.50	0.46	0.40	0.54
Rose Canyon	1	6.90	0.43	0.40	0.50
Coronado Bank	13	7.40	0.24	0.18	0.23
Palos Verdes Connected	13	7.70	0.26	0.19	0.26
Elsinore	42	7.85	0.14	0.09	0.11
Earthquake Valley	46	6.80	0.08	0.06	0.05

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2007) NGA USGS 2008 in the analysis. Table 6.4.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.4.2
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	0.57	0.50	0.61
5% in a 50 Year Period	0.39	0.34	0.41
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 California Building Code (CBC) guidelines currently adopted by the City of San Diego.

6.5 Seiches and Tsunamis

Seiches are free or standing-wave oscillations of an enclosed water body that continue, pendulum fashion, after the original driving forces have dissipated. Seiches usually propagate in the direction of longest axis of the basin. The site located approximately 2,000 feet from San Diego Bay and is at an elevation of approximately 50 to 60 feet above Mean Sea Level (MSL); therefore, the potential of seiches impacting the site is considered to be 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 may include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first-order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). The largest tsunami recorded in San Diego since 1950 occurred on May 22, 1960, which had maximum run-up amplitudes of 2.1 feet (0.7 meters) [URS, 2004]. Wave heights and run-up elevations from tsunamis along the San Diego Coast have historically fallen within the normal range of the tides. Our review of the map titled *Tsunami Inundation Map for Emergency Planning, State of California, County of San Diego, Point Loma Quadrangle, June 1, 2009*, by CEMA, CGS, and USC, shows that the site is not located within the mapped tsunami hazard zone.

6.6 Liquefaction

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

6.7 Hydroconsolidation

Hydroconsolidation is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the effected soil and overlying foundations or improvements supported thereon. Dry to damp (with a degree of saturation less than about 70 percent), loose to dense sand are typically prone to hydroconsolidation. Potentially compressible soil underlying the proposed structures and existing fill is typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydroconsolidation of the soil exists. The potential for hydroconsolidation can be mitigated by remedial grading and the use of stiffer foundation systems. Based on the results of the laboratory testing, hydroconsolidation potential ranges from about 0.1 to 3.5 percent within the Old Paralic Deposits. We expect the upper 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation within the upper portion of the Old Paralic Deposits ranges up to about 4¼ inches.

6.8 Landslides

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

7. SITE-SPECIFIC FAULT RUPTURE HAZARD INVESTIGATION

7.1 Purpose and Scope

No splays of the Rose Canyon Fault Zone were mapped at the site and the site does not fall within a State of California Alquist-Priolo Earthquake Fault Zone. However, the site is located within a City of San Diego Downtown Special Fault Zone and a site-specific fault rupture hazard investigation is required to evaluate the potential for surface fault rupture at the site.

The purpose of our investigation is to evaluate the presence or absence of faults bisecting the site that may impact the proposed development and to assess the age and continuity of on-site stratigraphy. Our investigation conforms to CGS *Guidelines for Evaluating the Hazard of Surface Fault Rupture* (CGS Note 49), Appendix D of the *City of San Diego Guidelines for Geotechnical Reports* (2011), and current geologic standards-of-practice for the evaluation of potential surface fault rupture.

7.2 Literature Review

We reviewed the following fault and/or geotechnical investigations within the immediate area of the site as shown on the Fault Study Map, Figure 8:

- *2025 Harbor Drive (Geocon, Inc., 2000; Project No. 06155-22-06);*
- *S. Evans Street, Main Street and Newton Avenue (Geocon, Inc., 1993; Project No. 04749-31-02).*

Based on our review of these documents, active faulting or off-fault deformation in the immediate site vicinity is not present. Trenches were excavated on nearby sites to the northwest (Geocon, 1993 and 2000), and no active or potentially active faults were observed at these sites

The closest known active faults are located approximately 4,000 feet to the west within the state-designated Alquist-Priolo Earthquake Fault Zone as shown on the Downtown Special Fault Zone Map, Figure 7. The trend of nearby active faults ranges from N16W to N16E.

7.3 Field Exploration

To investigate the presence or absence of faults at the site, we observed a trench excavation across the property, through the existing asphalt parking lot (Figure 2). As previously described, the predominant trend of documented active and potentially active faults in the area is N16W to N16E. The orientation of our exploratory trench (N70W to N83W) was selected to evaluate this trend and a 30-degree variation of this trend in either direction. Our fault trench does not provide coverage for the southwestern section of the subject property that is proposed to be a parking lot and/or a storm water management device. A detailed log of the south-facing wall of the trench is provided in Figure 4.

7.4 Trench Stratigraphy

The sediments exposed in the trench consist of Old Paralic Deposits, mapped as Unit 6 (Kennedy and Tan, 2008). The San Diego Formation, which often underlies the Old Paralic Deposits in the downtown San Diego area, was not encountered below the site to the maximum depth explored. We classified the sediments within the trench in accordance with the Unified Soil Classification System (USCS) as well as applicable soil taxonomy criteria. The Old Paralic Deposits were divided into three distinct, continuous or relatively continuous Horizons, E, B and C, which were further subdivided where other dominant soil characteristics were observed. An A-Horizon was also uncounted in limited areas, which may have been removed during original site grading. Detailed descriptions of the units are presented on the fault trench log (Figure 4).

The Old Paralic Deposits exposed in the trench generally consist of dense to very dense, brown, reddish brown, grayish brown and yellowish brown, silty and clayey, fine- to coarse-grained sand with variable amounts of fine angular gravel. Beds were generally massive, except for localized channeling. We also observed district lateral variations in grain size within the same beds related to changes in deposition. The base of the exposed Old Paralic Deposits was characterized by a medium- to coarse-grained sand unit that is weakly laminated and locally cross-bedded. The entire stratigraphic sequence is moderately to highly oxidized, with the exception of the lowest portion of the trench, which is unoxidized in some areas. The Old Paralic Deposits are interpreted to be continuous laterally and vertically within the fault trench exposure, and in the small diameter borings to maximum depth explored.

The primary marker bed that infers an un-faulted stratigraphic sequence across the site is the medium- to coarse-grained sand unit observed in the lower third of the trench referred to as Qop_6 (Cv) on the fault trench log (Figure 4). This unit is massive and locally laminated and/or overprinted by laminar oxide films (b-Lams) and in some areas, cross-bedded with well-defined foresets. In general, this unit does not appear to be related to overlying soil development, and is therefore considered equivalent to a C-Horizon. The upper contact with the overlying B-Horizon is sub-horizontal and undulatory implying localized erosion and scour prior to deposition of the overlying sediments.

The overlying B-Horizon is also continuous and unbroken along the length of the fault trench (Figure 4). However, there is some lateral variation within this unit related to changes in sediment deposition, variability in the accumulation of illuvial clays, secondary development of interstitial carbonate and silicate cements and construction of the existing building and parking lot. These lateral variations are typically observed to occur over several feet. For example, a lateral transition from a clayey sand to a clayey sand with gravel, rather than abrupt changes across a discontinuity that may be related to faulting.

Characteristic features observed in the fault trench that infer unbroken/unfaulted stratigraphy at the site are summarized in Table 7.4.

TABLE 7.4
SUMMARY OF MARKER BED CHARACTERISTICS

Fault Trench Unit No.	General Stratigraphic Description
Ci-iii	Continuous B-Horizon, locally subdivided into Bt-, Bk-, and Bkm- Horizons
Cv	Continuous C-Horizon

7.5 Absence of Faulting

As shown on Figure 4, the Pleistocene age geologic units are laterally continuous across the trench. The primary evidence for the absence of active faulting are:

1. No faults documented in the immediate area by Geocon Inc., or other consultants, were observed to project toward the site.
2. The Old Paralic Deposits (minimum age of 120,000 years) were observed to be laterally continuous in the exploratory trench and on adjacent sites to the west (Geocon, 1993), and no faults or fault-related features were observed.

The age, lateral continuity, and lack of deformation of these distinct geologic units, provide clear evidence for continuous, unfaulted, pre-Holocene age sediments across the site and rules out active faulting. Therefore, it is our opinion active, potentially active or inactive faulting is not present on the property. Structural setbacks will not be required for the planned development.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

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

8.1.2 With the exception of possible moderate to strong seismic shaking, we did not observe significant geologic hazards or know of them to exist on the site that would adversely affect the proposed project.

8.1.3 The site is not located within a State of California Earthquake Fault Zone but is located within a fault study zone established by the City of San Diego. Our review of fault investigations for the adjacent properties and our observations during our exploratory operations indicate that there is no evidence of active or potentially active faults traversing the site. The exposed stratigraphic section of Pleistocene aged Old Paralic Deposits observed during trenching is generally horizontal to sub-horizontal and unbroken. We did not observe evidence of shearing, fracturing or offset along sub-vertical discontinuity. It is our opinion that active or potentially active faulting does not pass beneath the site and building setbacks will not be required.

8.1.4 Restrictions on future development at the site are not necessary with respect to the hazard of surface fault rupture. However, a future earthquake originating on a nearby splay of the Rose Canyon Fault could produce very strong near-field ground motions at the site that should be taken into consideration during project design. Also, there is a potential for ground cracking or ground shatter associated with strong ground shaking during an earthquake event on nearby faults to occur beneath the site. The findings of our study are limited to detection of existing seismogenic faults (deep-seated structures) that propagate to the near surface and cannot predict the location of ground shatter associated with strong ground shaking.

8.1.5 Our field investigation indicates the site is underlain by undocumented fill overlying Old Paralic Deposits. The Old Paralic Deposits are considered suitable for the support of settlement-sensitive structures.

8.1.6 We did not encounter groundwater during our field investigation to the maximum depth explored of 51½ feet below the former ground surface or at approximate elevation of 8½ feet above MSL. It is typical to see groundwater from 0 to 5 feet above MSL in the subject area. The proposed bottom elevation of the excavation for the subterranean structure is at least 30 feet above groundwater. Therefore, we do not expect groundwater will be encountered during construction of the proposed development.

8.1.7 The proposed structure can be supported on conventional shallow foundations system founded in Old Paralic Deposits.

8.1.8 We expect the temporary excavations for the parking garage will be supported by a soldier pile and, if necessary, tieback anchor system.

8.1.9 Based on our review of the project plans, we opine the planned development can be constructed in accordance with our recommendations provided herein. We do not expect the planned development will destabilize or result in settlement of adjacent properties or the existing public improvements and street right-of-ways located adjacent to the site if the recommendations of this report are incorporated into project design.

8.1.10 We performed a storm water management investigation to help evaluate the potential for infiltration on the property. Based on the results of our field infiltration testing and laboratory testing, we opine full or partial infiltration on the property should be considered infeasible as discussed in Appendix C.

8.2 Excavation and Soil Conditions

8.2.1 Excavations within the Old Paralic Deposits should generally be possible with moderate to heavy effort using conventional heavy-duty equipment. Localized cemented or very hard zones will likely be encountered that will require very heavy effort to excavate with oversize material generated. The Old Paralic Deposits also can contain cobble and cohesionless sand layers. The contractors should be prepared to handle the potential for seepage and caving during the construction operations.

8.2.2 The soil encountered in our field investigation is considered to be “non-expansive” (expansion index [EI] of 20 or less) as defined by 2016 California Building Code (CBC) Section 1803.5.3. However, some of the soil may be classified and “expansive” (expansion index of greater than 20). Table 8.2 presents soil classifications based on the expansion index. Based on the results of our laboratory testing, presented in Appendix A, we expect the on-site materials will possess a “very low” to “low” expansion potential (expansion index of 50 or less).

TABLE 8.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

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

8.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

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

8.3 Grading

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

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

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

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

8.3.5 Based on our understanding of the project and the results of our prior field investigation, we expect the existing fill and some of the Old Paralic Deposits will be removed during the excavations for the planned subterranean levels and the clayey/silty sand materials of the Old Paralic Deposits will be exposed at the base of the subterranean levels. The actual extent of removals shall be determined in the field by Geocon Incorporated.

8.3.6 Excavated soil that is generally free of deleterious debris and contamination can be placed as fill and compacted in layers to the design finish-grade elevations, if necessary. Fill and backfill materials that will require placement for elevators or adjacent surface improvements should be placed in loose thicknesses of 6 to 8 inches and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM Test Method D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

8.3.7 Import fill (if necessary) should consist of granular materials with a “very low” to “low” expansion potential (EI of 50 or less) free of deleterious material or stones larger than 3 inches and should be compacted as recommended herein. Geocon Incorporated should be notified of the import source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

8.4 Excavation Slopes, Shoring, and Tiebacks

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

8.4.2 Temporary excavations should be made in conformance with OSHA requirements. Undocumented fill should be considered a Type C soil in accordance with OSHA requirements. Compacted fill materials can be considered a Type B soil (Type C soil if seepage or groundwater is encountered) and the Old Paralic Deposits can be considered a Type A soil (Type B soil if seepage or groundwater is encountered). In general, special shoring requirements will not be necessary if temporary excavations will be less than 4 feet in height and raveling of the excavations foes not occur. Temporary excavations greater than 4 feet in height, however, should be sloped back at an appropriate inclination. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

8.4.3 The design of temporary shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging. Excavations exceeding 15 feet (with a level backfill) may require soil nails, tieback anchors, or internal bracing to provide additional wall restraint.

8.4.4 Temporary shoring with a level backfill should be designed using a lateral pressure envelope acting on the back of the shoring and applying a pressure equal to 18H, 12H, or 14H, for a triangular, rectangular, or trapezoidal distribution, respectively, where H is the height of the shoring in feet (resulting pressure in pounds per square foot) as shown in Figure 9. These pressures assume a shoring height of up to about 25 feet and we should be contacted if deeper excavations are planned. Triangular distribution should be used for cantilevered shoring and, the trapezoidal and rectangular distribution should be used for multi-braced systems such as tieback anchors and rakers. The project shoring engineer should determine the applicable soil distribution for the design of the temporary shoring system. Additional lateral earth pressure due to the surcharging effects from construction equipment, sloping backfill, planned stockpiles, adjacent structures and/or traffic loads should be considered, where appropriate, during design of the shoring system.

8.4.5 Passive soil pressure resistance for embedded portions of soldier piles can be based on an equivalent passive soil fluid weight of 400D + 500 psf where D is the depth of embedment, in feet (resulting in pounds per square foot), as shown on Figure 10. This passive resistance assumes we do not encounter the groundwater during the installation of the soldier piles. The passive resistance can be assumed to act over a width of three pile diameters. Typically, soldier piles are embedded a minimum of 0.5 times the maximum height of the excavation (this depth is to include footing excavations) if tieback anchors are not employed. The project structural engineer should determine the actual embedment depth.

8.4.6 Drilled shafts for the soldier piles should be observed by Geocon Incorporated prior to the placement of steel reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata, and design depths. If unexpected soil conditions are encountered, foundation modifications may be required.

8.4.7 Lateral movement of shoring is associated with vertical ground settlement outside of the excavation. Therefore, it is essential that the soldier pile and tieback system allow very limited amounts of lateral displacement. Earth pressures acting on a lagging wall can cause movement of the shoring toward the excavation and result in ground subsidence outside of the excavation. Consequently, horizontal movements of the shoring wall should be accurately monitored and recorded during excavation and anchor construction.

8.4.8 Survey points should be established at the top of the pile on at least 20 percent of the soldier piles. An additional point located at an intermediate point between the top of the pile and the base of the excavation should be monitored on at least 20 percent of the piles if tieback anchors will be used. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter until the permanent support system is constructed.

8.4.9 The shoring system should be designed to limit horizontal and vertical soldier pile movement to a maximum of 1 inch and ½ inch, respectively. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary. The magnitude of movement for intermediate depths and distances from the shoring wall can be linearly interpolated.

8.4.10 The project civil engineer should provide the approximate location, depth, and pipe type of the underground utilities adjacent to the site to the shoring engineer to help select the appropriate shoring type and design. The shoring system should be designed to limit horizontal and vertical soldier pile movement to a maximum of 1 inch and ½ inch, respectively. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary. The magnitude of movement for intermediate depths and distances from the shoring wall can be linearly interpolated. We understand the City of San Diego may require the developer to prepare a hold harmless agreement for the planned construction and development regarding potential damage to the existing utilities and improvements.

8.4.11 Tieback anchors employed in shoring should be designed such that anchors fully penetrate the Active Zone behind the shoring. The Active Zone can be considered the wedge of soil from the face of the shoring to a plane extending upward from the base of the excavation at a 29-degree angle from vertical, as shown on Figure 11. Normally, tieback anchors are contractor-designed and installed, and there are numerous anchor construction methods available. Non-shrinkage grout should be used for the construction of the tieback anchors.

8.4.12 Experience has shown that the use of pressure grouting during formation of the bonded portion of the anchor will increase the soil-grout bond stress. A pressure grouting tube should be installed during the construction of the tieback. Post grouting should be performed if adequate capacity cannot be obtained by other construction methods.

8.4.13 Anchor capacity is a function of construction method, depth of anchor, batter, diameter of the bonded section, and the length of the bonded section. Anchor capacity should be evaluated using the strength parameters shown in Table 8.4.

TABLE 8.4
SOIL STRENGTH PARAMETERS FOR TEMPORARY SHORING

Description	Cohesion (psf)	Friction Angle (degrees)
Old Paralic Deposits (Qop)	450	33

8.4.14 Grout should only be placed in the tieback anchor's bonded section prior to testing. Tieback anchors should be proof-tested to at least 130 percent of the anchor's design working load. Following a successful proof test, the tieback anchors should be locked off at 80 percent of the allowable working load. Tieback anchor test failure criteria should be established in project plans and specifications. The tieback anchor test failure criteria should be based upon a maximum allowable displacement at 130 percent of the anchor's working load (anchor creep) and a maximum residual displacement within the anchor following stressing. Tieback anchor stressing should only be conducted after sufficient hydration has occurred within the grout. Tieback anchors that fail to meet project specified test criteria should be replaced or additional anchors should be constructed.

8.4.15 Lagging should keep pace with excavation and tieback anchor construction. The excavation should not be advanced deeper than three feet below the bottom of lagging at any time. These unlagged gaps of up to three feet should only be allowed to stand for short periods of time in order to decrease the probability of soil instability and should never be unsupported overnight. Backfilling should be conducted when necessary between the back of lagging and excavation sidewalls to reduce sloughing in this zone and all voids should be filled by the end of each day. Further, the excavation should not be advanced further than four feet below a row of tiebacks prior to those tiebacks being proof tested and locked off.

8.4.16 If tieback anchors are employed, an accurate survey of existing utilities and other underground structures adjacent to the shoring wall should be conducted. The survey should include both locations and depths of existing utilities. Locations of anchors should be adjusted as necessary during the design and construction process to accommodate the existing and proposed utilities.

8.4.17 If a raker system is employed, the rakers should not be inclined steeper than 1:1 (horizontal to vertical) to provide an excavation to the raker foundation system with an inclination less than 1:1. A shallow or deep foundation system can be used for the raker system.

8.4.18 Shallow foundations for the raker system should consist of continuous strip footings and/or isolated spread footings. Continuous and isolated footings should be at least 24 inches wide and extend at least 12 inches below lowest adjacent pad grade. Steel reinforcement for the footings should be designed by the project structural engineer. Foundations may be designed for an allowable soil bearing pressure of 4,000 psf for footings bearing in the Old Paralic Deposits.

8.4.19 The condition of existing buildings, streets, sidewalks, and other structures/improvements around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements and other improvements. Underground utilities sensitive to settlement should be videotaped prior to construction to check the integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation and upon existing buildings. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter. Inclinometers should be installed and monitored behind any shoring sections that will be advanced deeper than 30 feet below the existing ground surface.

8.5 Seismic Design Criteria

8.5.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 8.5.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements should be designed using a Site Class C. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 8.5.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 8.5.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.210g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.466g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.000	Table 1613.3.3(1)
Site Coefficient, F_V	1.334	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.210g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.622g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.807g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.414g	Section 1613.3.4 (Eqn 16-40)

8.5.2 Table 8.5.2 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 8.5.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.542g	Figure 22-7
Site Coefficient, F_{PGA}	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.542g	Section 11.8.3 (Eqn 11.8-1)

8.5.3 Conformance to the criteria in Tables 8.5.1 and 8.5.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.6 Conventional Shallow Foundations

8.6.1 The proposed structure can be supported on a conventional shallow foundation system bearing on the properly compacted fill. Foundations for the structures should consist of continuous strip footings and/or isolated spread footings. Continuous footings should be at least 12 inches wide and extend at least 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 24 inches and depth of 24 inches. Figure 12 presents a footing dimension detail depicting the depth to lowest adjacent grade.

8.6.2 Steel reinforcement for continuous footings should consist of at least four No. 4 steel reinforcing bars placed horizontally in the footings, two near the top and two near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer. The minimum reinforcement recommended herein is based on soil characteristics only (Expansion Index of 50 or less) and is not intended to replace reinforcement required for structural considerations.

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

8.6.4 The recommended allowable bearing capacity for foundations with minimum dimensions described herein and bearing in formational materials at least 10 feet below the ground surface is 6,000 pounds per square foot (psf). An additional 1,000 psf can be added to the allowable bearing capacity for excavations of 20 feet or greater below the ground surface. The allowable soil bearing pressure may be increased by an additional 500 psf for each additional foot of depth and 300 psf for each additional foot of width, to a maximum allowable bearing capacity 8,000 psf. The values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces. These values are based on an anticipated maximum excavation depth of 25 feet.

8.6.5 Total and differential settlement of the building founded on the Old Paralic Deposits is expected to be less than ½-inch for a 9-foot square footing. The total and differential settlement for a 16-foot square footing is 1 inch and ½ inch, respectively.

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

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

8.7 Concrete Slabs-on-Grade

8.7.1 Interior concrete slabs-on-grade for the subterranean parking structure should be at least 5 inches thick. As a minimum, reinforcement for slabs-on-grade should consist of No. 4 reinforcing bars placed at 18 inches on center in both horizontal directions.

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

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

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

8.7.5 To control the location and spread of concrete shrinkage cracks, crack control joints should be provided. The crack control joints should be created while the concrete is still fresh using a grooving tool, or shortly thereafter using saw cuts. The structural engineer should take into consideration criteria of the American Concrete Institute when establishing crack control spacing patterns.

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

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

8.8 Concrete Flatwork

8.8.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and, when in excess of 8 feet square, should be reinforced with 6 x 6 - W2.9/W2.9 (6 x 6 - 6/6) welded wire mesh or No. 3 reinforcing bars at 18 inches on center in both directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be checked prior to placing concrete.

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

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

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

8.9 Retaining Walls

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

8.9.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform (rectangular) pressure of 7H psf and 13H psf should be added to the active soil pressure where the planned walls are 8 feet or less and the portion of walls greater than 8 feet, respectively. 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. In addition, the loading from adjacent structures should be incorporated into the design of the planned retaining walls by the structural engineer.

8.9.3 The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted free-draining backfill material (EI of 50 or less) with no hydrostatic forces or imposed surcharge load. Figures 13 and 14 present typical retaining wall drain details for conventional and soldier pile walls. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

8.9.4 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure. A seismic load of 18H psf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. We used the site specific peak ground acceleration, PGA_M, of 0.542g calculated from ASCE 7-10 Section 11.8.3.

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

8.10 Lateral Loading

8.10.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 350 pounds per cubic foot (pcf) should be used for the design of footings or shear keys poured neat in compacted fill. The passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

8.10.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.35 should be used for design. The friction coefficient may be reduced depending on the vapor barrier or waterproofing material used for construction in accordance with the manufacturer’s recommendations (typically a reduced friction coefficient of about 0.2 to 0.25).

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

8.11 Preliminary Pavement Recommendations

8.11.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the

R-Value of the subgrade soil encountered at final subgrade elevation. Based on the results of our R-value testing of the subgrade soils, we have assumed an R-Value of 6 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 8.11.1 presents the preliminary flexible pavement sections.

