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

Scripps Mercy Hospital

PTS #658548

[Insert Drawing Number (if applicable) and Internal Order Number (if applicable)]

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

Engineer of Work:

Jeffrey Gavazza C-59894

Provide Wet Signature and Stamp Above Line

Prepared For:

Scripps Health

10140 Campus Point Drive, Suite 210

San Diego, CA 92037

[Insert Applicant Phone Number]

Prepared By:

KPFF Consulting Engineers

700 South Flower Street, Suite 2100
Los Angeles, CA 90017

3131 Camino Del Rio North, Suite 1080
San Diego, CA 92108

213-418-0201

Date:

06/01/2022

Approved by: City of San Diego   Date

The City of SAN DIEGO

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Project Name: Scripps Mercy Hospital

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- Attachment 3: Structural BMP Maintenance Plan
    - Maintenance Agreement (Form DS-3247) (when applicable)
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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: Scripps Mercy Hospital
Permit Application PTS 658548

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.

Engineer of Work's Signature

C-59894 | 12/31/2022

PE# | Expiration Date

Jeffrey Gavazza

Print Name

KPFF Consulting Engineers

Company

Date

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	4/10/2020	☑ Preliminary Design/Planning/CEQA ☐ Final Design	**Initial Submittal**
2	8/14/2020	☑ Preliminary Design/Planning/CEQA ☐ Final Design	2nd Submittal
3	11/18/2020	☑ Preliminary Design/Planning/CEQA ☐ Final Design	3rd Submittal
4	6/1/2022	☐ Preliminary Design/Planning/CEQA ☑ Final Design	Revision 7

Project Vicinity Map

Project Name: Scripps Mercy Hospital
Permit Application PTS 658548

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
November 2018

Project Address: 4077 FIFTH AVENUE, SAN DIEGO, CA	Project Number: 658548

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 Regional Water Quality 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/or contact with storm water?

☐ Yes; WPCP required, skip questions 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 question 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 that qualify as Risk Level 2 or Risk Level 3 per the Construction General Permit (CGP) and not located in the ASBS watershed.

b. Projects that qualify as LUP Type 2 or LUP Type 3 per the CGP and not located in the ASBS watershed.

3. ☐ **Medium Priority**

a. Projects that are not located in an ASBS watershed or designated as a High priority site.

b. Projects that qualify as Risk Level 1 or LUP Type 1 per the CGP and not located in an ASBS watershed.

c. WPCP projects (>5,000sf of ground disturbance) located within the Los Penasquitos watershed management area.

4. ☐ **Low Priority**

a. Projects not subject to a Medium or High site priority designation and are not located in an ASBS watershed.

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

Clear Page 2

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

Clear Page 3

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 ☒

Name of Owner or Agent *(Please Print)*	Title
Signature	Date

Clear Page 4

Clear Form

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Applicability of Permanent, Post-Construction Storm Water BMP Requirements			Form I-1
Project Identification			
Project Name: Scripps Mercy Hospital			
Permit Application Number: 658548			Date: 03/04/2022
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):			
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:			

Project Name: Scripps Mercy Hospital

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:		
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: No CCSYAs were identified onsite or upstream.		

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.

See Attachment 2a

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Site Information Checklist For PDPs	Form I-3B
Project Summary Information	
Project Name	Scripps Mercy Hospital
Project Address	4077 Fifth Ave, San Diego, CA 92103
Assessor's Parcel Number(s) (APN(s))	444-710-25, 444-533-25, 444-533-26
Permit Application Number	658548
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)	Mission San Diego (907.11)
Project Area (total area of Assessor's Parcel(s) associated with the project or total area of the right-of-way)	15.39 Acres (670,400 Square Feet)
Area to be disturbed by the project (Project Footprint)	11.08 Acres (482,800 Square Feet)
Project Proposed Impervious Area (subset of Project Footprint)	7.61 Acres (331,500 Square Feet)
Project Proposed Pervious Area (subset of Project Footprint)	3.47 Acres (151,300 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	-9 %

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 infrastructure includes: Cancer Center, 6th Ave Parking Structure, Mercy Staff Memorial Garden, Parking Lot 4.1 and 12, 550 Washington and its parking structure, Central Energy Plant, College building, Chapel, Emergency Department, Behavioral Health Clinic, and the main hospital building.
Existing Land Cover Includes (select all that apply): ☐Vegetative Cover ☑Non-Vegetated Pervious Areas ☑Impervious Areas Description / Additional Information:
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:

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. 2. Runoff from offsite is not conveyed through the site. 3. Drainage conveyance networks within this project site consist of various elements. Lateral pipes ranging from 4" to 8" collect stormwater and converge into larger pipes that continue to convey runoff out of the project site. For the northern half of the site, stormwater is discharged via a connection to 24" RCP at the corner of 4th and Lewis St. For the southern half of the site, stormwater is collected and directed towards a manhole located by 6th Ave. The manhole discharges the collected stormwater eastward across 6th Ave with a 18" CMP. For a small portion at the intersection of 4th Ave and Washington St, stormwater is captured and discharged via an existing 24" RCP along Washington. 4. There are three main existing discharge locations in this project site. First discharge location