TABLE 8.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	6	3	10
Driveways for automobiles and light-duty vehicles	5.5	6	3	12
Medium truck traffic areas	6.0	6	3.5	13
Driveways for heavy truck traffic	7.0	6	4	16

8.11.2 The subgrade soils for pavement areas should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above the optimum moisture content. The depth of subgrade compaction should be approximately 12 inches.

8.11.3 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 (Greenbook)*.

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

8.11.5 A rigid Portland Cement concrete (PCC) pavement section should be placed in driveway entrance aprons, trash bin loading/storage areas and loading dock areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 8.11.2.

TABLE 8.11.2
RIGID PAVEMENT DESIGN PARAMETERS

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

8.11.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.11.3.

TABLE 8.11.3
RIGID PAVEMENT RECOMMENDATIONS

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

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

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

8.11.9 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 20 feet for the slabs and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report. The depth of the crack-control joints should be at least ¼ of the slab thickness when using a conventional saw, or at least 1 inch when using early-entry saws on slabs 9 inches or less in thickness, as determined by the referenced ACI report discussed in the pavement section herein. Cuts at least ¼ inch wide are required for sealed joints, and a ⅜ inch wide cut is commonly recommended. A narrow joint width of 1/10 to 1/8 inch-wide is common for unsealed joints.

8.11.10 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed at the as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

8.11.11 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, cross-gutters, or sidewalk so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

8.12 Site Drainage and Moisture Protection

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

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

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

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

8.13 Improvement/Grading and Foundation Plan Review

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

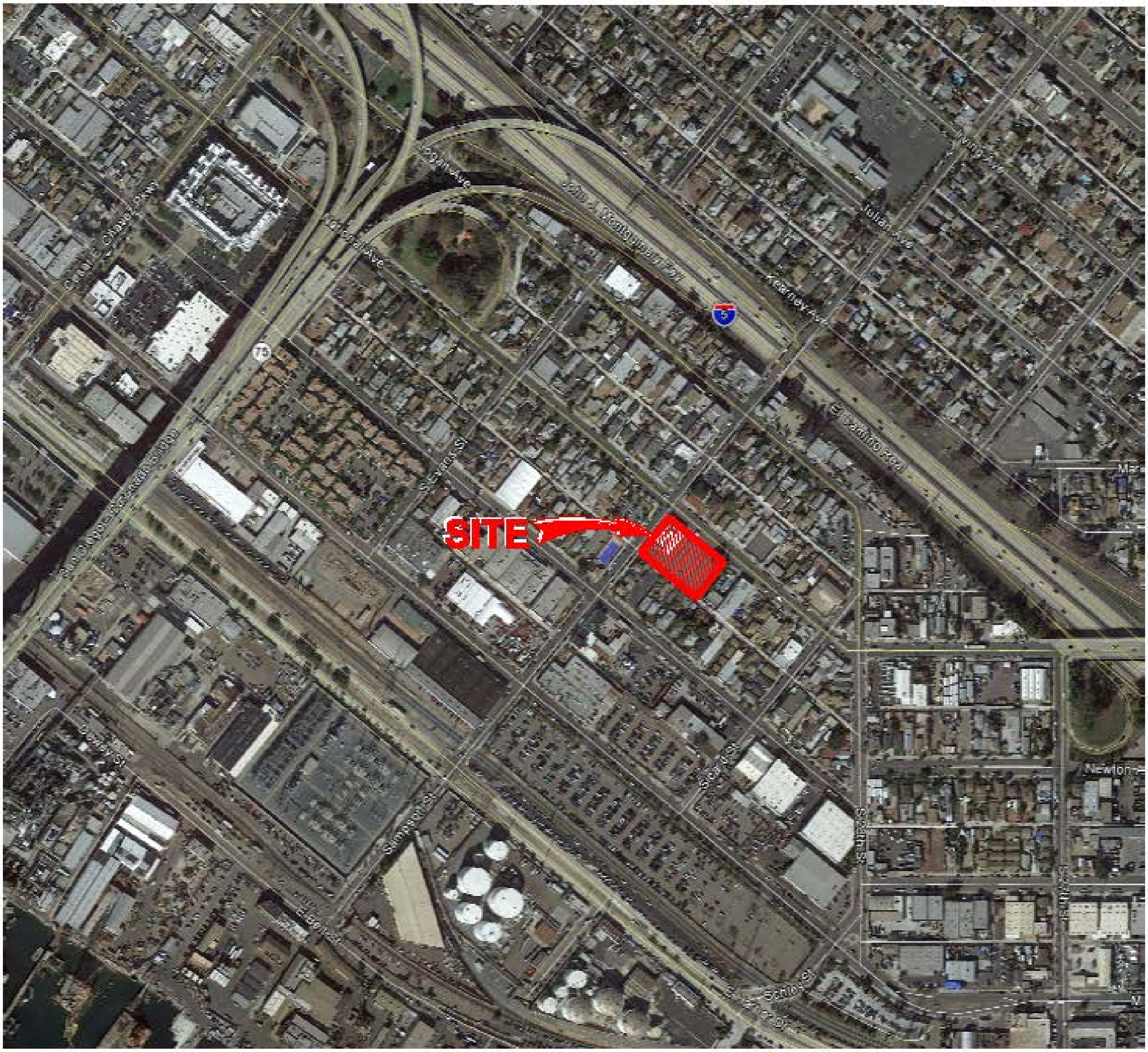
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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



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.

N

NO SCALE

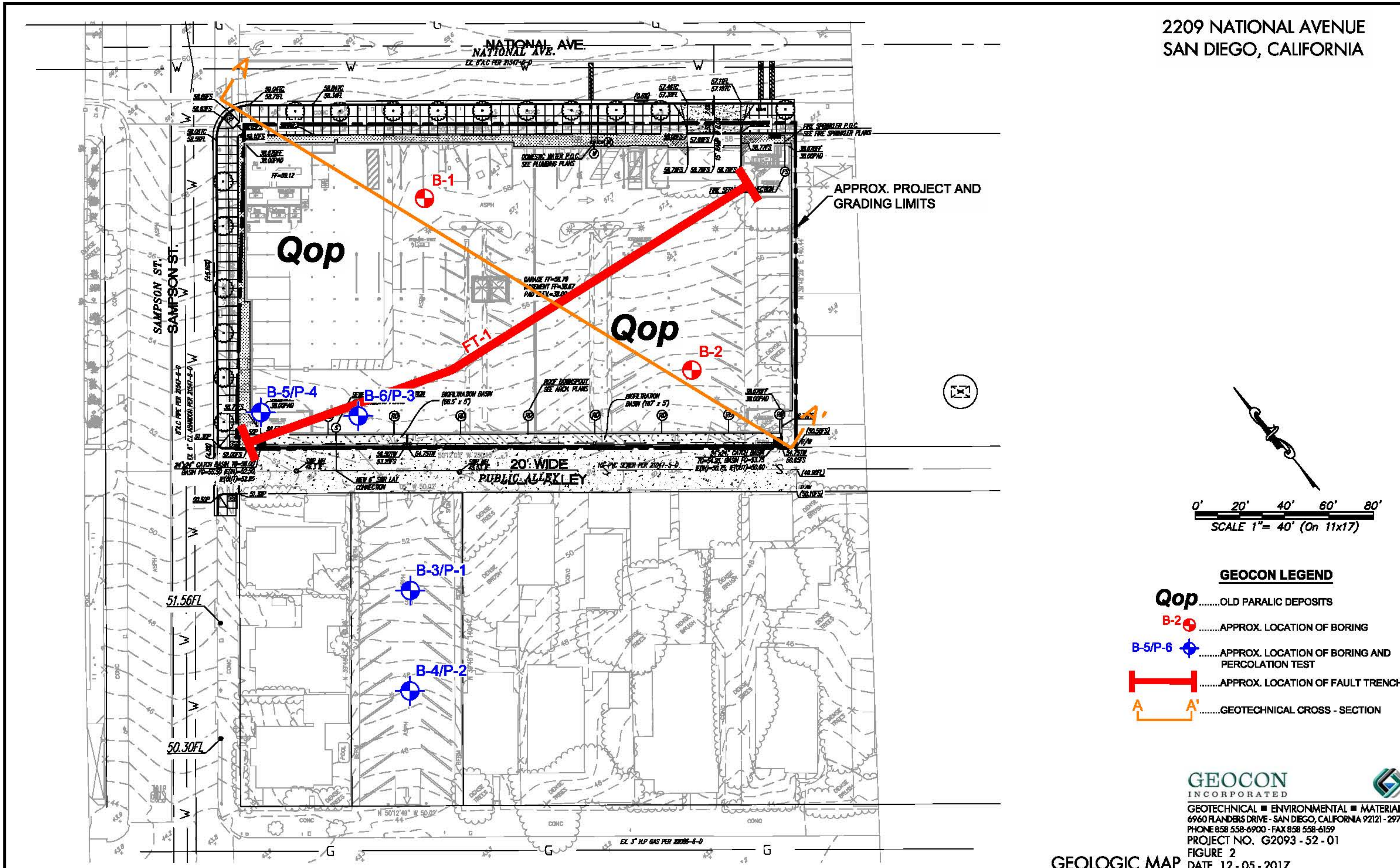
VICINITY MAP

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 06 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 1

Plotted:12/04/2017 4:36PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\DETAILS\G2093-52-01_Vicinity Map.dwg



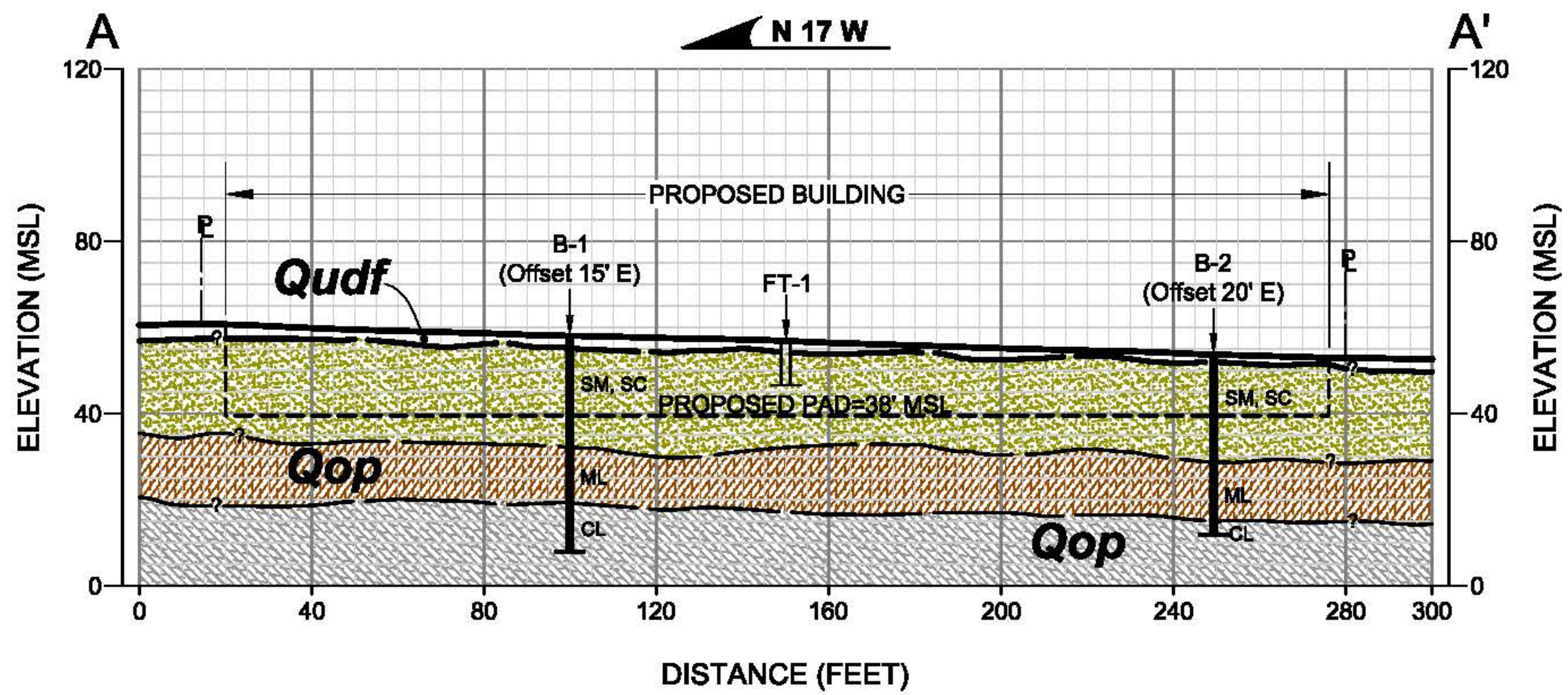
GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

GEOLOGIC MAP

PROJECT NO. G2093 - 52 - 01
FIGURE 2
DATE 12 - 05 - 2017

Plotted:12/04/2017 4:14PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\SHEETS\G2093-52-01 GeologicMAP.dwg

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION A-A'

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

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qop ....OLD PARALIC DEPOSITS

Silty/Clayey Sand

Silt

Clay

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

FT-1 ....APPROX. LOCATION OF TRENCH

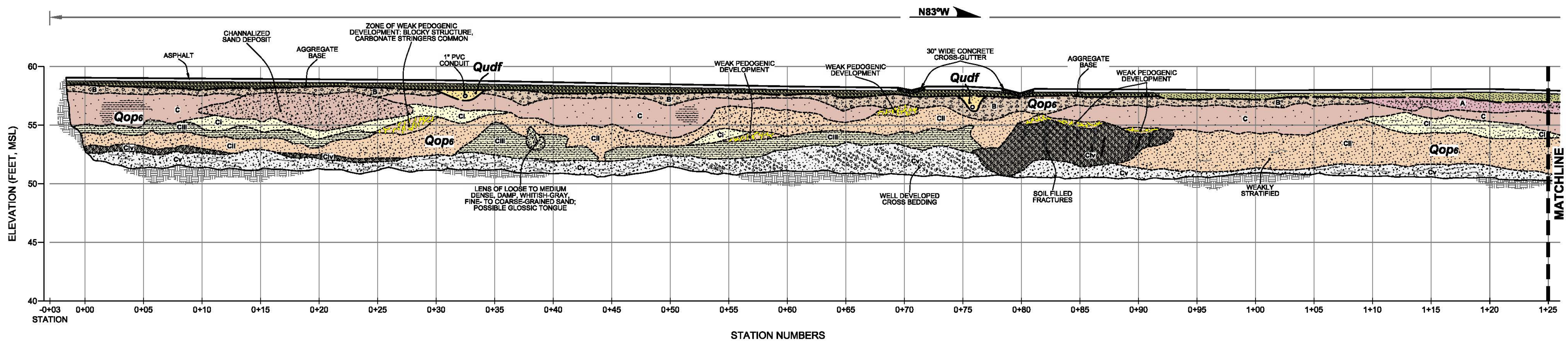
? ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

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

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

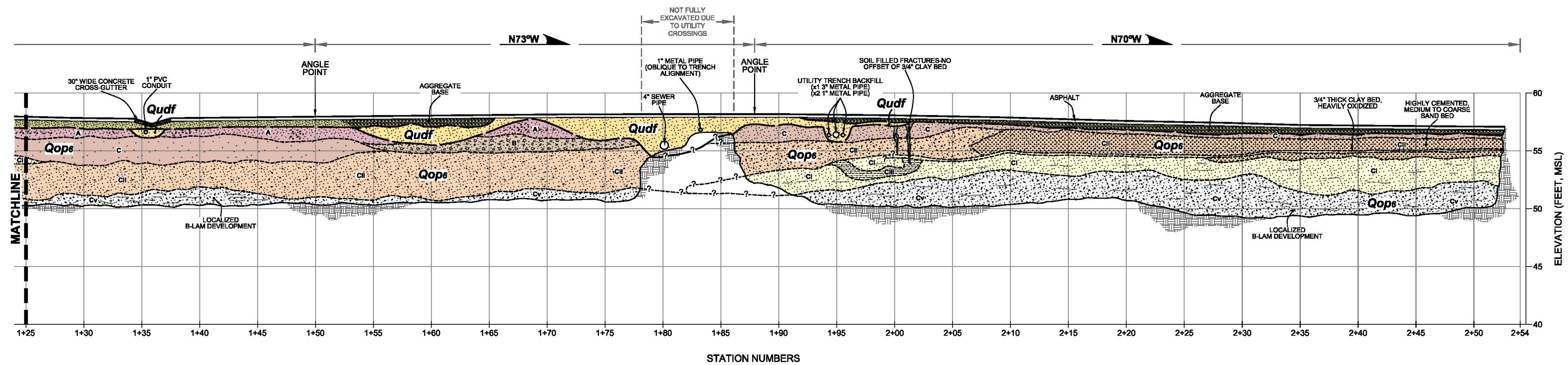
GEOLOGIC CROSS - SECTION

PROJECT NO. G2093 - 52 - 01
FIGURE 3
DATE 12 - 05 - 2017



FAULT TRENCH

SCALE: 1" = 5' (Vert. = Horiz.)



FAULT TRENCH

SCALE: 1" = 5' (Vert. = Horiz.)

PAVEMENT SECTION:

a. Station 0+00 to 0+92: 3 to 4 inches asphaltic concrete over 6 inches of aggregate base. At least one overlay present, with petromat observed in some areas.

b. Station 0+92 to 1+52: 3 to 4 inches of asphaltic concrete over dense, dry to damp, gray, medium grained Silty Sand (SM).

c. Station 1+52 to 1+65: 3 to 4 inches asphaltic concrete over 6 inches of aggregate base.

d. Station 1+65 to 1+91.5: 4 inches asphaltic concrete over subgrade soil consisting of medium dense, damp to moist, brown to grayish brown Clayey Sand (SC) and Silty Sand (SM).

e. Station 1+91.5 to 2+52.5: 3 inches asphaltic concrete over 6 inches of aggregate base.

Qudf ....**UNDOCUMENTED FILL:** Loose to medium dense, damp to moist, brown, yellowish brown to grayish brown (mottled) Silty Sand (SM) and Clayey Sand (SC) mixture; trace rock fragments <1.5 inches and occasional trash and debris observed. Fill soil is confined to zones of localized trench backfill with the exception of station 1+54 to 2+05 where fill is present below the pavement section to depths up to 4 feet below existing grade.

Qop6 ....**OLD PARALIC DEPOSITS (Late to middle Pleistocene):** Poorly sorted, moderately permeable, reddish-brown, inter-fingered strandline beach, estuarine and colluvial deposits of siltstone, sandstone and conglomerate. These deposits rest on the 22-23m Nestor terrace (Kennedy and Tan, 2008). This unit is further subdivided on the fault trench log as follows:

A ....Stiff, moist, red to reddish-brown Clay (CL); trace fine gravel and coarse sand. Localized pedogenic development with 4-6 inch tall peds in some areas. Carbonate stringers common, locally reworked during paving operations. Possibly equivalent to an A-Horizon.

B ....Dense, damp to moist, brown to grayish-brown, fine grained Silty Sand (SM); massive with occasional fine, <1/4-inch angular gravel. Locally reworked and/or bisected by trench backfill. Equivalent to an E-Horizon due to low organic content and light color characteristic of oxide leaching.

C ....Dense, damp, brown, fine to medium grained Silty Sand (SM) with trace clay; occasional fine, <1/4-inch angular gravel and <1/8 inch manganese nodules. Pinhole porosity common throughout, generally massive, but locally channelized as noted on log. Equivalent to a B-Horizon, but can be further subdivided as noted:

i ....Dense, damp to moist, pale yellowish-brown to grayish-brown (Mottled), Clayey Sand (SC); discontinuous lenses, pods and films of clay throughout (translocated clays), laterally discontinuous. Equivalent to a Bt-Horizon.

ii ....Very Dense, dry to damp, pale reddish-brown to orange brown, fine to coarse grained Silty Sand (SM); weakly to moderately cemented by interstitial carbonate as noted on log. Localized zones of weak pedogenic development noted at contact with overlying B-Horizon, often with localized fracture infills. Equivalent to a Bk-Horizon. Grades laterally into dense to very dense, reddish-brown, medium to coarse grained Clayey Sand with gravel. Heavily cemented with non-carbonate cement north of station 2+06 (Bm-Horizon)

iii ....Very dense, dry, white to pale brown, fine grained Sand; heavily cemented, oxide coatings noted along fractures. Laterally discontinuous with variable thickness. Equivalent to a Bkm-Horizon.

iv ....Dense, damp, gray to grayish-brown, very fine grained Sandy Clay (SC-CL); laterally discontinuous, interfingers with sand below.

v ....Loose to medium dense, whitish-gray to orange brown, medium to coarse grained Sand (SP); Laminated and locally cross bedded. Subhorizontal B-Lams defined by oxide grain coatings noted in some areas. Equivalent to a C-Horizon.

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

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

........INTERSTITIAL CARBONATE DEVELOPMENT

........HEAVY CEMENTATION

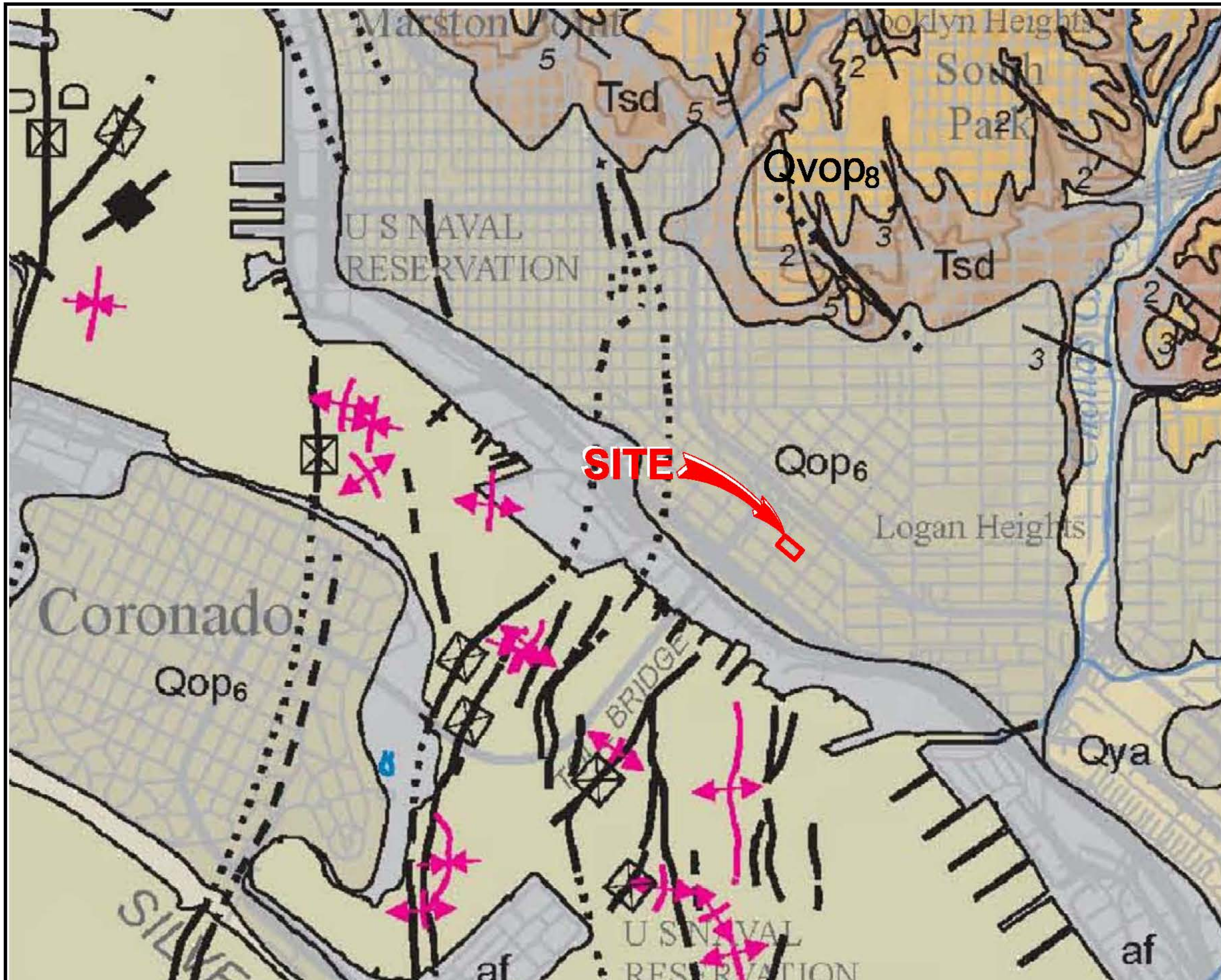
FAULT TRENCH FT-1

NATIONAL AND SAMPSON
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" = 5'	DATE 12 - 05 - 2017
PROJECT NO. G2093 - 52 - 01	FIGURE 4
SHEET 1 OF 1	

Plotted:12/05/2017 7:30AM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\SHEETS\G2093-52-01 FaultTrench.dwg



2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

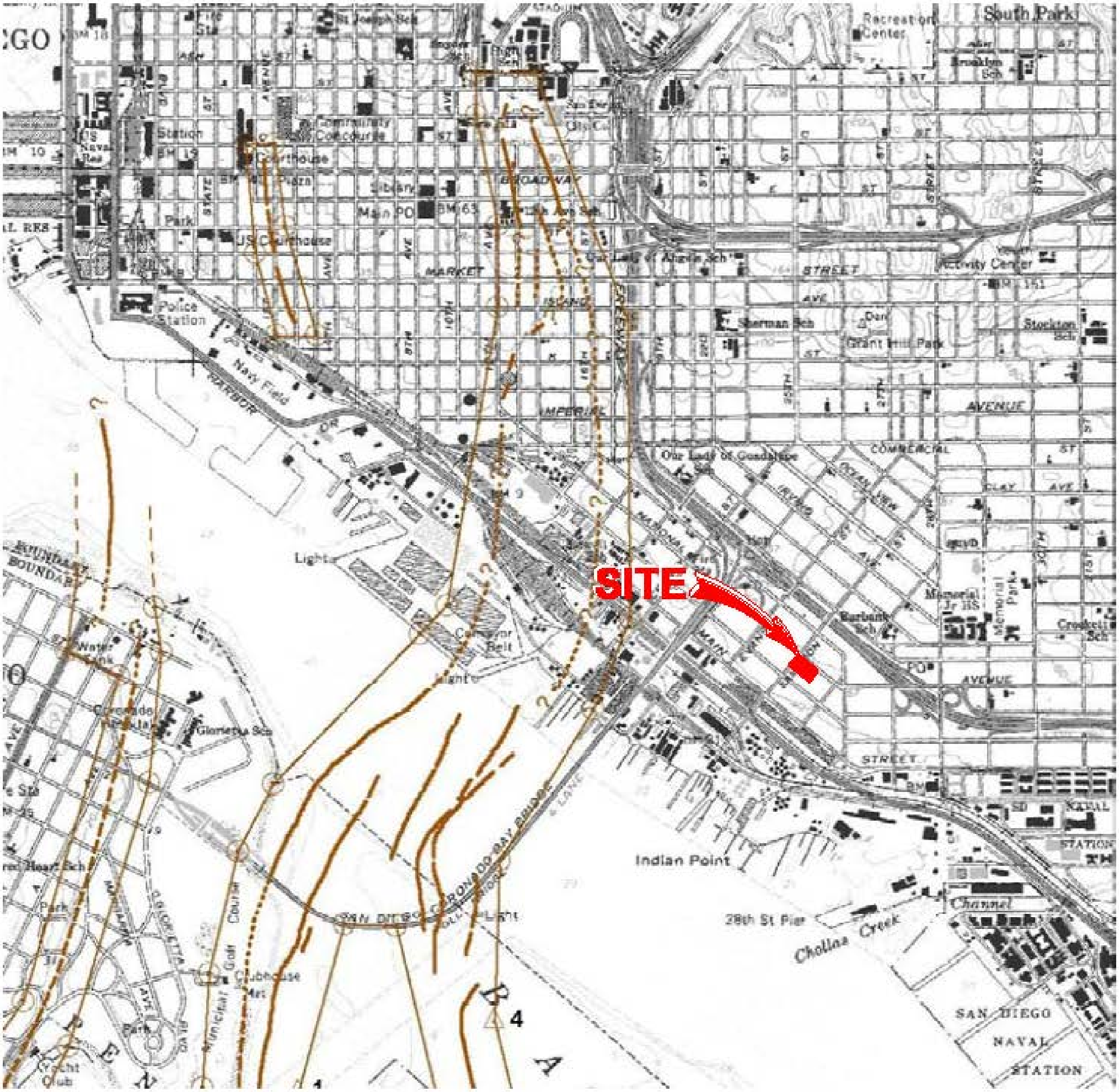
GEOCON LEGEND

af ....ARTIFICIAL FILL
Qop$_6$ ....OLD PARALIC DEPOSITS
Qvop$_8$ ....VERY OLD PARLIC DEPOSITS
Tsd ....SAN DIEGO FORMATION

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

PROJECT NO. G2093 - 52 - 01
FIGURE 5
DATE 12 - 05 - 2017

REGIONAL GEOLOGIC MAP



REGIONAL FAULT MAP

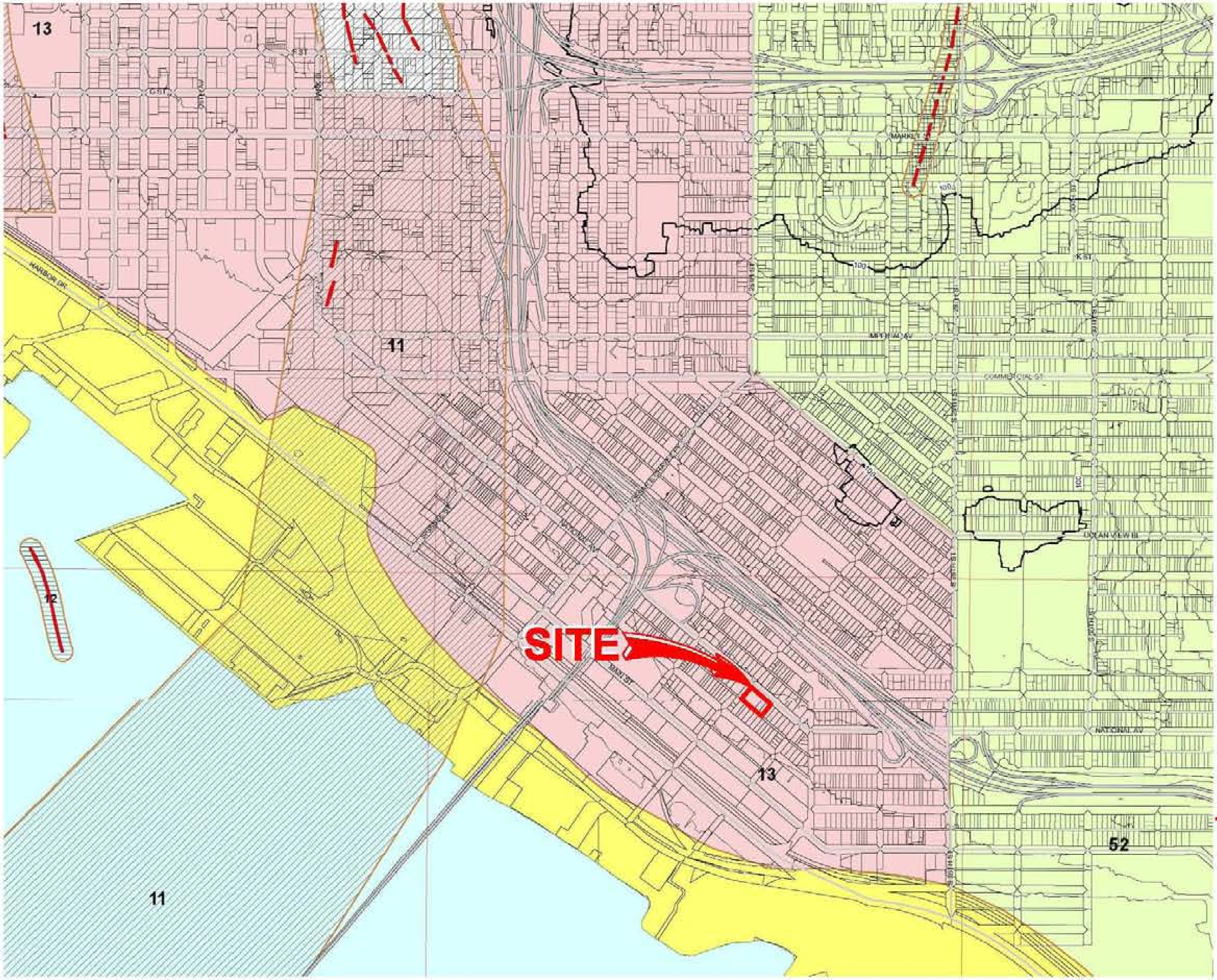
GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 05 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 6

Plotted:12/04/2017 4:40PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\DETAILS\G2093-52-01 RegionalFaultMap.dwg

2209 NATIONAL AVENUE
DIEGO, CALIFORNIA



0' 1000' 2000'
SCALE 1"= 1000' (On 11x17)

........DOWNTOWN SPECIAL FAULT ZONE

........HIGH POTENTIAL–SHALLOW GROUNDWATER MAJOR DRAINAGES, HYDRAULIC PILLS

........OTHER LEVEL AREAS, GENTLY SLOPING TO STEEP TERRAIN, FAVORABLE GEOLOGIC STRUCTURE, LOW RISK

........WATER

11 ....ACTIVE ALQUIST-PRIOLO EARTHQUAKE FAULT ZONE

12 ....POTENTIALLY ACTIVE, INACTIVE, PRESUMED INACTIVE OR ACTIVELY UNKNOWN

........FAULT (Dashed Where Inferred)

GEOCON
INCORPORATED

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

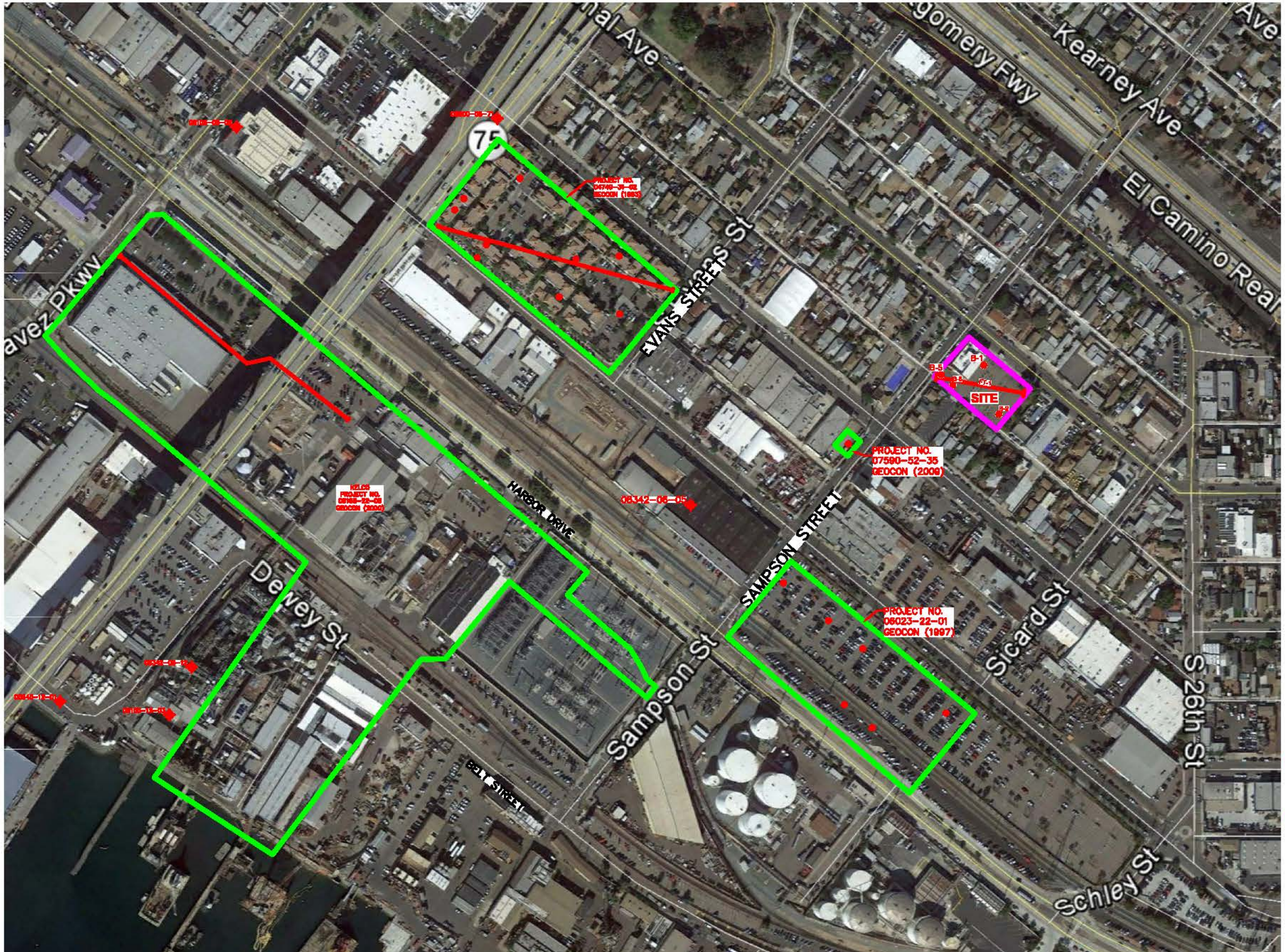
PROJECT NO. G2093 - 52 - 01

DOWNTOWN SPECIAL FAULT ZONE MAP

FIGURE 7
DATE 12 - 05 - 2017

Plotted:12/04/2017 4:36PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\SHEETS\G2093-52-01 DowntownSpecialFaultMAP.dwg

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA



0' 300' 600'
SCALE 1"= 300' (On 11x17)

GEOCON LEGEND

B-6 ....APPROX. LOCATION OF BORING

FT-1 ....APPROX. LOCATION OF FAULT TRENCH

........APPROX. LOCATION OF FAULT TRENCH, COMPANY THAT PERFORMED THE TRENCH (Year Reported)

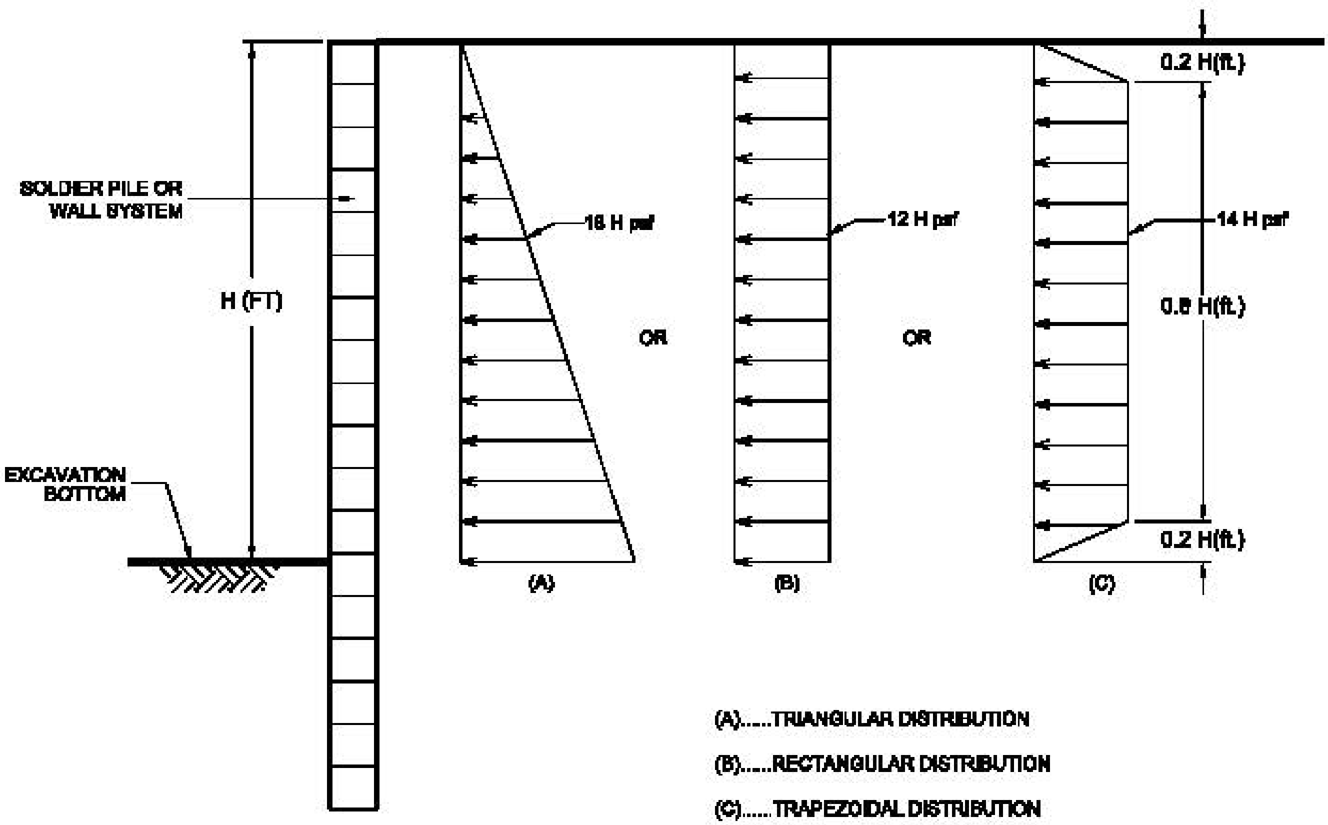
........APPROX. LOCATION OF PREVIOUS PROJECT BOUNDARY

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

FAULT STUDY MAP

PROJECT NO. G2093 - 52 - 01
FIGURE 8
DATE 12 - 05 - 2017

Plotted:12/04/2017 4:31PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\SHEETS\G2093-52-01 SD-DwntnFaultStudyMap.dwg



(A)......TRIANGULAR DISTRIBUTION

(B)......RECTANGULAR DISTRIBUTION

(C)......TRAPEZOIDAL DISTRIBUTION

NO SCALE

LATERAL ACTIVE PRESSURES FOR TEMPORARY SHORING

GEOCON INCORPORATED

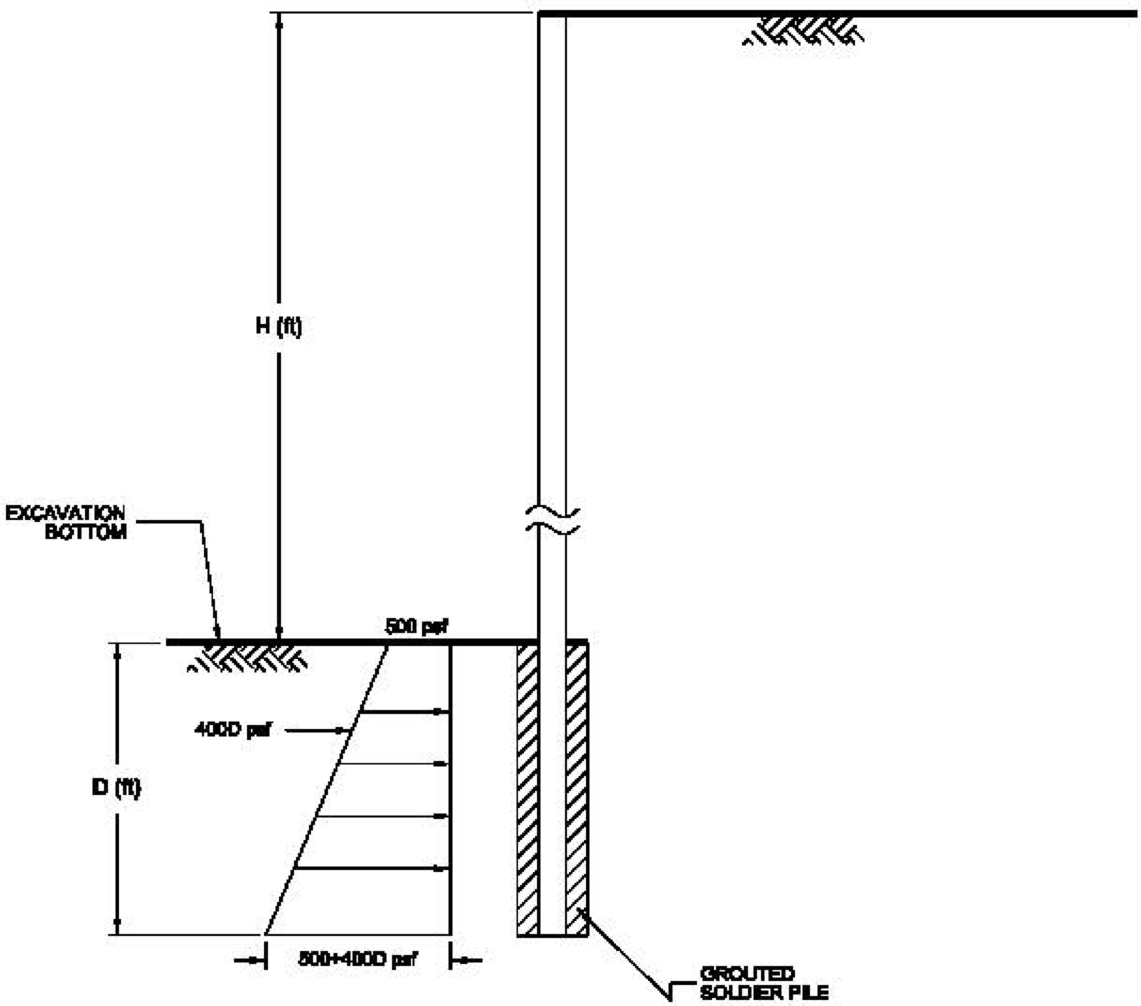
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974

PHONE 858 558-6900 - FAX 858 558-6159

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 06 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 9



NO SCALE

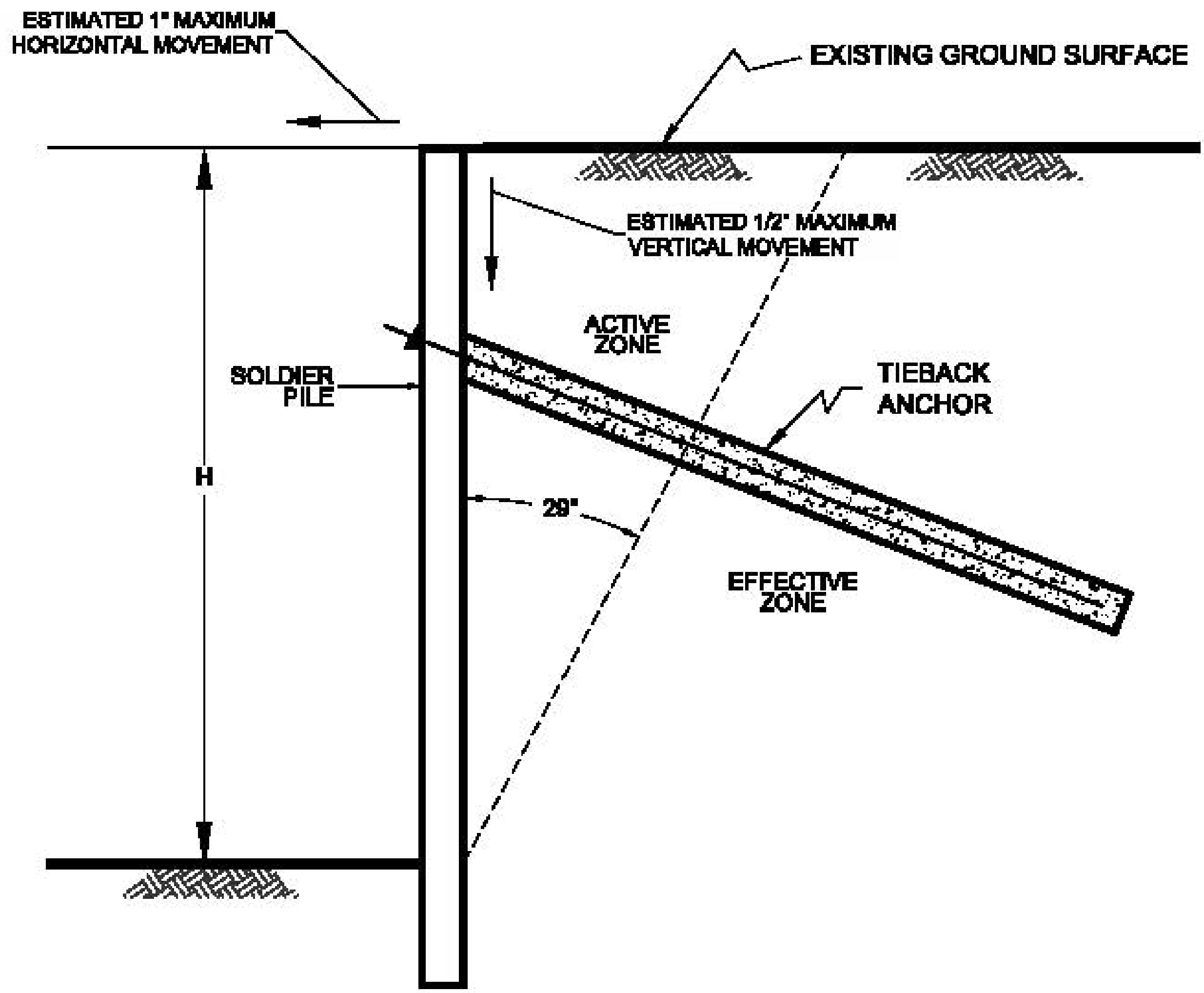
SOLDIER PILE PASSIVE PRESSURE DISTRIBUTION

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 06 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 10

Plotted:12/04/2017 4:42PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\DETAILS\Grouted Soldier Pile Passive Pressure (RGSPPD8).dwg



NOTE: NO ESTIMATED MOVEMENT AT EFFECTIVE ZONE

NO SCALE

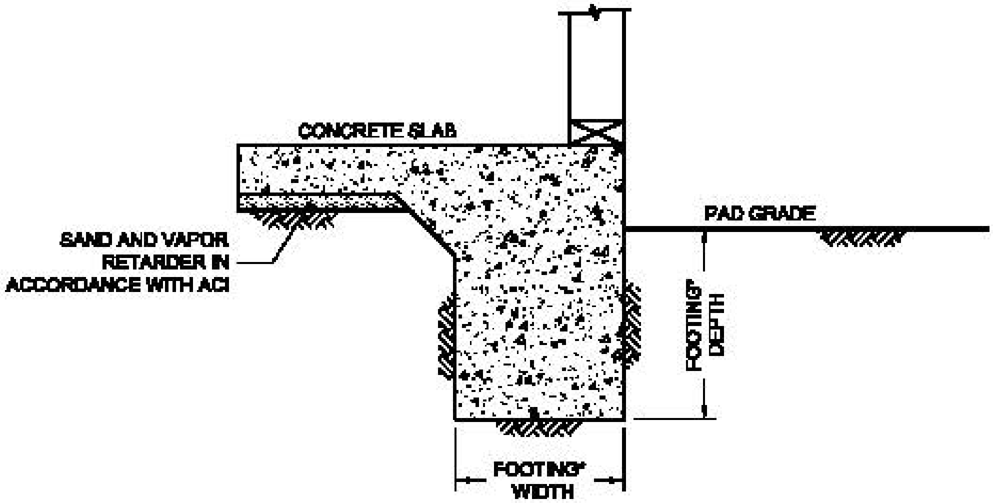
RECOMMENDED EFFECTIVE ZONE FOR TIEBACK ANCHORS

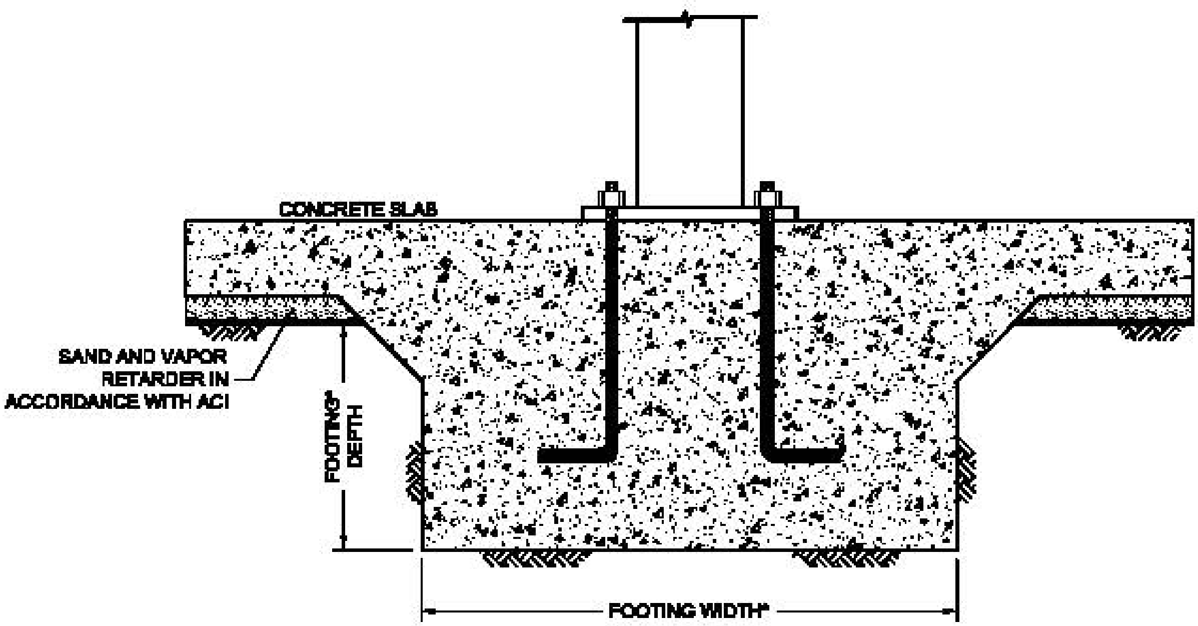
GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 06 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 11

Plotted:12/04/2017 4:44PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\DETAILS\Effective Zone For Tieback Anchors (REZTA8).dwg





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

NO SCALE

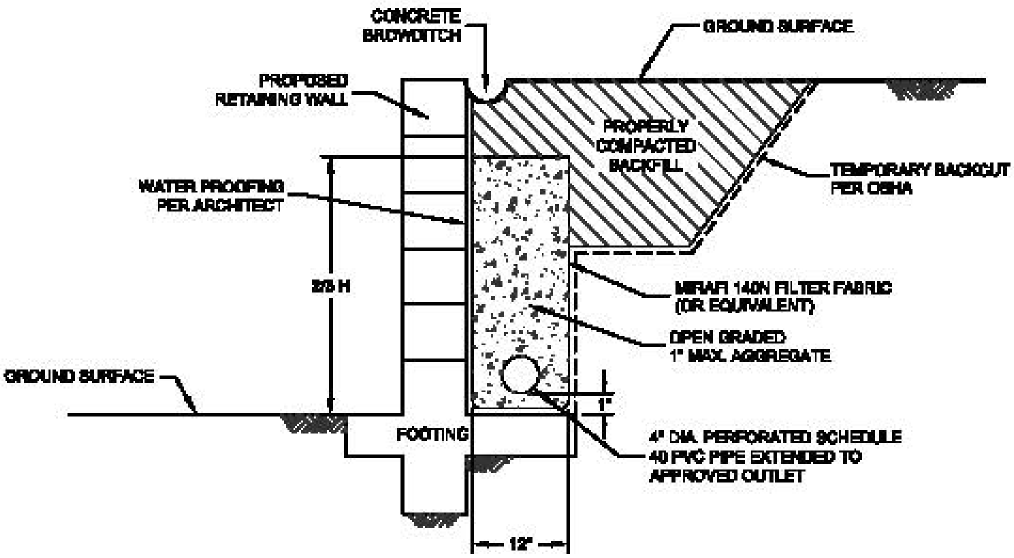
WALL / COLUMN FOOTING DIMENSION DETAIL

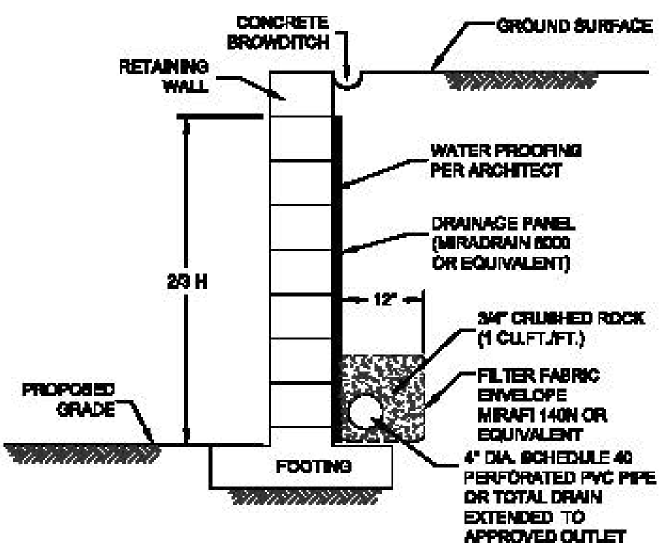
GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

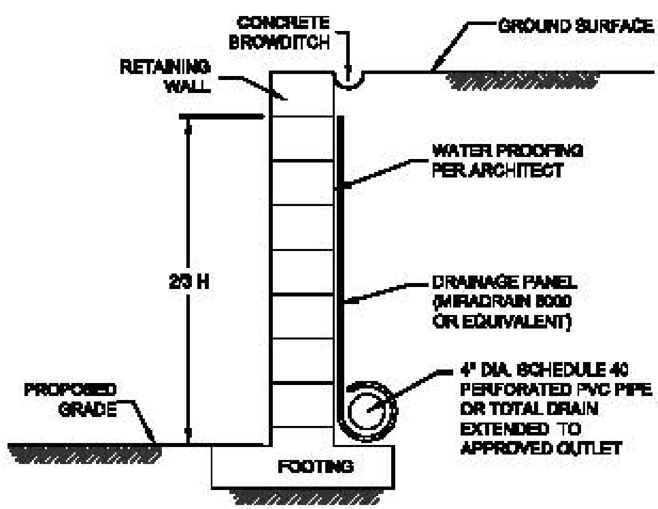
2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 06 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 12

Plotted:12/04/2017 4:45PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2093-52-01 2209 National Avenue\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

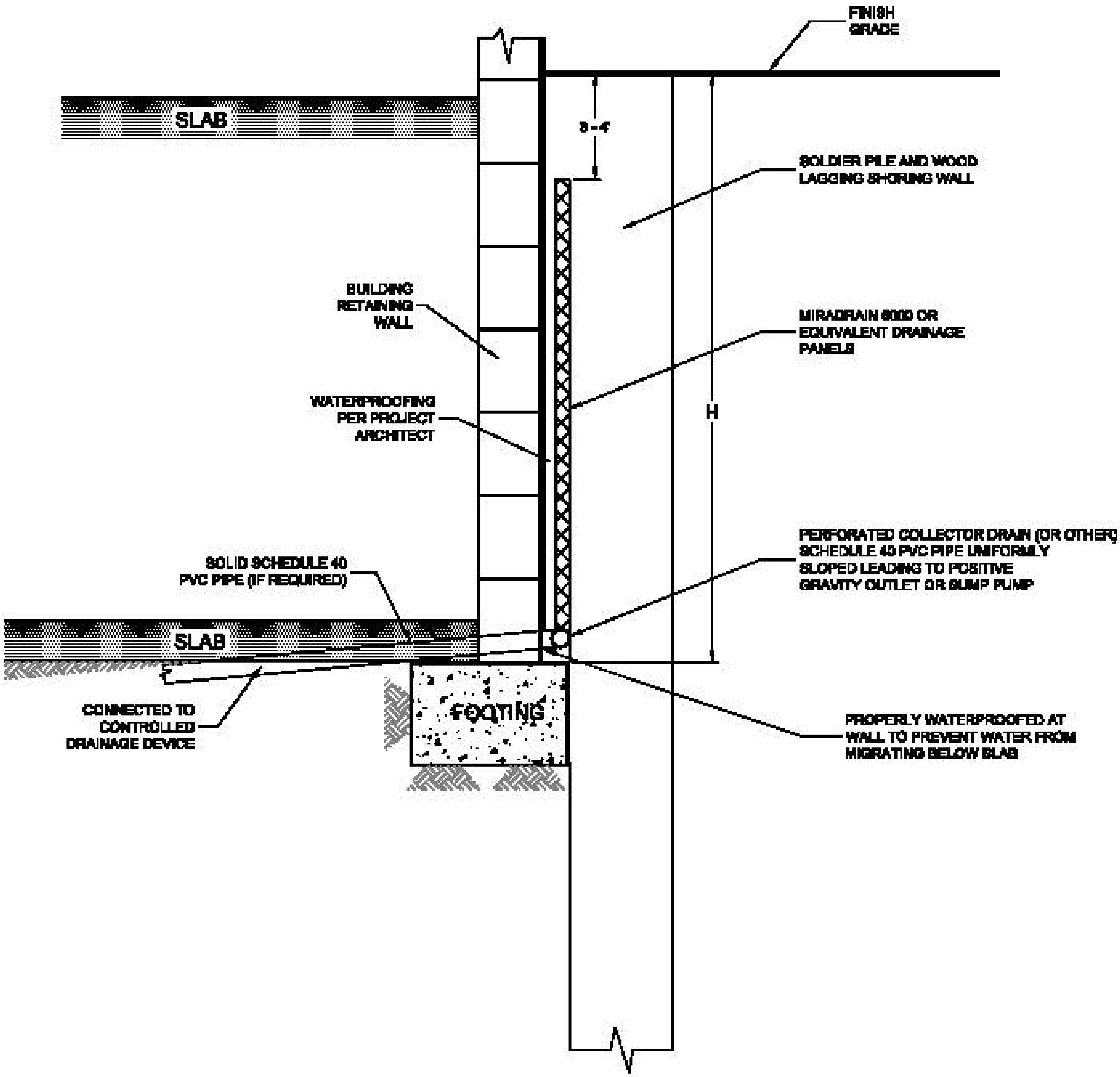
NO SCALE

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 06 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 13



NO SCALE

SOLDIER PILE WALL DRAINAGE DETAIL

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

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

ML / CW		DSK/GTYPD	DATE 12 - 06 - 2017	PROJECT NO. G2093 - 52 - 01	FIG. 14

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on February 22, 2017, that consisted of a visual site reconnaissance, drilling 6 exploratory borings and conducting 4 infiltration tests. The approximate locations of the borings and infiltration tests are shown on the Geologic Map, Figure 2.

The exploratory borings, performed by Pacific Drilling Company, were advanced to depths of 5 to 51½ feet using a Marl M-5 truck-mounted drill rig equipped with 6-inch diameter augers. We obtained samples during our subsurface exploration using a California split-spoon sampler. The sampler is composed of steel and are driven to obtain the soil samples. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 2.875 inches. Up to 18 rings are placed inside the sampler that is 2.4 inches in diameter and 1 inch in height. We obtained ring samples in moisture-tight containers at appropriate intervals and transported them to the laboratory for testing. We also obtained disturbed bulk soil samples from the borings for laboratory testing. The type of sample is noted on the exploratory boring logs.

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

We visually classified and logged the soil encountered in the excavations in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual Manual Procedure D 2488).

PROJECT NO. G2093-52-01

BORING B 1

ELEV. (MSL.) **60'** DATE COMPLETED **02-22-2017**

EQUIPMENT **MARL M-5** BY: **M. LOVE**

DEP H N FEE	SAMPLE NO	LITHOLOGY	GROUNDWATER	SO L CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3-INCH AC / 3-INCH BASE**			
				SC	**UNDOCUMENTED FILL (Qudf)** Reddish brown, moist, medium dense, Clayey, fine to medium SAND			
2				SM	**OLD PARALIC DEPOSITS (Qop)** Light reddish brown, moist, medium dense, Silty, fine SAND			
4								
6	B1 1					24	99.5	10.5
8								
10	B1 2			SC	Reddish brown and yellowish brown, moist, very dense, Clayey, fine to medium SAND; slight cementation	50/5"	116.1	13.5
12	B1 3							
14								
16	B1 4				Becomes light brown	50/3"	126.6	11.8
18								
20	B1 5					50/5.5"	101.4	24.7
22								
24								
26	B1 6			ML	Grayish brown, moist, very stiff, fine, Sandy SILT; slight mottling	35	101.0	25.9
	B1 7							
28								

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

G2093-52-01 GPJ

SAMPLE SYMBOLS		
SAMPL NG UNSUCCESSFUL	S ANDARD PENE RA ON ES	DR VE SAMPLE (UND S URBED)
D S URBED OR BAG SAMPLE	CHUNK SAMPLE	WA ER ABLE OR SEEPAGE

NO E HE LOG OF SUBSURFACE COND ONS SHOWN HEREON APPL ES ONLY A HE SPEC F C BOR NG OR RENCH LOCA ON AND A HE DA E ND CA ED S NO WARRAN ED O BE REPRESEN A VE OF SUBSURFACE COND ONS A O HER LOCA ONS AND MES

GEOCON

DEP H N FEE	SAMPLE NO	LITHOLOGY	GROUNDWATER	SO L CLASS (USCS)	BORING B 1 ELEV. (MSL.) 60' DATE COMPLETED 02-22-2017 EQUIPMENT MARL M-5 BY: M. LOVE	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B1 8			ML	Becomes laminated	30	83.8	38.5
32								
34								
36	B1 9					36	94.2	31.3
38								
40	B1 10			CL	Dark gray, moist, very stiff, CLAY; laminated, slight mottling	29	91.7	32.7
42								
44								
46	B1 11					40	86.2	36.4
48								
50	B1 12					32	90.6	33.3
					BORING TERMINATED AT 51.5 FEET No groundwater encountered			

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

G2093-52-01 GPJ

SAMPLE SYMBOLS			
	SAMPL NG UNSUCCESSFUL	S ANDARD PENE RA ON ES	DR VE SAMPLE (UND S URBED)
	D S URBED OR BAG SAMPLE	CHUNK SAMPLE	WA ER ABLE OR SEEPAGE

GEOCON

PROJECT NO. G2093-52-01

DEP H N FEE	SAMPLE NO	LITHOLOGY	GROUNDWATER	SO L CLASS (USCS)	BORING B 2 ELEV. (MSL.) 53' DATE COMPLETED 02-22-2017 EQUIPMENT MARL M-5 BY: M. LOVE	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**3-INCH AC / 4-INCH BASE**			
				SP	**OLD PARALIC DEPOSITS (Qop)** Reddish brown, damp, dense, fine to medium SAND; trace gravel			
2								
4								
	B2 1				Becomes very dense	30/5"	108.5	5.9
6								
8								
10	B2 2			SM	Light brown, damp, very dense, Silty, fine SAND; porous	50/5"	109.6	18.9
12								
14								
	B2 3				Becomes dense	71	112.8	8.9
16								
	B2 4							
18								
20	B2 5				Slight oxidation staining	50/4"	107.7	20.8
22								
24								
	B2 6			ML	Gray brown, moist, very stiff, fine Sandy SILT	44	101.4	25.1
26								
28								

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

G2093-52-01 GPJ

SAMPLE SYMBOLS			
	SAMPL NG UNSUCCESSFUL	S ANDARD PENE RA ON ES	DR VE SAMPLE (UND S URBED)
	D S URBED OR BAG SAMPLE	CHUNK SAMPLE	WA ER ABLE OR SEEPAGE

NO E HE LOG OF SUBSURFACE COND ONS SHOWN HEREON APPL ES ONLY A HE SPEC F C BOR NG OR RENCH LOCA ON AND A HE DA E ND CA ED
S NO WARRAN ED O BE REPRESEN A VE OF SUBSURFACE COND ONS A O HER LOCA ONS AND MES

GEOCON

BORING B 2

ELEV. (MSL.) **53'** DATE COMPLETED **02-22-2017**

EQUIPMENT **MARL M-5** BY: **M. LOVE**

DEP H N FEE	SAMPLE NO	LITHOLOGY	GROUNDWATER	SO L CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2 7			ML		38	95.9	28.5
32								
34								
36								
38								
40	B2 8			CL	Dark olive gray, moist, very stiff, CLAY; laminated, trace sand	42	89.7	32.9

BORING TERMINATED AT 41.5 FEET
No groundwater encountered

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

G2093-52-01 GPJ

SAMPLE SYMBOLS			
	SAMPL NG UNSUCCESSFUL	S ANDARD PENE RA ON ES	DR VE SAMPLE (UND S URBED)
	D S URBED OR BAG SAMPLE	CHUNK SAMPLE	WA ER ABLE OR SEEPAGE

GEOCON

BORING B 3 (OFF SITE)

ELEV. (MSL.) 50' DATE COMPLETED 02-22-2017

EQUIPMENT MARL M-5 BY: M. LOVE

DEP H N FEE	SAMPLE NO	LITHOLOGY	GROUNDWATER	SO L CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3-INCH AC / 3-INCH BASE**			
2				CL	**UNDOCUMENTED FILL (Qudf)** Dark reddish brown, moist, stiff, CLAY with gravel			
4	B3 1			SC	**OLD PARALIC DEPOSITS (Qop)** Reddish brown and yellowish brown, moist, Clayey, fine to medium SAND	50/5"	124.7	11.9
					BORING TERMINATED AT 5 FEET No groundwater encountered			

Figure A-3,
Log of Boring B 3 (OFF SITE), Page 1 of 1

G2093-52-01 GPJ

SAMPLE SYMBOLS			
	SAMPL NG UNSUCCESSFUL	S ANDARD PENE RA ON ES	DR VE SAMPLE (UND S URBED)
	D S URBED OR BAG SAMPLE	CHUNK SAMPLE	WA ER ABLE OR SEEPAGE

NO E HE LOG OF SUBSURFACE COND ONS SHOWN HEREON APPL ES ONLY A HE SPEC F C BOR NG OR RENCH LOCA ON AND A HE DA E ND CA ED S NO WARRAN ED O BE REPRESEN A VE OF SUBSURFACE COND ONS A O HER LOCA ONS AND MES

GEOCON

PROJECT NO. G2093-52-01

DEP H N FEE	SAMPLE NO	LITHOLOGY	GROUNDWATER	SO L CLASS (USCS)	BORING B 4 (OFF SITE) ELEV. (MSL.) 47' DATE COMPLETED 02-22-2017 EQUIPMENT MARL M-5 BY: M. LOVE	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**4-INCH AC / 5-INCH BASE**			
2				CL	**OLD PARALIC DEPOSITS (Qop)** Reddish brown, moist, stiff, CLAY; trace sand			
4	B4 1			SC	Reddish brown and yellowish brown, moist, very stiff, fine to medium grained, Clayey SAND	40	119.6	14.8
					BORING TERMINATED AT 5 FEET No groundwater encountered			

Figure A-4,
Log of Boring B 4 (OFF SITE), Page 1 of 1

G2093-52-01 GPJ

SAMPLE SYMBOLS			
	SAMPL NG UNSUCCESSFUL	S ANDARD PENE RA ON ES	DR VE SAMPLE (UND S URBED)
	D S URBED OR BAG SAMPLE	CHUNK SAMPLE	WA ER ABLE OR SEEPAGE

NO E HE LOG OF SUBSURFACE COND ONS SHOWN HEREON APPL ES ONLY A HE SPEC F C BOR NG OR RENCH LOCA ON AND A HE DA E ND CA ED S NO WARRAN ED O BE REPRESEN A VE OF SUBSURFACE COND ONS A O HER LOCA ONS AND MES

GEOCON

BORING B 5

ELEV. (MSL.) 54' DATE COMPLETED 02-22-2017

EQUIPMENT MARL M-5 BY: M. LOVE

DEP H N FEE	SAMPLE NO	LITHOLOGY	GROUNDWATER	SO L CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4-INCH AC / 5-INCH BASE**			
2				SM	**OLD PARALIC DEPOSITS (Qop)** Yellowish brown, damp, very dense, Silty, fine to medium SAND; trace gravel			
	B5 1					81	119.1	12.6
4				SP	Light reddish brown, damp, medium dense, fine to medium SAND			
	B5 2					19	112.8	5.4
6								
8	B5 3					26	114.8	7.0

BORING TERMINATED AT 8.5 FEET
No groundwater encountered

Figure A-5,
Log of Boring B 5, Page 1 of 1

G2093-52-01 GPJ

SAMPLE SYMBOLS	SAMPL NG UNSUCCESSFUL	S ANDARD PENE RA ON ES	DR VE SAMPLE (UND S URBED)
	D S URBED OR BAG SAMPLE	CHUNK SAMPLE	WA ER ABLE OR SEEPAGE

NO E HE LOG OF SUBSURFACE COND ONS SHOWN HEREON APPL ES ONLY A HE SPEC F C BOR NG OR RENCH LOCA ON AND A HE DA E ND CA ED S NO WARRAN ED O BE REPRESEN A VE OF SUBSURFACE COND ONS A O HER LOCA ONS AND MES

DEP H N FEE	SAMPLE NO	LITHOLOGY	GROUNDWATER	SO L CLASS (USCS)	BORING B 6 ELEV. (MSL.) 54' DATE COMPLETED 02-22-2017 EQUIPMENT MARL M-5 BY: M. LOVE	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**3-INCH AC / 5-INCH BASE**			
				SM	**OLD PARALIC DEPOSITS (Qop)** Light reddish brown, moist, very dense, Silty, fine to medium SAND; slight lamination			
2								
	B6 1					50/5"	118.1	6.6
4	B6 2							
	B6 3					50/4"	118.8	8.6
6					BORING TERMINATED AT 6 FEET No groundwater encountered			

Figure A-6,
Log of Boring B 6, Page 1 of 1

G2093-52-01 GPJ

SAMPLE SYMBOLS	SAMPL NG UNSUCCESSFUL	S ANDARD PENE RA ON ES	DR VE SAMPLE (UND S URBED)
	D S URBED OR BAG SAMPLE	CHUNK SAMPLE	WA ER ABLE OR SEEPAGE

NO E HE LOG OF SUBSURFACE COND ONS SHOWN HEREON APPL ES ONLY A HE SPEC F C BOR NG OR RENCH LOCA ON AND A HE DA E ND CA ED S NO WARRAN ED O BE REPRESEN A VE OF SUBSURFACE COND ONS A O HER LOCA ONS AND MES

GEOCON

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current and generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We selected samples to test for in-place density and moisture content, shear strength, expansion potential, water-soluble sulfate content, R-Value, gradation, and consolidation characteristics. The results of our laboratory tests are summarized on Tables B-I through B-V and Figures B-1 through B-5 and on the boring logs in Appendix A.

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

Sample No.	Dry Density (pcf)	Moisture Content (%)	Peak [Ultimate[1]] Cohesion (psf)	Peak [Ultimate[1]] Angle of Shear Resistance (degrees)
B1-2	116.1	13.5	34 [31]	950 [600]
B1-6	101.0	25.9	26 [26]	900 [650]

[1] Ultimate at end of test at 0.2-inch deflection.

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

Sample No.	Geologic Unit	Moisture Content (%)		Dry Density (pcf)	Expansion Index	ASTM Soil Expansion Classification	2016 CBC Expansion Classification
		Before Test	After Test				
B1-3	Qop	7.0	11.9	123.2	0	Very Low	Non-Expansive
B1-7	Qop	9.5	16.8	111.9	14	Very Low	Non-Expansive

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

Sample No.	Depth (Feet)	Description (Geologic Unit)	R-Value
B6-2	3-5	Light reddish brown, Silty SAND (Qop)	6

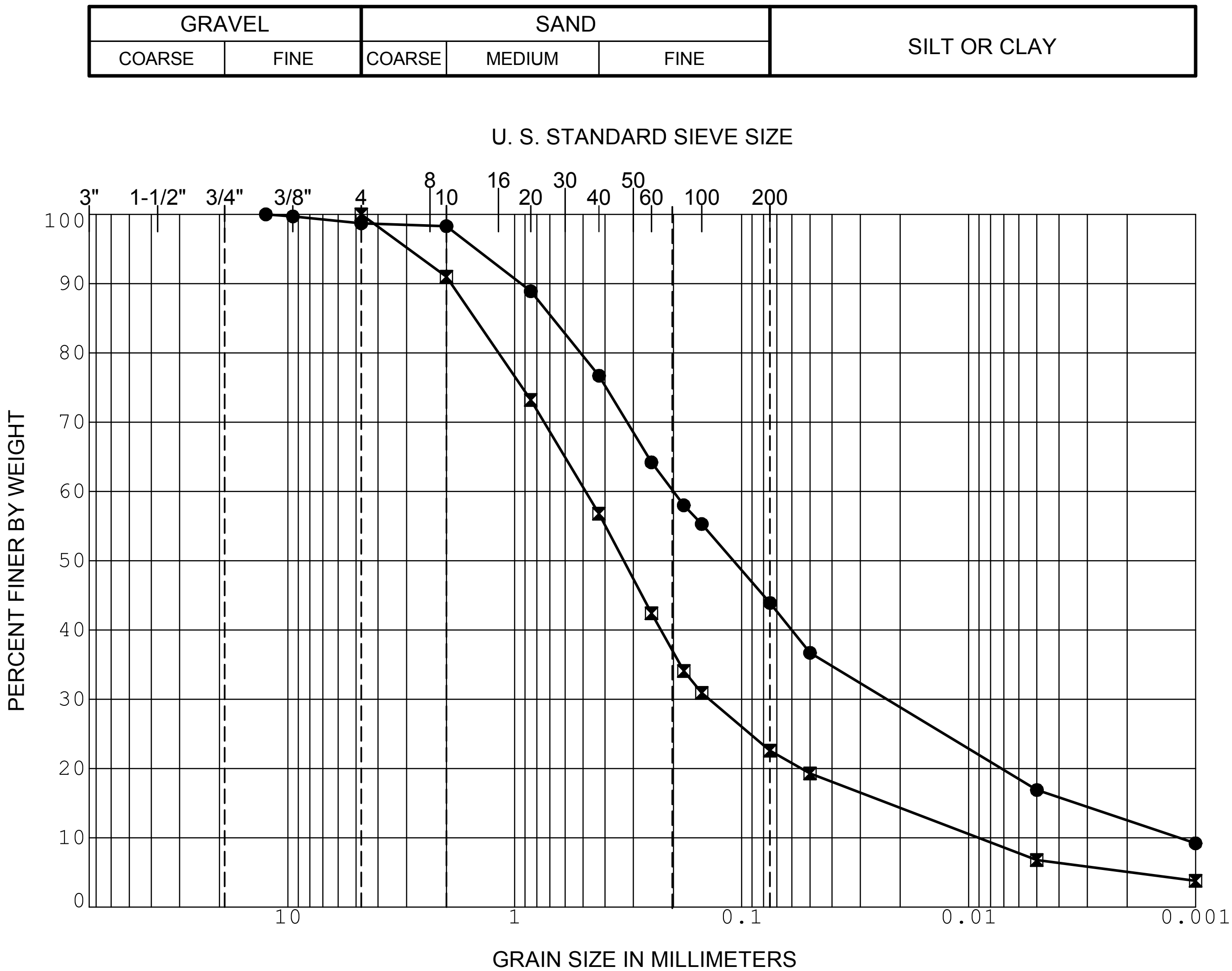
TABLE B-IV
SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (%)	ACI 318-14 Sulfate Class
B1-7	0.009	S0

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

Sample No.	Depth (feet)	Geologic Unit	Hand Penetrometer Reading, Unconfined Compression Strength (tsf)	Undrained Shear Strength (ksf)
B1-1	6	Qop	3.5	3.5
B1-4	16	Qop	4.5+	4.5+
B1-5	21	Qop	3.5	3.5
B1-9	36	Qop	4.0	4.0
B1-10	41	Qop	4.0	4.0
B1-11	46	Qop	3.5	3.5
B1-12	51	Qop	3.0	3.0
B2-1	5	Qop	4.5+	4.5+
B2-5	21	Qop	4.5+	4.5+
B2-6	26	Qop	4.5+	4.5+
B2-7	31	Qop	4.5+	4.5+
B2-8	41	Qop	4.5	4.5
B3-1	5	Qop	4.5+	4.5+
B5-1	3	Qop	4.5+	4.5+
B5-2	6	Qop	3.0	3.0
B5-3	8	Qop	4.0	4.0
B6-1	3	Qop	4.5+	4.5+

PROJECT NO. G2093-52-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B4 1	4.0	SC C ayey SAND				
⊠	B6 3	5.0	SM S ty SAND				
▲							

GRADATION CURVE

U STOR IT BURRIO LOGAN

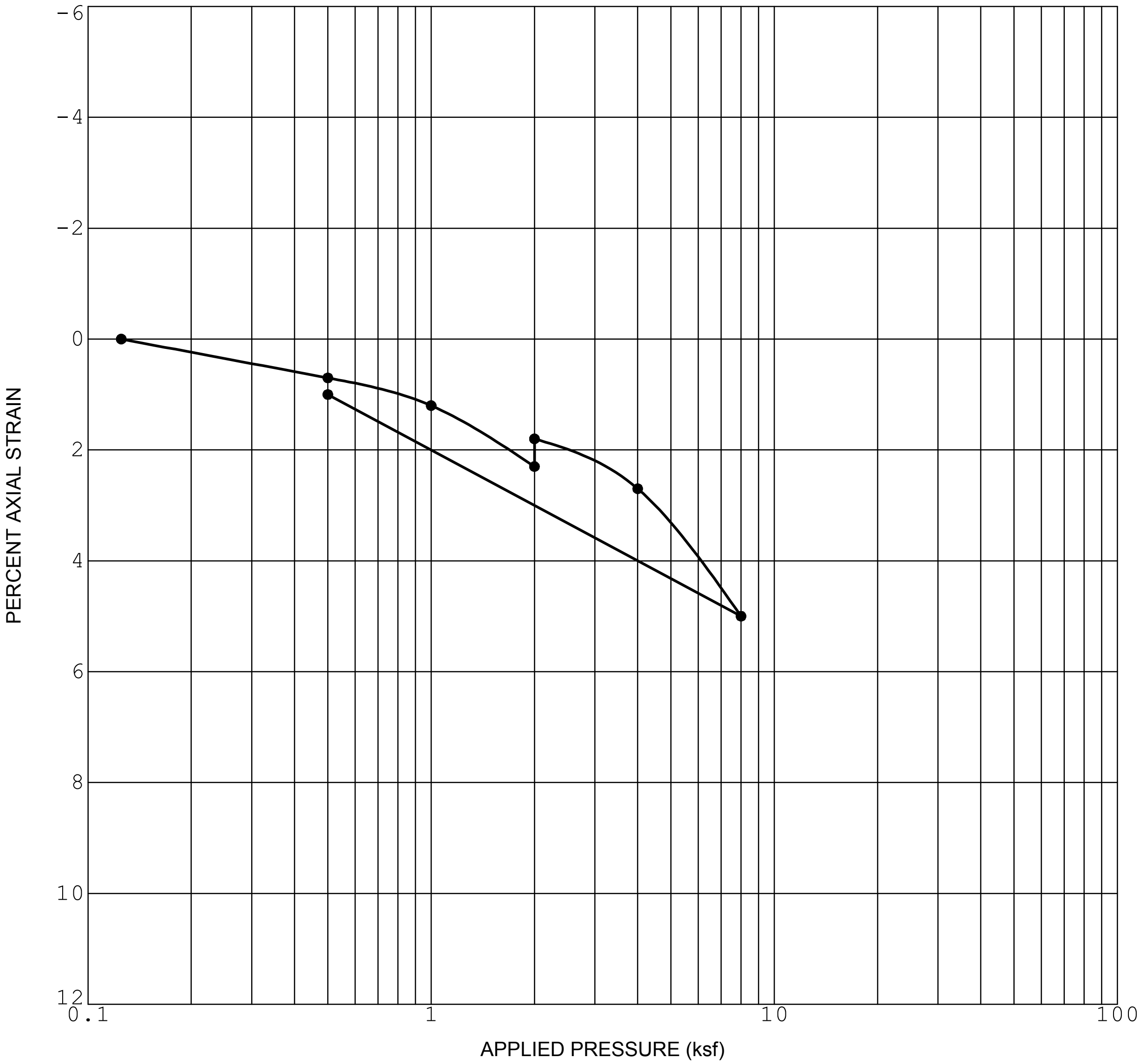
SAN DIEGO, CALIFORNIA

G2093-52-01.GPJ

Figure B-1

GEOCON

SAMPLE NO. B1-8



Initial Dry Density (pcf)	83.8		**Initial Saturation (%)**	100+
Initial Water Content (%)	38.5		**Sample Saturated at (ksf)**	2.0

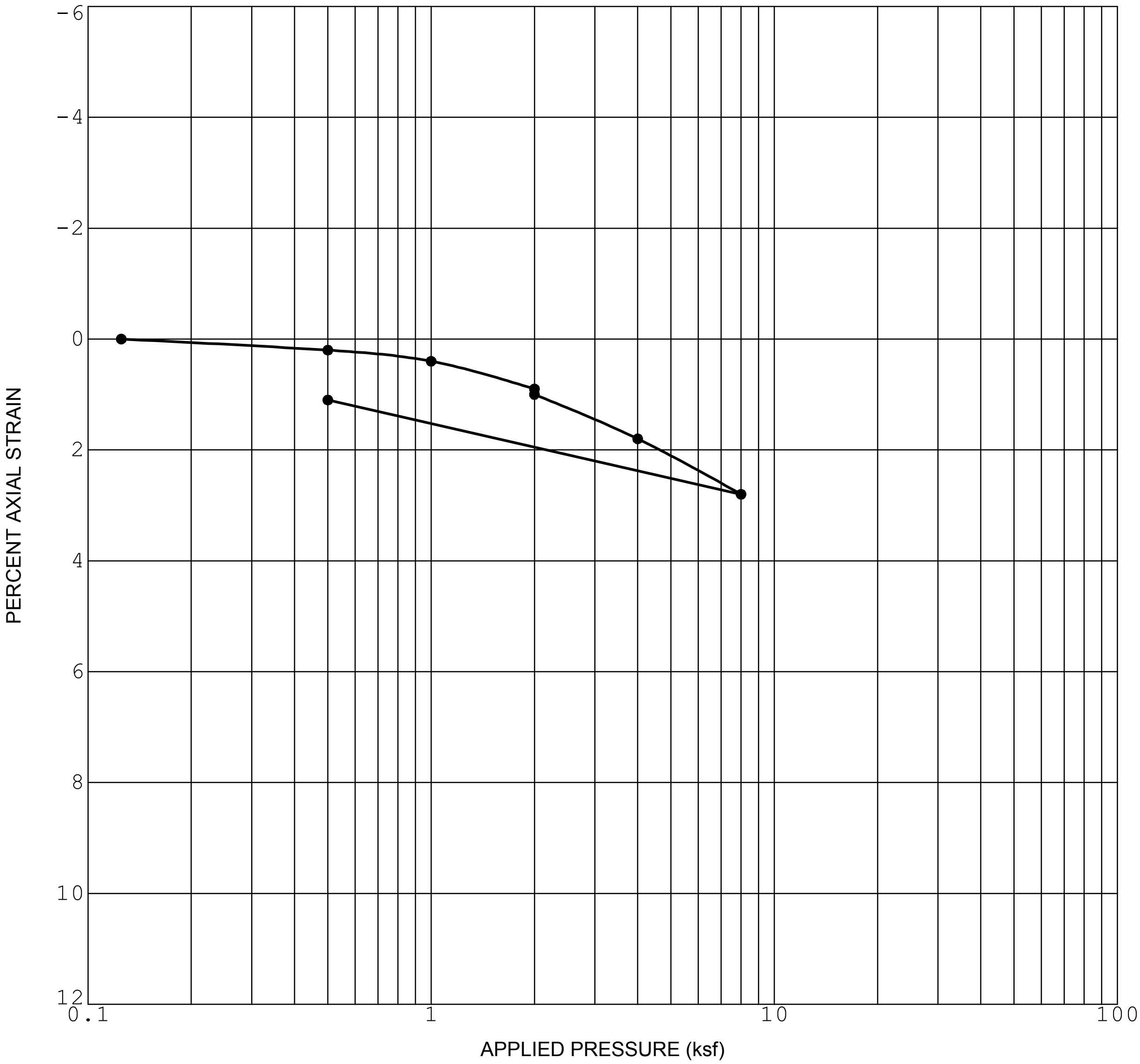
CONSOLIDATION CURVE

U STOR IT BURRIO LOGAN

SAN DIEGO, CALIFORNIA

PROJECT NO. G2093-52-01

SAMPLE NO. B2-2



Initial Dry Density (pcf)	109.6		**Initial Saturation (%)**	97.5
Initial Water Content (%)	18.9		**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

U STOR IT BURRIO LOGAN

SAN DIEGO, CALIFORNIA

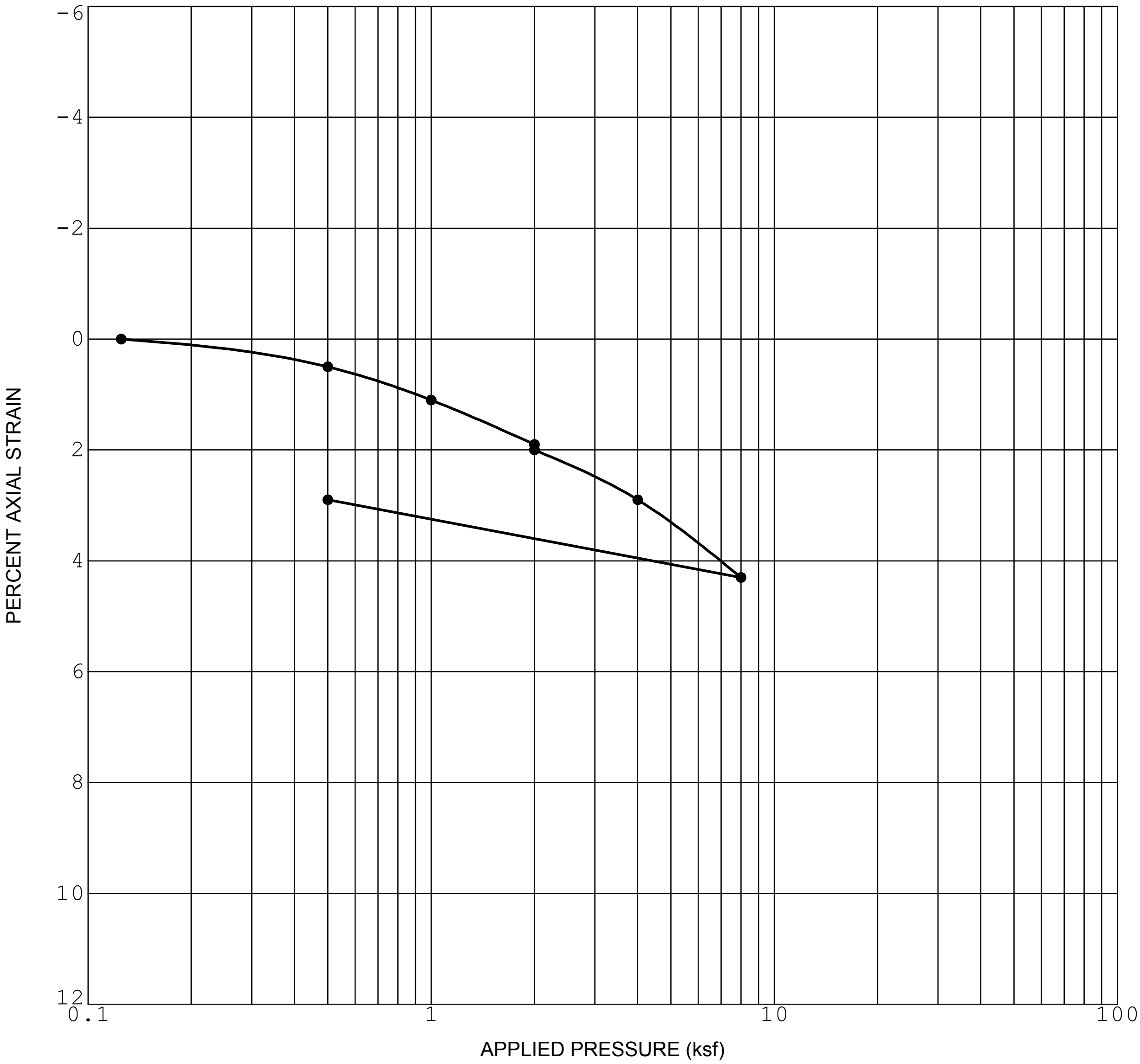
G2093-52-01.GPJ

Figure B-3

GEOCON

PROJECT NO. G2093-52-01

SAMPLE NO. B4-1



Initial Dry Density (pcf)	119.6		**Initial Saturation (%)**	100+
Initial Water Content (%)	14.8		**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

U STOR IT BURRIO LOGAN

SAN DIEGO, CALIFORNIA

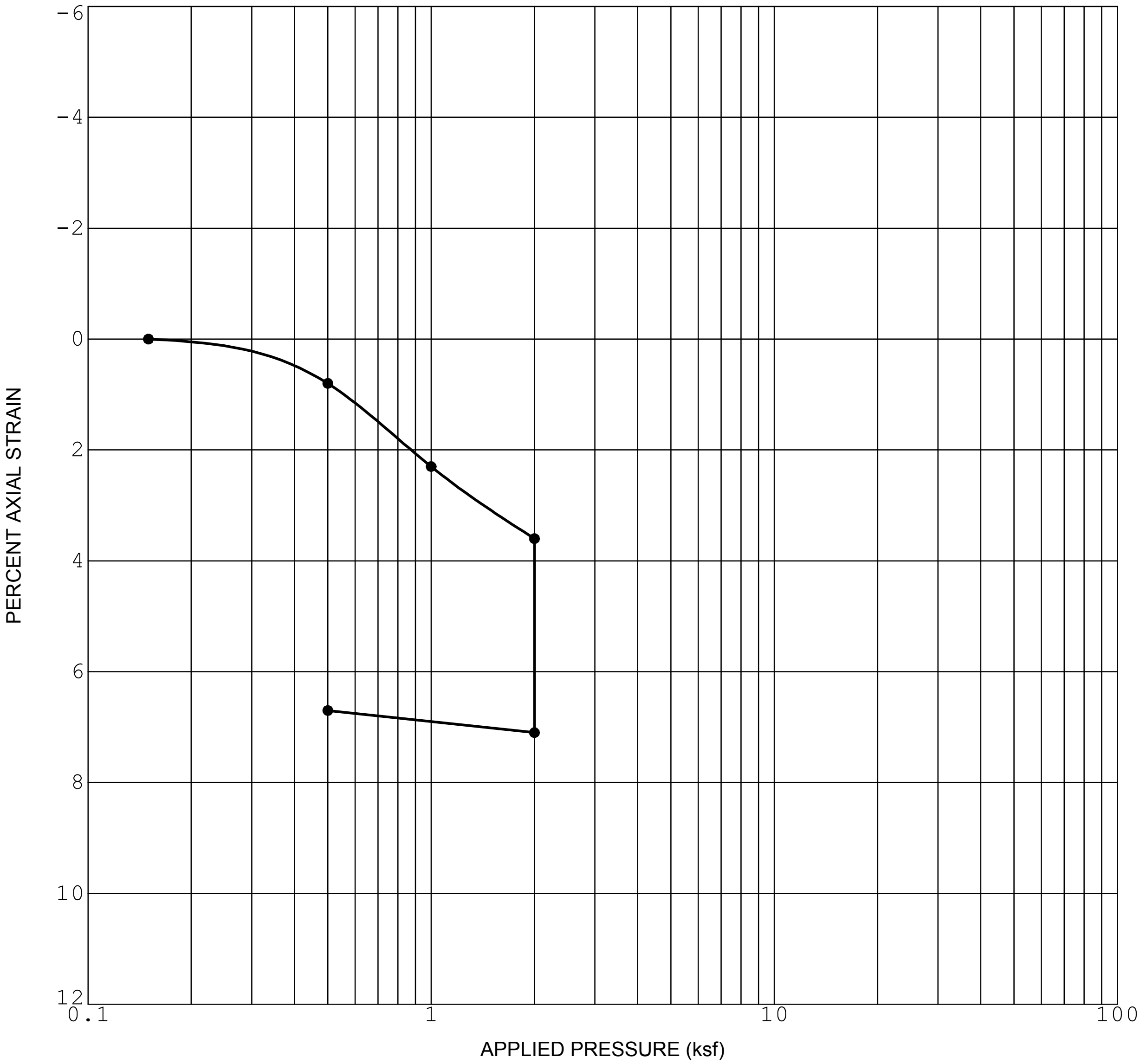
G2093-52-01.GPJ

Figure B-4

GEOCON

PROJECT NO. G2093-52-01

SAMPLE NO. B6-3



Initial Dry Density (pcf)	118.8	**Initial Saturation (%)**	57.4
Initial Water Content (%)	8.6	**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

U STOR IT BURRIO LOGAN

SAN DIEGO, CALIFORNIA

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

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 hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

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

TABLE C-I
HYDROLOGIC SOIL GROUP DEFINITIONS

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

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

In Situ Testing

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

TABLE C-II
SOIL PERMEABILITY DEFINITIONS

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

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

We performed 2 Aardvark Permeameter tests at the locations shown on the attached Geologic Map, Figure 2. The test borings were 6-inches in diameter. The results of the tests provide parameters regarding the saturated hydraulic conductivity and infiltration characteristics of on-site soil and geologic units. Table C-III presents the results of the estimated field saturated hydraulic conductivity and estimated infiltration rates obtained from the Aardvark Permeameter tests. The field sheets are also attached herein. The designer of storm water devices should apply an appropriate factor of safety. Soil infiltration rates from in-situ tests can vary significantly from one location to another due to the heterogeneous characteristics inherent to most soil. Based on a discussion in the County of Riverside *Design Handbook for Low Impact Development Best Management Practices*, the infiltration rate should be considered equal to the saturated hydraulic conductivity rate.

TABLE C-III
FIELD PERMEAMETER INFILTRATION TEST RESULTS

Test No. [1]	Geologic Unit	Test Elevation (feet MSL)	Field-Saturated Infiltration Rate (inch/hour)	Worksheet Infiltration Rate[2] (inch/hour)
P-3	Qop	49	0.024	0.012
P-4	Qop	50	0.002	0.001
		Average:	0.013	0.007

1 Infiltration tests P-1 and P-2 were performed outside of the project limits and have not been taken into consideration for this assessment. The field sheets for tests P-1 and P-2 are included herein for reference only.
1 Using a Factor of Safety of 2.0.

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

TABLE C-IV
INFILTRATION CATEGORIES

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

*Using a Factor of Safety of 2.

STORM WATER MANAGEMENT CONCLUSIONS

The Geologic Map, Figure 2, depicts the existing property, the approximate lateral limits of the geologic units, the locations of the field excavations and the in-situ infiltration test locations. The following presents a discussion of the soil types on site regarding storm water infiltration feasibility.

Soil Types

Undocumented Fill (Qudf) – Undocumented fill is present across the site. The undocumented fill was not tested or observed during placement and should be considered highly variable. Water that is allowed to migrate within the undocumented fill soil cannot be controlled due to lateral migration potential, would destabilize support for the existing improvements, and would shrink and swell. Therefore, full and partial infiltration should be considered infeasible within the undocumented fill. We anticipate that the undocumented fill will be completely removed during excavations for the proposed subterranean levels.

Old Paralic Deposits – The surficial soils on the property are underlain by Old Paralic Deposits. Based on the boring logs, laboratory tests and our observations, the Old Paralic Deposits are highly variable due to the sedimentary nature of the materials. The Old Paralic Deposits have a greater propensity for lateral water migration over vertical water migration. The infiltration rates within the Old Paralic Deposits are considered to be very low due to the dense nature of the materials. In addition, the Old Paralic Deposits possess hydroconsolidation potential as discussed herein. As a result, full and partial infiltration should be considered infeasible.

Compacted Fill – We expect that compacted fill, if any, will be comprised of on-site materials that will consist predominantly of silty and clayey sand. The fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density. In our experience, compacted fill using the on-site materials does not possess infiltration rates appropriate with infiltration and the water would destabilize the existing fill causing distress to existing and proposed improvements. The intent of the compacted fill is to support structures and infrastructure (utilities, pavement, and flatwork). Therefore, full and partial infiltration should be considered infeasible.

Infiltration Rates

The results of the infiltration rates within the Old Paralic Deposits ranges from 0.002 to 0.024 inches per hour with an average of 0.013 inches per hour (average of 0.007 inches per hour including a factor of safety of 2.0). Therefore, based on the results of the field infiltration tests, the laboratory tests and our experience, full and partial infiltration should be considered infeasible within the Old Paralic Deposits. Mitigation for very low infiltration rates does not exist.

Groundwater Elevations

We did not encounter groundwater during the drilling operations at the property to the maximum depth of 50 feet or an elevation of about 10 feet MSL. We expect groundwater is present at an elevation of 0 to 5 feet MSL. The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. Therefore, infiltration would be considered feasible above an elevation of 15 feet MSL.

New or Existing Utilities

Utilities are located adjacent to the property on the northern, western, and southern property boundaries and existing streets. Therefore, full infiltration near these utilities should be considered infeasible within these areas. The setback for infiltration devices would be a minimum of a 1:1 plane from 5 feet outside the invert of the deepest adjacent utility. Mitigation measures to prevent water from infiltrating the utilities consist of installing cutoff walls around the utilities and installing subdrains and/or installing liners. Liners would be the preferred option because of the potential for lateral migration within the Old Paralic Deposits.

Soil or Groundwater Contamination

We are unaware of contaminated soil or groundwater on the property. Therefore, infiltration associated with this risk is considered feasible. We should be contacted if contaminated soil exists on the property.

Slopes and Other Geologic Hazards

Slopes are not currently planned or exist on the property that would be affected by potential infiltration locations. As discussed herein, the Old Paralic Deposits possess a hydroconsolidation potential ranging from 0.1 to 3.5 percent. We expect the upper 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation up to about 4¼ inches. Therefore, infiltration in regards the geologic hazards would be considered infeasible.

Existing and Planned Structures

Existing structures are located along the western, eastern and southern property lines. If water is allowed to infiltrate into the soil, the water could migrate laterally and into other properties in the vicinity of the subject site and negatively affect other buildings and improvements in the area (e.g. saturating soil adjacent to existing foundations). Therefore, infiltration near these structures or any other proposed structures should be considered infeasible within these areas, and setbacks for infiltration should be incorporated. Mitigation for existing structures consists of not allowing water infiltration within a 1:1 plane from 20 feet below the existing foundations.

Storm Water Management Devices

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

Storm Water Standard Worksheets

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

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

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

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

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

TABLE C-VI
FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A[1]

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

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

Categorization of Infiltration Feasibility Condition			Worksheet C.4-1
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: We encountered field infiltration rates of: P-3: 0.024 inches/hour (0.012 with a FOS of 2.0) P-4: 0.002 inches/hour (0.001 with a FOS of 2.0) These tests results in an average of about 0.013 inches/hour (0.007 with a FOS of 2.0). The results of the infiltration tests indicate rates of less than 0.5 inches per hour (including the factor of safety); therefore, infiltration is not considered feasible. 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: Undocumented fill and Old Paralic Deposits underlie the property. Water that would be allowed to infiltrate would migrate laterally outside of the property limits to the existing right-of-ways (located to the south) and toward existing and proposed structures (located to the north and west). The Old Paralic Deposits possess hydroconsolidation potential ranging from 0.1 to 3 percent. We expect the upper 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation is up to about 4¼ inches. Therefore, infiltration in regards the geologic hazards would be considered infeasible. Liners and subdrains should be incorporated into the design and construction of the planned storm water devices to prevent saturation and potential hydroconsolidation of the soil supporting the existing or proposed development. 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:

We did not encounter groundwater during the drilling operation at the property to the maximum depth of 50 feet or an elevation of 10 feet MSL. Groundwater is anticipated to be present at an elevation of 0 to 5 feet MSL. The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. Therefore, infiltration due to groundwater elevations would be considered feasible above an elevation of 15 feet MSL.

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 full infiltration would cause water balance issues including change of ephemeral streams or discharge of contaminated water 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 encountered field infiltration rates of: P-3: 0.024 inches/hour (0.012 with a FOS of 2.0) P-4: 0.002 inches/hour (0.001 with a FOS of 2.0) These tests results in an average of about 0.013 inches/hour (0.007 with a FOS of 2.0). The results of the infiltration tests indicate rates of less than 0.05 inches per hour (including the factor of safety); therefore, infiltration is not considered feasible. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
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: Undocumented fill and Old Paralic Deposits underlie the property. Water that would be allowed to infiltrate could migrate laterally outside of the property limits to the existing right-of-ways (located to the south) and toward existing and proposed structures (located to the north and west). The Old Paralic Deposits possess hydroconsolidation potential ranging from 0.1 to 3 percent. We expect the upper 10 feet of the Old Paralic Deposits may possess the hydroconsolidation potential and the resulting amount of potential settlement due to hydroconsolidation is up to about 4¼ inches. Therefore, infiltration in regards the geologic hazards would be considered infeasible. Liners and subdrains should be incorporated into the design and construction of the planned storm water devices to prevent saturation and potential hydroconsolidation of the soil supporting the existing or proposed development. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not encounter groundwater during the drilling operation at the property to the maximum depth of 50 feet or an elevation of 10 feet MSL. Groundwater is anticipated to be present at an elevation of 0 to 5 feet MSL. The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. Therefore, infiltration due to groundwater elevations would be considered feasible above an elevation of 15 feet MSL. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not provide a study regarding water rights. However, these rights are not typical in the San Diego County area. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	No Infiltration	

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

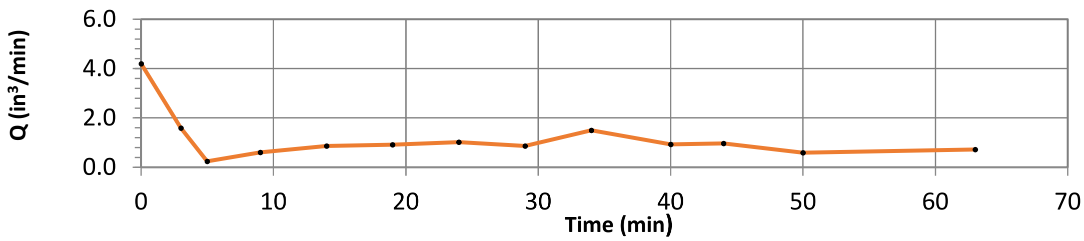
GEOCON

Aardvark Permeameter Data Analysis

Project Name:	2209 National Ave.	Date:	2/22/2017
Project Number:	G2093-52-01	By:	JML
Test Number:	P-3	Ref. EL (feet, MSL):	54.0
		Bottom EL (feet, MSL):	49.1

Borehole Diameter, d (in.):	6.00
Borehole Depth, **H** (in):	59.00
Distance Between Reservoir & Top of Borehole (in.):	29.50
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No
Distance Between Resevoir and APM Float, **D** (in.):	80.25
Head Height Calculated, **h** (in.):	4.77
Head Height Measured, **h** (in.):	4.00
Distance Between Constant Head and Water Table, **L** (in.):	1145.00

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	3.00	0.455	12.60	4.200
3	2.00	0.115	3.18	1.592
4	4.00	0.035	0.97	0.242
5	5.00	0.110	3.05	0.609
6	5.00	0.155	4.29	0.858
7	5.00	0.165	4.57	0.914
8	5.00	0.185	5.12	1.025
9	5.00	0.155	4.29	0.858
10	6.00	0.325	9.00	1.500
11	4.00	0.135	3.74	0.935
12	6.00	0.210	5.82	0.969
13	13.00	0.275	7.62	0.586
14	17.00	0.440	12.18	0.717
			Steady Flow Rate, Q (in^3/min):	0.757



Soil Matric Flux Potential, Φ_m

Φ_m= 0.016 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 4.03E-04 in/min | 0.024 in/hr

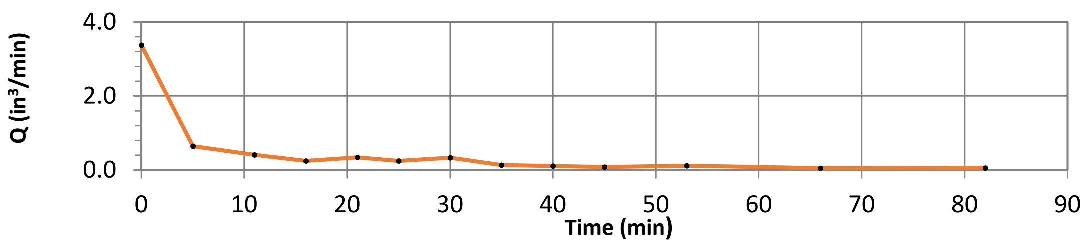
GEOCON

Aardvark Permeameter Data Analysis

Project Name:	2209 National Ave.	Date:	2/22/2017
Project Number:	G2093-52-01	By:	JML
Test Number:	P-4	Ref. EL (feet, MSL):	54.0
		Bottom EL (feet, MSL):	50.0

Borehole Diameter, d (in.):	6.00
Borehole Depth, **H** (in):	48.00
Distance Between Reservoir & Top of Borehole (in.):	29.00
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No
Distance Between Resevoir and APM Float, **D** (in.):	68.75
Head Height Calculated, **h** (in.):	4.73
Head Height Measured, **h** (in.):	4.50
Distance Between Constant Head and Water Table, **L** (in.):	1156.50

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.610	16.89	3.378
3	6.00	0.140	3.88	0.646
4	5.00	0.075	2.08	0.415
5	5.00	0.045	1.25	0.249
6	4.00	0.050	1.38	0.346
7	5.00	0.045	1.25	0.249
8	5.00	0.060	1.66	0.332
9	5.00	0.025	0.69	0.138
10	5.00	0.020	0.55	0.111
11	8.00	0.025	0.69	0.087
12	13.00	0.055	1.52	0.117
13	16.00	0.030	0.83	0.052
14	25.00	0.050	1.38	0.055
			Steady Flow Rate, Q (in^3/min):	0.055



Soil Matric Flux Potential, Φ_m

Φ_m= 0.001098419 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.79E-05 in/min 0.002 in/hr

GEOCON

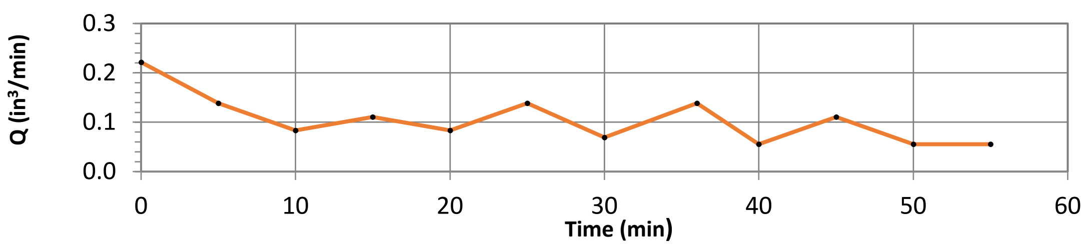
Aardvark Permeameter Data Analysis

Project Name:	2209 National Ave.	Date:	2/22/2017
Project Number:	G2093-52-01	By:	JML
Test Number:	P-1	Ref. EL (feet, MSL):	50.0
		Bottom EL (feet, MSL):	45.4

Borehole Diameter, d (in.):	8.00
Borehole Depth, **H** (in):	55.00
Distance Between Reservoir & Top of Borehole (in.):	30.50
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	77.25
Head Height Calculated, **h** (in.):	4.76
Head Height Measured, **h** (in.):	5.25
Distance Between Constant Head and Water Table, **L** (in.):	1150.25

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.040	1.11	0.222
3	5.00	0.025	0.69	0.138
4	5.00	0.015	0.42	0.083
5	5.00	0.020	0.55	0.111
6	5.00	0.015	0.42	0.083
7	5.00	0.025	0.69	0.138
8	6.00	0.015	0.42	0.069
9	4.00	0.020	0.55	0.138
10	5.00	0.010	0.28	0.055
11	5.00	0.020	0.55	0.111
12	5.00	0.010	0.28	0.055
13	5.00	0.010	0.28	0.055
			Steady Flow Rate, Q (in^3/min):	0.055



Soil Matric Flux Potential, Φ_m

Φ_m= 0.00101035 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.57E-05 in/min 0.002 in/hr

GEOCON

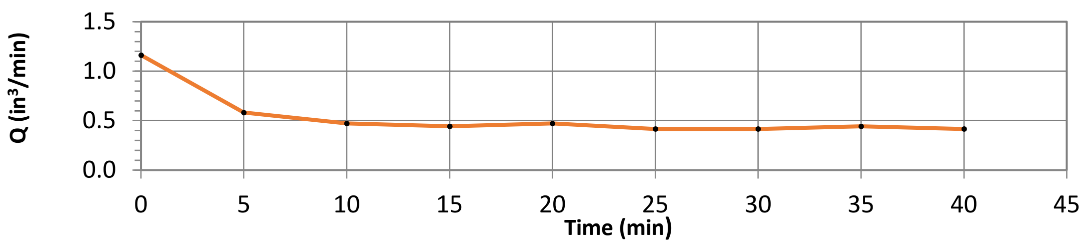
Aardvark Permeameter Data Analysis

Project Name:	2209 National Ave.	Date:	2/22/2017
Project Number:	G2093-52-01	By:	JML
Test Number:	(Offsite) P-2	Ref. EL (feet, MSL):	47.0
		Bottom EL (feet, MSL):	42.8

Borehole Diameter, d (in.):	6.00
Borehole Depth, **H** (in):	50.00
Distance Between Reservoir & Top of Borehole (in.):	29.00
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	70.75
Head Height Calculated, **h** (in.):	4.74
Head Height Measured, **h** (in.):	4.00
Distance Between Constant Head and Water Table, **L** (in.):	1154.00

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.210	5.82	1.163
3	5.00	0.105	2.91	0.582
4	5.00	0.085	2.35	0.471
5	5.00	0.080	2.22	0.443
6	5.00	0.085	2.35	0.471
7	5.00	0.075	2.08	0.415
8	5.00	0.075	2.08	0.415
9	5.00	0.080	2.22	0.443
10	5.00	0.075	2.08	0.415
11	6.00	0.080	2.22	0.369
12	4.00	0.060	1.66	0.415
			Steady Flow Rate, Q (in^3/min):	0.415



Soil Matric Flux Potential, Φ_m

Φ_m= 0.009 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.21E-04 in/min 0.013 in/hr

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

2209 NATIONAL AVENUE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2093-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

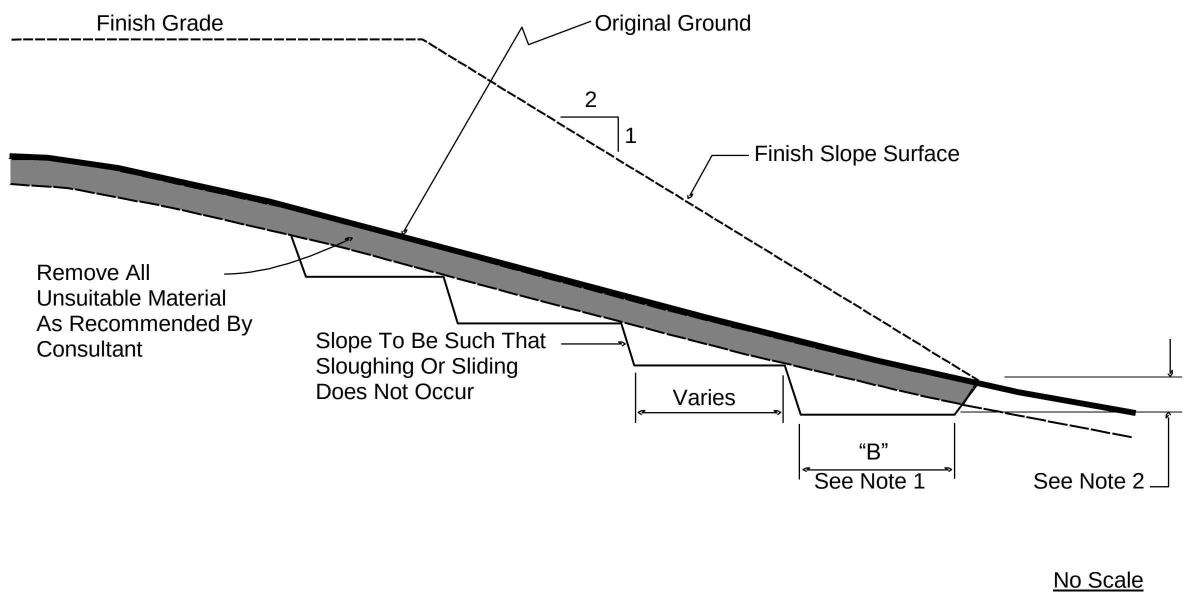
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

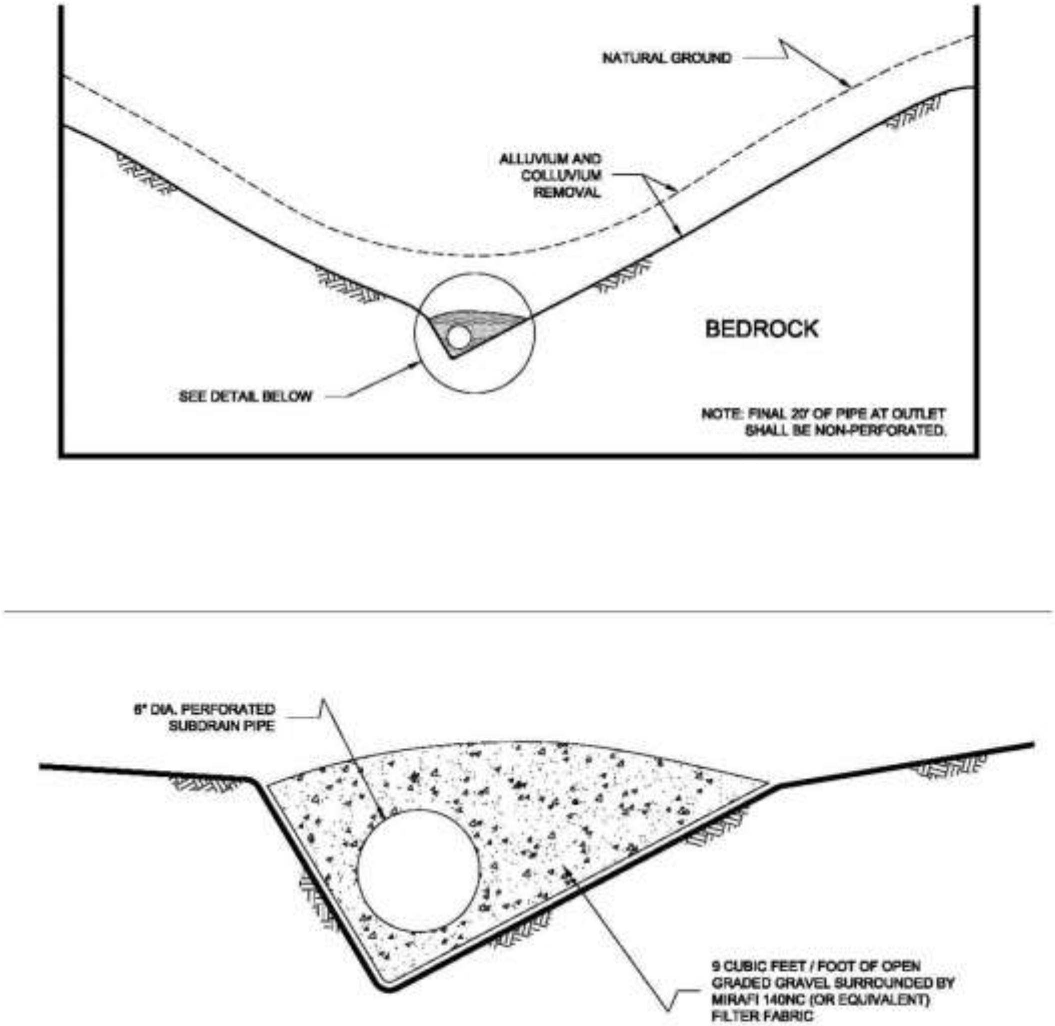
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

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

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL



NOTES:

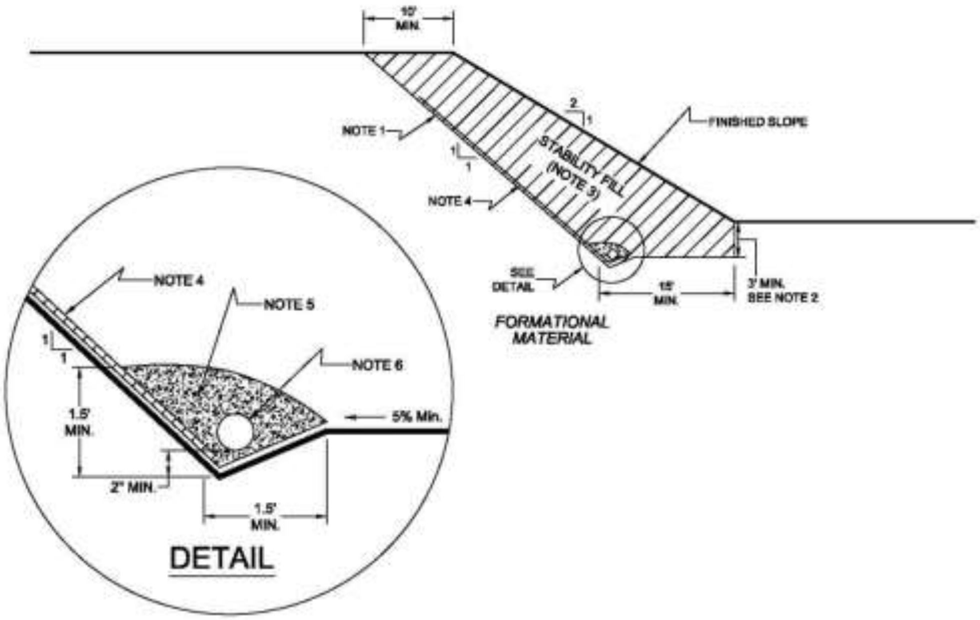
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

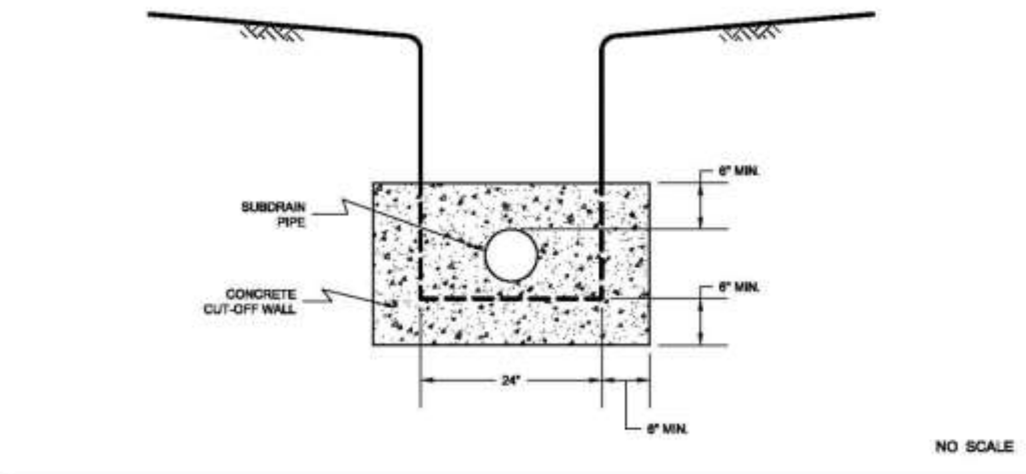
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

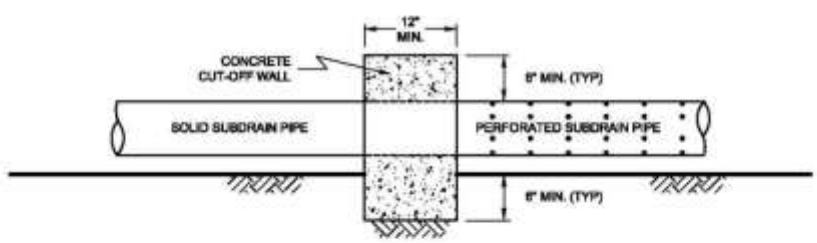
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



SIDE VIEW

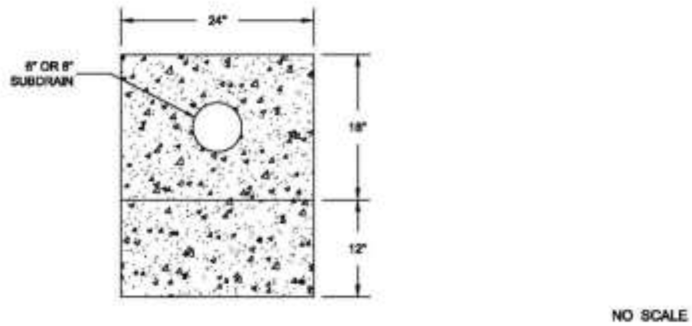


NO SCALE

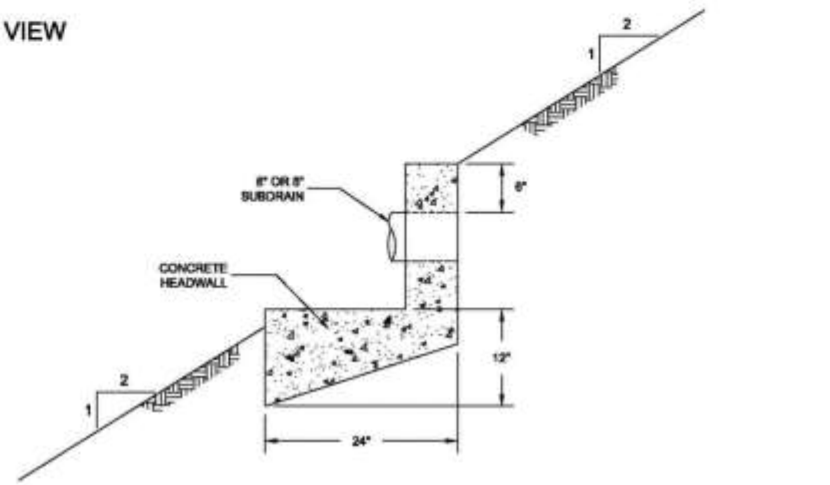
7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL

FRONT VIEW



SIDE VIEW



NOTE: HEADWALL SHOULD OUTLET AT TOE OF FILL SLOPE OR INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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

LIST OF REFERENCES

1. *2016 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2015 International Building Code*, prepared by California Building Standards Commission, dated July 1, 2016.

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

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

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

5. Bryant, W. A., and E. W. Hart, 2007, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps,* California Division of Mines and Geology Special Publication 42, interim revision.

6. California Department of Water Resources, Water Data Library. http://www.water.ca.gov/waterdatalibrary.

7. California Division of Mines and Geology, *Landslide Hazards in the Southern Part of the San Diego Metropolitan Area, Point Loma Quadrangle (Plate D), San Diego County, California*, dated 1995.

8. California Emergency Management Agency (CEMA, CGS and USC), *Tsunami Inundation Map for Emergency Planning, Point Loma Quadrangle,* dated June 1, 2009.

9. California Geological Survey, 2003, *State of California Earthquake Fault Zones, Point Loma Quadrangle*, dated May 1, 2003.

10. California Geological Survey, 2002, *Guidelines for Evaluating the Hazard of Surface Fault Rupture*, CGS Note 49.

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

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

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

14. City of San Diego 2011, *Guidelines for Geotechnical Reports*.

15. *City of San Diego Seismic Safety Study, Geologic Hazards and Faults*, 2008, Map Sheets 13 and 17.

16. County of San Diego, *San Diego County Multi Jurisdiction Hazard Mitigation Plan, San Diego, California* – Final Draft, July, 2010.

17. Geocon Incorporated, 1993, *Geotechnical Engineering Investigation for Mercado Apartments, San Diego, California,* dated January 1993, (Project No. 04749-31-02).

18. Geocon Incorporated, 1997, *Limited Geotechnical Investigation, Southwest Marine Parking Lot, Main Street And Sampson Street, San Diego, California*, dated December 1997 (Project No. 06023-22-01).

19. Geocon Incorporated, 2000, *Fault Hazard Study, Kelco Plant, 2025 Harbor Drive, San Diego, California,* dated August 2000 (Project No. 06155-22-06).

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

21. Kennedy, M. P., and S. H. Clarke, 1999, *Age of Faulting in San Diego Bay in the Vicinity of the Coronado Bridge,* California Division of Mines and Geology, Open File Report 97-10A.

22. Kennedy, M. P., and S. H. Clarke, 2001, *Late Quaternary Faulting in San Diego Bay and Hazard to the Coronado Bridge*, in California Geology, July/August 2001.

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

24. Munsell® Soil Color Charts," GretagMacbeth, New Windsor, NY, 1998.

25. Legg, M. R., J. C. Borrero, and C. E. Synolakis (2002), *Evaluation of Tsunami Risk to Southern California Coastal Cities*, 2002 NEHRP Professional Fellowship Report, dated January.

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

LIST OF REFERENCES (Concluded)

27. Omega Engineering, *Conceptual Grading Plans, U STOR IT Self Storage, 2209 National Avenue, San Diego, California*, dated October 4, 2017 (Project No. 117138).

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

29. SCS Engineers, *Phase I Environmental Site Assessment, 3350 El Cajon Boulevard, San Diego, California*, dated February 8, 2017 (Project No. 01217029.00).

30. Treiman, Jerome A., *The Rose Canyon Fault Zone Southern California*, California Division of Mines and Geology Publication, 1993.

31. Treiman, J. A, 2002, *Silver Strand Fault, Coronado Fault, Spanish Bight Fault, San Diego Fault, and Downtown Graben Fault, Southern Rose Canyon Fault Zone, San Diego, California*, dated July 17, 2002, California Division of Mines and Geology Fault Evaluation Report FER-245.

32. Treiman, J. A., 2003, *Silver Strand Fault, Coronado Fault, Spanish Bight Fault, San Diego Fault, and Downtown Graben Fault, Southern Rose Canyon Fault Zone, San Diego, California*, dated April 22, 2003, California Division of Mines and Geology Fault Evaluation Report FER-245, Supplement No. 1.

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

34. United States Geological Survey computer program, *U.S. Seismic Design Maps*, http://earthquake.usgs.gov/designmaps/us/application.php.

35. United States Geological Survey, 1934, Point Loma Quadrangle, California, 6.0-Minute Quadrangle, 1:2,400, based on 1923 Topographic Survey.

36. United States Geological Survey, 1942, Point Loma, California 7½-Minute Quadrangle, 1:2,400.

37. URS, 2004, *San Diego County Multi-Jurisdictional Hazard Mitigation Plan, San Diego County, California*, dated March 15, (URS Project No. 27653042.00500).

City of San Diego
Development Services
1222 First Ave., MS-302
San Diego, CA 92101
(619) 446-5000

Storm Water Requirements Applicability Checklist

FORM DS-560
OCTOBER 2016

Project Address: 2209 National Avenue, San Diego CA, 92113	Project Number *(for City Use Only)*:

SECTION 1. Construction Storm Water BMP Requirements:

All construction sites are required to implement construction BMPs in accordance with the performance standards in the Storm Water Standards Manual. Some sites are additionally required to obtain coverage under the State Construction General Permit (CGP)[1], which is administered by the State Water Resources Control Board.

For all projects complete PART A: If project is required to submit a SWPPP or WPCP, continue to PART B.

PART A: Determine Construction Phase Storm Water Requirements.

1. Is the project subject to California's statewide General NPDES permit for Storm Water Discharges Associated with Construction Activities, also known as the State Construction General Permit (CGP)? (Typically projects with land disturbance greater than or equal to 1 acre.)

☐ Yes; SWPPP required, skip questions 2-4 ☒ No; next question

2. Does the project propose construction or demolition activity, including but not limited to, clearing, grading, grubbing, excavation, or any other activity resulting in ground disturbance and contact with storm water runoff?

☒ Yes; WPCP required, skip 3-4 ☐ No; next question

3. Does the project propose routine maintenance to maintain original line and grade, hydraulic capacity, or original purpose of the facility? (Projects such as pipeline/utility replacement)

☐ Yes; WPCP required, skip 4 ☒ No; next question

4. Does the project only include the following Permit types listed below?

- Electrical Permit, Fire Alarm Permit, Fire Sprinkler Permit, Plumbing Permit, Sign Permit, Mechanical Permit, Spa Permit.
- Individual Right of Way Permits that exclusively include only ONE of the following activities: water service, sewer lateral, or utility service.
- Right of Way Permits with a project footprint less than 150 linear feet that exclusively include only ONE of the following activities: curb ramp, sidewalk and driveway apron replacement, pot holing, curb and gutter replacement, and retaining wall encroachments.

☐ Yes; no document required

Check one of the boxes below, and continue to PART B:

☐ If you checked "Yes" for question 1, **a SWPPP is REQUIRED. Continue to PART B**

☒ If you checked "No" for question 1, and checked "Yes" for question 2 or 3, **a WPCP is REQUIRED.** If the project proposes less than 5,000 square feet of ground disturbance AND has less than a 5-foot elevation change over the entire project area, a Minor WPCP may be required instead. **Continue to PART B.**

☐ If you checked "No" for all questions 1-3, and checked "Yes" for question 4 PART B **does not apply and no document is required. Continue to Section 2.**

1. More information on the City's construction BMP requirements as well as CGP requirements can be found at: www.sandiego.gov/stormwater/regulations/index.shtml

PART B: Determine Construction Site Priority

This prioritization must be completed within this form, noted on the plans, and included in the SWPPP or WPCP. The city reserves the right to adjust the priority of projects both before and after construction. Construction projects are assigned an inspection frequency based on if the project has a "high threat to water quality." The City has aligned the local definition of "high threat to water quality" to the risk determination approach of the State Construction General Permit (CGP). The CGP determines risk level based on project specific sediment risk and receiving water risk. Additional inspection is required for projects within the Areas of Special Biological Significance (ASBS) watershed. **NOTE:** The construction priority does **NOT** change construction BMP requirements that apply to projects; rather, it determines the frequency of inspections that will be conducted by city staff.

Complete PART B and continued to Section 2

1. ☐ **ASBS**
   a. Projects located in the ASBS watershed.

2. ☐ **High Priority**
   a. Projects 1 acre or more determined to be Risk Level 2 or Risk Level 3 per the Construction General Permit and not located in the ASBS watershed.
   b. Projects 1 acre or more determined to be LUP Type 2 or LUP Type 3 per the Construction General Permit and not located in the ASBS watershed.

3. ☐ **Medium Priority**
   a. Projects 1 acre or more but not subject to an ASBS or high priority designation.
   b. Projects determined to be Risk Level 1 or LUP Type 1 per the Construction General Permit and not located in the ASBS watershed.

4. ☒ **Low Priority**
   a. Projects requiring a Water Pollution Control Plan but not subject to ASBS, high, or medium priority designation.

SECTION 2. Permanent Storm Water BMP Requirements.

Additional information for determining the requirements is found in the Storm Water Standards Manual.

PART C: Determine if Not Subject to Permanent Storm Water Requirements.

Projects that are considered maintenance, or otherwise not categorized as "new development projects" or "redevelopment projects" according to the Storm Water Standards Manual are not subject to Permanent Storm Water BMPs.

If "yes" is checked for any number in Part C, proceed to Part F and check "Not Subject to Permanent Storm Water BMP Requirements".

If "no" is checked for all of the numbers in Part C continue to Part D.

1.	Does the project only include interior remodels and/or is the project entirely within an existing enclosed structure and does not have the potential to contact storm water?	☐ Yes ☒ No
2.	Does the project only include the construction of overhead or underground utilities without creating new impervious surfaces?	☐ Yes ☒ No
3.	Does the project fall under routine maintenance? Examples include, but are not limited to: roof or exterior structure surface replacement, resurfacing or reconfiguring surface parking lots or existing roadways without expanding the impervious footprint, and routine replacement of damaged pavement (grinding, overlay, and pothole repair).	☐ Yes ☒ No

PART D: PDP Exempt Requirements.

PDP Exempt projects are required to implement site design and source control BMPs.

If "yes" was checked for any questions in Part D, continue to Part F and check the box labeled "PDP Exempt."

If "no" was checked for all questions in Part D, continue to Part E.

1. **Does the project ONLY include new or retrofit sidewalks, bicycle lanes, or trails that:**
   - **Are designed and constructed to direct storm water runoff to adjacent vegetated areas, or other non-erodible permeable areas? Or;**
   - **Are designed and constructed to be hydraulically disconnected from paved streets and roads? Or;**
   - **Are designed and constructed with permeable pavements or surfaces in accordance with the Green Streets guidance in the City's Storm Water Standards manual?**

   ☐ Yes; PDP exempt requirements apply ☒ No; next question

2. Does the project ONLY include retrofitting or redeveloping existing paved alleys, streets or roads designed and constructed in accordance with the Green Streets guidance in the City's Storm Water Standards Manual?

   ☐ Yes; PDP exempt requirements apply ☒ No; project not exempt.

PART E: Determine if Project is a Priority Development Project (PDP).

Projects that match one of the definitions below are subject to additional requirements including preparation of a Storm Water Quality Management Plan (SWQMP).

If "yes" is checked for any number in PART E, continue to PART F and check the box labeled "Priority Development Project".

If "no" is checked for every number in PART E, continue to PART F and check the box labeled "Standard Development Project".

1. **New Development that creates 10,000 square feet or more of impervious surfaces collectively over the project site.** This includes commercial, industrial, residential, mixed-use, and public development projects on public or private land. ☐ Yes ☒ No
2. **Redevelopment project that creates and/or replaces 5,000 square feet or more of impervious surfaces on an existing site of 10,000 square feet or more of impervious surfaces**. This includes commercial, industrial, residential, mixed-use, and public development projects on public or private land. ☒ Yes ☐ No
3. **New development or redevelopment of a restaurant.** Facilities that sell prepared foods and drinks for consumption, including stationary lunch counters and refreshment stands selling prepared foods and drinks for immediate consumption (SIC 5812), and where the land development creates and/or replace 5,000 square feet or more of impervious surface. ☐ Yes ☒ No
4. **New development or redevelopment on a hillside.** The project creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site) and where the development will grade on any natural slope that is twenty-five percent or greater. ☐ Yes ☒ No
5. **New development or redevelopment of a parking lot that creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site).** ☒ Yes ☐ No
6. **New development or redevelopment of streets, roads, highways, freeways, and driveways.** The project creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site). ☐ Yes ☒ No

7. **New development or redevelopment discharging directly to an Environmentally Sensitive Area.** The project creates and/or replaces 2,500 square feet of impervious surface (collectively over project site), and discharges directly to an Environmentally Sensitive Area (ESA). "Discharging directly to" includes flow that is conveyed overland a distance of 200 feet or less from the project to the ESA, or conveyed in a pipe or open channel any distance as an isolated flow from the project to the ESA (i.e. not commingled with flows from adjacent lands).	☐ Yes ☒ No
8. **New development or redevelopment projects of a retail gasoline outlet (RGO) that create and/or replaces 5,000 square feet of impervious surface.** The development project meets the following criteria: (a) 5,000 square feet or more or (b) has a projected Average Daily Traffic (ADT) of 100 or more vehicles per day.	☐ Yes ☒ No
9. **New development or redevelopment projects of an automotive repair shops that creates and/or replaces 5,000 square feet or more of impervious surfaces.** Development projects categorized in any one of Standard Industrial Classification (SIC) codes 5013, 5014, 5541, 7532-7534, or 7536-7539.	☐ Yes ☒ No
10. **Other Pollutant Generating Project.** The project is not covered in the categories above, results in the disturbance of one or more acres of land and is expected to generate pollutants post construction, such as fertilizers and pesticides. This does not include projects creating less than 5,000 sf of impervious surface and where added landscaping does not require regular use of pesticides and fertilizers, such as slope stabilization using native plants. Calculation of the square footage of impervious surface need not include linear pathways that are for infrequent vehicle use, such as emergency maintenance access or bicycle pedestrian use, if they are built with pervious surfaces of if they sheet flow to surrounding pervious surfaces.	☐ Yes ☒ No

PART F: Select the appropriate category based on the outcomes of PART C through PART E.

1.	The project is **NOT SUBJECT TO PERMANENT STORM WATER REQUIREMENTS**.	☐
2.	The project is a **STANDARD DEVELOPMENT PROJECT**. Site design and source control BMP requirements apply. See the Storm Water Standards Manual for guidance.	☐
3.	The project is **PDP EXEMPT**. Site design and source control BMP requirements apply. See the Storm Water Standards Manual for guidance.	☐
4.	The project is a **PRIORITY DEVELOPMENT PROJECT**. Site design, source control, and structural pollutant control BMP requirements apply. See the Storm Water Standards Manual for guidance on determining if project requires a hydromodification plan management	☒

Jonathan Teas	Staff Engineer
Name of Owner or Agent *(Please Print)*	Title
Jonathan Teas	06/27/2018
Signature	Date

WASTE MANAGEMENT PLAN

BARRIO LOGAN U-STOR-IT
2209 NATIONAL AVENUE
CITY OF SAN DIEGO, CALIFORNIA

PTS No. 586276

FEBRUARY 2018

Prepared for: U-STOR-IT Barrio Logan, LLC.
501 W. BROADWAY, STE. 2020
SAN DIEGO, CA 92101

Prepared by: DDCA ARCHITECTS
3321 STATE ROUTE 31
PRAIRE GROVE, IL 60012
(815) 444-8444

TABLE OF CONTENTS

ATTACHMENTS

I. INTRODUCTION

According to the City of San Diego, Development Services Department, California Environmental Quality Act (CEQA) Significance Determination Thresholds (January 2011), projects that include the demolition, construction, and/or renovation of 40,000 square feet or more of building space generate 60 tons of waste or more. This amount of waste is further identified as a potentially significant cumulative impact which can be mitigated by the implementation of a project-specific Waste Management Plan (WMP). This plan will identify measures to reduce the amount of waste generated during site development, demolition/construction and occupancy in order to produce impacts below a level of significance.

The following regulations apply to Site Development, Demolition/Construction and through Occupancy to assure waste is being diverted from landfills. **Construction and Demolition (C&D) Debris Diversion Deposit Ordinance** requires that the majority of construction, demolition, and remodeling projects requiring building, combination, and demolition permits pay a refundable C&D Debris Recycling Deposit and divert at least 50% of their debris by recycling, reusing or donating usable materials.

The Recycling Ordinance requires recycling of plastic and glass bottles and jars, paper, newspaper, metal containers and cardboard at private residences, commercial buildings, and at special events requiring a City permit.

Lastly, **Refuse and Recyclable Materials Storage Regulations Ordinance** requires the diversion of recyclable materials from landfill disposal to conserve the capacity and extend the useful life of the Miramar landfill, and reduce greenhouse gas emissions.

II. PROJECT DESCRIPTION

The proposed project is the construction of a 158,670 s.f. self-storage building which includes 26,625 s.f. of open parking area. The building is to be a total of five (5) stories, two (2) below ground and three (3) above with a 31,734 s.f. footprint at each level. The existing 6,275 s.f. abandoned masonry building and adjacent asphalt parking lot are to be demolished. Project site is located at 2209 National Ave. at the southeast corner of National Avenue and Sampson Street. See Figure 1 for aerial view of outlined project location

Figure 1 – Aerial of project site

The total project site is 35,130 s.f. (0.806 acres). The site is surrounded by a combination of residential & commercial. The project includes the demolition of the existing onsite building and parking lot followed by the construction of a new building and perimeter surface improvements

III. PRECONSTRUCTION

U-Stor-It Barrio Logan, LLC will monitor the project at 2209 National Avenue to ensure proper measures are taken by the contractor(s) and staff to implement waste reduction and recycling efforts. Following is a list, though not inclusive, of procedures to assist in carrying out the Waste Management Plan:

1. Review and understand the Waste Management Plan.
2. Work with contractor(s) to estimate quantities of each type of material that will be salvaged, recycled, or disposed of as waste, then assist contractor(s) with documentation.
3. Review and update procedures as needed for material separation and verify availability of containers and bins needed to avoid delays.
4. Review and update procedures for periodic solid waste collection and transportation to recycling and disposal facilities.
5. Review and update solid waste management requirements for each trade.

From preconstruction to occupancy of the Self-storage building project, this Waste Management plan will provide contractors and tenants guidelines to ensure the proper reduction, segregation, recycling, and disposal of demolition, construction, and on-going operational waste. Proper segregation of recyclable materials is required based on the type of materials generated and the availability of recycling facilities able to accept those materials.

IV. DEMOLITION AND CONSTRUCTION WASTE

In order to mitigate for any solid waste impacts identified for the Barrio Logan U-Stor-It project, offsite waste disposal shall target a minimum of 75% of all Construction, Demolition, and Land-Clearing waste to be diverted by weight from landfills.

Contractor Requirements. U-Stor-It Barrio Logan, LLC. shall provide specific contract language for the Barrio Logan self-storage building project to implement this Waste Management Plan (WMP). The contract language will be made available to City personnel for verification. Contract language will require that:

- Specified demolition and construction materials will be reused or recycled onsite; others will be segregated for transport to specified recycling facilities.
- The contractor hired must determine the necessary capacity of dumpsters for each material type prior to obtaining the first demolition permit.
- The contractor(s) will be required to perform daily inspections of the demolition/construction site to ensure compliance with the requirements of the WMP and all other applicable laws and ordinances.
- Daily inspections will include verifying the availability and number of dumpsters based on amount of debris being generated, assuring correct labeling of dumpsters, proper sorting and segregation of materials.
- No more than 10% by volume of contamination may occur in each dumpster.
- The contractors and subcontractors will coordinate and work within the Waste Management Plan guidelines to minimize the over-purchasing of construction materials to lower the amount of materials taken to recycling and disposal facilities.

It is expected that approximately 83.0% of the material generated from the Barrio Logan self-storage building's project demolition will be diverted by salvaging or source separating the asphalt, concrete, landscape debris and other materials noted in Table 4.1. Approximately 830 tons of waste is expected to be generated during demolition. This is an assumption and is used as a place holder until the hired contractor can accurately assess expected demolition quantities. Approximately 685 tons of materials would be recycled, to include trees/shrubs, concrete, asphalt, building materials, ceiling tiles, drywall, and scrap metal. Approximately 145 tons of debris would be disposed in a landfill. Tonnage of each material is subject to change based upon the contractor's actual data. U-Stor-It Barrio Logan, LLC. may utilize the Certified Facilities list found in Appendix B.

TABLE 4.1 Estimated Demolition Quantities & Tons Diverted

Material	Estimated Tonnage	Handling Facility	Diversion Rate (Percent)	Tons Diverted	Tons Disposed
Asphalt/Concrete	568	Hanson Aggregates West - Miramar	90%	512	56
Landscaping	15	Miramar Greenery	100%	15	0
Building Materials	22	Habitat for Humanity ReStore	76%	16	6
Floor Tile	11	Hanson Aggregates West - Miramar	82%	9	2
Masonry Brick	114	Vulcan Carol Canyon	88%	100	14
Scrap Metal	44	Edco Station Transfer & Buy Back Center	75%	33	11
Garbage/Trash	56	Miramar Landfill	0%		56
Total	830		83%	685	145
Note: Portions of material type based on demolition estimates of similar industrial developments.					

Excavation/Grading. The surrounding foundation of Barrio Logan building will implement the shotcrete method which does not require an over dig and therefore reduce amount total of existing soil excavation. The project would include approximately 18,600 cubic yards of cut soil of which 64 cubic yards can be used in biofiltration area. The balance of 18,536 cubic yards that can be used as clean fill will be taken to Hanson Aggregates West – Miramar site for 100% diversion.

Construction Waste. During the construction, the debris generated is expected to include the materials listed in Table 4.2. Materials shall be source separated as indicated in Table 4.2.

The City of San Diego ESD requires projects to estimate tonnage of expected construction waste. The Barrio Logan project includes a total of 158,670 square feet of new construction. As provided by Environmental Services Department and for purposes of this Waste Management Plan, an amount of 3 pounds of waste per square foot for waste generation on new construction is used to calculate expected tonnage as follows:

158,670 sq. ft. x 3/2,000lbs = approx. 238 tons

The approximate 238 tons is an assumption and is used as a place holder until further detail is provided and the project contractor can accurately assess expected waste. The exact quantity of each material is also approximate at this time and amounts used as a place holder in Table 4.2 which should be referenced by project contractors to separate waste materials according to the material types.

TABLE 4.2 Estimated Construction Waste

Material/Type	Generated (tons)	Handling Facility	Estimated Diverted	Estimated Disposed
Metals (Pipes, rebar, flashing, steel, aluminum, copper, brass, stainless steel)	2	Edco Station Transfer & Buy Back Center	2	0
Polystyrene	1	Cactus Recycling	1	0
Blocks, CMU	13.25	Vulcan Carol Canyon	13.25	0
Asphalt, concrete	16.75	Hanson Aggregate West - Miramar	16.75	0
Roofing, SSR	2.5	Edco Station Transfer & Buy Back Center	2.5	0
Mixed Debris (Insulation, vinyl, doors, floor tile, plastic pipes, film, broken glass, drywall)	180	Edco Station Transfer & Buy Back Center	145	35
Carpet/Carpet Padding	1	DFS Flooring	1	0
Trash	21.5	Miramar Landfill	0	21.5
Total	238		181.5	56.5
Note: Portions of material type based on construction estimates of similar industrial developments				

Based on these estimates, and on providing segregation of these materials, the project would accomplish 76.2% diversion of construction waste. An estimated 56.5 tons would end up going to landfill disposal. When construction waste is considered together with demolition waste 1,068 tons of demolition and construction waste would be generated, but approximately 81% is expected to be diverted from disposal. To ensure this result, contractors will be required to comply with the following methods and procedures below:

1. Construction and Land-Clearing containers will be provided for waste that is to be recycled. Containers shall be clearly labeled, with a list of acceptable and unacceptable materials. The list of acceptable materials must be the same as the materials recycled at the receiving material recovery facility or recycling processor.
2. The collection containers for recyclable Construction and Land-Clearing waste must contain no more than 10% non-recyclable materials, by volume.
3. Use detailed material estimates to reduce risk of unplanned and potentially wasteful material cuts.
4. Conduct daily visual inspections of dumpsters and recycling bins to remove contaminants.
5. Remove demolition and construction waste materials from the project site at least once every week to ensure no over-topping of waste bins. The accumulation and burning of on-site Construction, Demolition, and Land-Clearing waste materials will be prohibited.

Furthermore, the proposed building will be required to meet the following State law and City of San Diego Municipal Code requirements:

1. The City's C&D Debris Diversion Deposit Program which requires a refundable deposit based on the tonnage and value of the expected recyclable waste materials as part of the building permit requirements.
2. The City's C&D Recycling Ordinance which requires identification and sorting of demolition and construction waste materials to be diverted to the appropriate recycling facility.
3. The City's Recycling Ordinance which requires that collection of recyclable materials must be provided.
4. The City's Storage Ordinance which requires that areas for recyclable material collection must be provided.
5. This Waste Management Plan –The waste contractor will provide monthly reports regarding the amount of waste and recyclable materials to U-Stor-It Barrio Logan, LLC. who will be responsible for compliance actions with the aforementioned guidelines and make adjustments as needed to maintain conformance.

V. OCCUPANCY WASTE

The Barrio Logan self-storage building development will be managed by U-Stor-It Barrio Logan, LLC. During the Occupancy Phase, it is estimated that 280.5 tons per year will be generated by the new development. The expected waste generation was calculated using the equivalent self-storage factor provided by City of San Diego ESD

TABLE 5.1: Waste Generation – Occupancy Waste

Use	Intensity (sq.ft.)	Waste Generation Rate (tons/year/sq. ft.)	Estimated Waste Generated (tons/year)
Industrial Office	158,670	0.0017	269.7
Note: Based on City of San Diego Waste Generation Factors, Appendix C.			

The Barrio Logan U-Stor-It building will be required to comply with City of San Diego Municipal Code section 142.0830 Refuse and Recyclable Material Storage Regulations for Non-Residential Development (Table 142.08C). The minimum storage amount required can be found in Table 5.2 below.

Table 5.2 Minimum Refuse and Recyclable Material Storage Areas for Non-Residential Development

Gross Floor Area per Development (square feet)	Minimum Refuse Storage Area per Development (sq. ft.)	Minimum Recyclable Material Storage Area per Development (sq. ft.)	Total Minimum Storage Area per Development (sq. ft.)
158,670	336	336	672

In order to continually reduce waste delivered to the landfill during the life of the project, trash, recycling, and green waste bins will be provided for each development. Information will be provided to occupants to encourage recycling of all paper products, cardboard, glass, aluminum cans, recyclable plastics, and yard waste. Compliance with the recycling ordinance, which requires the provision of educational materials and separate recycling bins, and with the storage ordinance, which requires that sufficient space for recycling bins be provided, is estimated to reduce waste by 40%. Thus 161.8 tons per year would still be destined for disposal. Additional measures often taken to help mitigate this quantity of trash include:

- Ensuring that landscape debris is minimized, used onsite when possible, and what remains is composted.
- Surpassing the 75% waste reduction target during demolition and construction.
- Providing recyclable materials collection in the open/ parking areas.

VI. CONCLUSION

The Barrio Logan U-Stor-It project anticipates 830 tons of demolition waste and 248 tons of construction waste for a total of 1,068 tons of waste. The materials in Tables 4.1 and 4.2 are expected to be diverted either by reuses or source separating and sent to the certified facilities mentioned in Chapter 4 or similar, reaching a potential 81.0% reduction of waste disposal.

The proposed self-storage building project at 2209 National Avenue of 158,670 SF would generate approximately 269.7 tons of waste per year and be required to provide 672 square feet of refuse and recyclable material storage area.

To ensure that waste is properly managed, U-Stor-It shall establish waste management contract language ensuring:

- Specified demolition and construction materials will be reused or recycled onsite; others will be segregated for transport to specified recycling facilities.
- The contractor hired must determine the necessary capacity of dumpsters for each material type prior to obtaining the first demolition permit.
- The contractor(s) will be required to perform daily inspections of the demolition/construction site to ensure compliance with the requirements of the WMP and all other applicable laws and ordinances.
- Daily inspections will include verifying the availability and number of dumpsters based on amount of debris being generated, assuring correct labeling of dumpsters, proper sorting and segregation of materials.
- No more than 10% by volume of contamination may occur in each dumpster.
- The contractors and subcontractors will coordinate and work within the plan guidelines to minimize the over-purchasing of construction materials to lower the amount of materials taken to recycling and disposal facilities. Ways in which the project will minimize over-purchasing is to purchase pre-cut materials, work closely amongst designers, contractors, and suppliers.

APPENDIX A
CONSTRUCTION & DEMOLITION DEBRIS-CONVERSION RATE TABLE

CITY OF SAN DIEGO

Construction & Demolition (C&D) Debris
Conversion Rate Table

This worksheet lists materials typically generated from a constructionor demolition project and provides formulas for converting common units (i.e. cubic yards, square feet, and board feet) to tons. It is a tool that should be used for preparing your Waste Mangement Form - Part I, which requires that quantities be provided in tons.

Note: Weigh receipts are required for your refund request.

Step 1: Enter the estimated quantity for each applicable material in Column I, based on units

Step 2: Multiply by Tons/Unit figure listed in Column II. Enter the result for each material in Column III.

If using Excel version, column III will automatically calculate tons.

Step 3: Enter quantities for each separated material from Column III on this worksheet into the corresponding section of your Waste Management Form - Part I.

		Column I			Column II		Column III
Category	**Material**	**Volume**	**Unit**		**Tons/Unit**		**Tons**
Asphalt/Concrete	Asphalt (broken)		cy	x	0.70	=	
	Concrete (broken)		cy	x	1.20	=	
	Concrete (solid slab)		cy	x	1.30	=	
Brick/Masonry/Tile	Brick (broken)		cy	x	0.70	=	
	Brick (whole, palletized)		cy	x	1.51	=	
	Masonry Brick (broken)		cy	x	0.60	=	
	Tile		sq ft	x	0.00175	=	
Building Materials (doors, windows, cabinets, etc.)			cy	x	0.15	=	
Cardboard (flat)			cy	x	0.05	=	
Carpet	By square foot		sq ft	x	0.0005	=	
	By cubic yard		cy	x	0.30	=	
Carpet Padding/Foam			sq ft	x	0.000125	=	
Ceiling Tiles	Whole (palletized)		sq ft	x	0.0003	=	
	Loose		cy	x	0.09	=	
Drywall (new or used)	1/2" (by square foot)		sq ft	x	0.0008	=	
	5/8" (by square foot)		sq ft	x	0.00105	=	
	Demo/used (by cubic yd)		cy	x	0.25	=	
Earth	Loose/Dry		cy	x	1.20	=	
	Excavated/Wet		cy	x	1.30	=	
	Sand (loose)		cy	x	1.20	=	
Landscape Debris (brush, trees, etc)			cy	x	0.15	=	
Mixed Debris	Construction		cy	x	0.18	=	
	Demolition		cy	x	1.19	=	
Scrap metal			cy	x	0.51	=	
Shingles, asphalt			cy	x	0.22	=	
Stone (crushed)			cy	x	2.35	=	
Unpainted Wood & Pallets	By board foot		bd ft	x	0.001375	=	
	By cubic yard		cy	x	0.15	=	
Garbage/Trash			cy	x	0.18	=	
Other (estimated weight)			cy	x	estimate	=	
			cy	x	estimate	=	
			cy	x	estimate	=	
					Total All		

APPENDIX B
CONSTRUCTION & DEMOLITION RECYCLING FACILITIES DIRECTORY

SD Environmental Services

2018 Certified Construction & Demolition (C&D) Recycling Facility Directory

The City of San Diego certifies these facilities to accept the materials listed in each category. Hazardous materials are not accepted. The diversion rate for these materials shall be considered 100 percent, except mixed C&D debris, which updates quarterly. The City is not responsible for changes in facility information. Please call ahead to confirm details such as accepted materials, days and hours of operation, limitations on vehicle types and cost. For more information visit: www.recyclingworks.com.

The Miramar Landfill and other landfills do not recycle mixed C&D debris. *To receive recycling credit:* *A. The mixed C&D facility and transfer station receipts have to be coded as C&D debris and have a project address or permit number on the receipt.* *B. You must notify weighmaster that your load is subject to the City of San Diego C&D Ordinance.*	Mixed C&D Debris	Asphalt/Concrete	Brick/Block/Rock	Building Materials for Reuse	Cardboard	Carpet	Carpet Padding	Ceiling Tile	Ceramic Tile/Porcelain	Clean Fill Dirt	Clean Wood/Green Waste	Drywall	Industrial Plastics	Lamps/Light Fixtures	Metal	Mixed Inerts	Styrofoam Blocks
EDCO Recovery & Transfer 3660 Dalbergia St., San Diego, CA 92113 619-234-7774 | www.edcodisposal.com/public-disposal	71%											•					
EDCO Station Transfer Station & Buy Back Center 8184 Commercial St., La Mesa, CA 91942 619-466-3355 | www.edcodisposal.com/public-disposal	71%				•							•			•		
EDCO CDI Recycling & Buy Back Center 224 S. Las Posas Rd., San Marcos, CA 92078 760-744-2700 | www.edcodisposal.com/public-disposal	90%				•										•		
Escondido Resource Recovery 1044 W. Washington Ave., Escondido 760-745-3203 | www.edcodisposal.com/public-disposal	71%																
Fallbrook Transfer Station & Buy Back Center 550 W. Aviation Rd., Fallbrook, CA 92028 760-728-6114 | www.edcodisposal.com/public-disposal	71%				•										•		
Otay C&D/Inert Debris Processing Facility 1700 Maxwell Rd., Chula Vista, CA 91913 619-421-3773 | www.sd.disposal.com	72%																
Ramona Transfer Station & Buy Back Center 324 Maple St., Ramona, CA 92065 760-789-0516 | www.edcodisposal.com/public-disposal	71%				•										•		
SANCO Resource Recovery & Buy Back Center 6750 Federal Blvd, Lemon Grove, CA 91945 619-287-5696 | www.edcodisposal.com/public-disposal	71%				•										•		
All American Recycling 10805 Kenney St., Santee, CA 92071 619-508-1155 (Must call for appointment)						•											
Allan Company 6733 Consolidated Way, San Diego, CA 92121 858-578-9300 | www.allancompany.com/facilities.htm					•										•		
Allan Company Miramar Recycling 5165 Convoy St., San Diego, CA 92111 858-268-8971 | www.allancompany.com/facilities.htm					•										•		
AMS 4674 Cardin St., San Diego, CA 92111 858-541-1977 | www.a-m-s.com								•									

The Miramar Landfill and other landfills do not recycle mixed C&D debris. *To receive recycling credit:* *A. The mixed C&D facility and transfer station receipts have to be coded as C&D debris <u>and</u> have a project address or permit number on the receipt.* *B. You must notify weighmaster that your load is subject to the City of San Diego C&D Ordinance.*	Mixed C&D Debris	Asphalt/Concrete	Brick/Block/Rock	Building Materials for Reuse	Cardboard	Carpet	Carpet Padding	Ceiling Tile	Ceramic Tile/Porcelain	Clean Fill Dirt	Clean Wood/Green Waste	Drywall	Industrial Plastics	Lamps/Light Fixtures	Metal	Mixed Inerts	Styrofoam Blocks
Armstrong World Industries, Inc. 300 S. Myrida St., Pensacola, FL 32505 877-276-7876 (Press 1, Then 8) www.armSt.rong.com/commceilingsna								•									
Cactus Recycling 8710 Avenida De La Fuente, San Diego, CA 92154 619-661-1283 | www.cactusrecycling.com					•								•		•		•
DFS Flooring 10178 Willow Creek Rd., San Diego, CA 92131 858-630-5200 | www.dfsflooring.com						•	•										
Duco Metals 220 Bingham Drive Suite 100, San Marcos, CA 92069 760-747-6330 | www.ducometals.com															•		
Enniss Incorporated 12421 Vigilante Rd., Lakeside, CA 92040 619-443-9024 | www.ennissinc.com		•	•						•	•							
Escondido Sand and Gravel 500 N. Tulip St., Escondido, CA 92025 760-432-4690 | www.weirasphalt.com/esg		•															
Habitat for Humanity Restore 10222 San Diego Mission Rd., San Diego, CA 92108 619-516-5267 | www.sdhfh.org/reSt.ore.php				•													
Hanson Aggregates West. – Lakeside Plant 12560 Highway 67, Lakeside, CA 92040 858-547-2141		•															
Hanson Aggregates West. – Miramar 9229 Harris Plant Rd., San Diego, CA 92126 858-974-3849		•								•							
HVAC Exchange 2675 Faivre St., Chula Vista, CA 91911 619-423-1855 | www.thehvacexchange.com															•		
IMS Recycling Services 2740 Boston Ave., San Diego, CA 92113 619-423-1564 | www.imsrecyclingservices.com					•								•				
IMS Recycling Services 2697 Main St., San Diego, CA 92113 619-231-2521 | www.imsrecyclingservices.com													•		•		
Inland Pacific Resource Recovery 12650 Slaughterhouse Canyon Rd., Lakeside, CA 92040 619-390-1418											•						
Lamp Disposal Solutions 1405 30th St., San Diego, CA 92154 858-569-1807 | www.lampdisposalsolutions.com														•			
Los Angeles Fiber Company 4920 S. Boyle Ave., Vernon, CA 90058 323-589-5637 | www.lafiber.com						•	•										
Miramar Greenery, City of San Diego 5180 Convoy St., San Diego, CA 92111 858-694-7000 | www.sandiego.gov/environmental-services/miramar/greenery.shtml											•						

The Miramar Landfill and other landfills do not recycle mixed C&D debris. *To receive recycling credit:* *A. The mixed C&D facility and transfer station receipts have to be coded as C&D debris <u>and</u> have a project address or permit number on the receipt.* *B. You must notify weighmaster that your load is subject to the City of San Diego C&D Ordinance.*	Mixed C&D Debris	Asphalt/Concrete	Brick/Block/Rock	Building Materials for Reuse	Cardboard	Carpet	Carpet Padding	Ceiling Tile	Ceramic Tile/Porcelain	Clean Fill Dirt	Clean Wood/Green Waste	Drywall	Industrial Plastics	Lamps/Light Fixtures	Metal	Mixed Inerts	Styrofoam Blocks
Moody's 3210 Oceanside Blvd., Oceanside, CA 92056 760-433-3316		•								•						•	
Otay Valley Rock, LLC 2041 Heritage Rd., Chula Vista, CA 91913 619-591-4717 | www.otayrock.com		•															
Reclaimed Aggregates Chula ViSt.a 855 Energy Way, Chula Vista, CA 91913 619-656-1836		•														•	
Reconstruction Warehouse 3650 Hancock St.., San Diego, CA 92110 619-795-7326 | www.recowarehouse.com				•													
Robertson's Ready Mix 2094 Willow Glen Dr., El Cajon, CA 92019 619-593-1856		•								•						•	
Romero General Construction Corp. 8354 Nelson Way, Escondido, CA 92026 760-749-9312 | www.romerogc.com/crushing/nelsonway.htm		•															
SA Recycling 3055 Commercial St.., San Diego, CA 92113 619-238-6740 | www.sarecycling.com															•		
SA Recycling 1211 S. 32nd St.., San Diego, CA 92113 619-234-6691 | www.sarecycling.com															•		
Universal Waste Disposal 8051 Wing Avenue, El Cajon, CA 92020 619-438-1093 | www.universalwaSt.edisposal.com														•			
Vulcan Carol Canyon Landfill and Recycle Site 10051 Black Mountain Rd., San Diego, CA 92126 858-530-9465 | www.vulcanmaterials.com		•	•							•						•	
Vulcan Otay Asphalt Recycle Center 7522 Paseo de la Fuente, San Diego, CA 92154 619-571-1945 | www.vulcanmaterials.com		•															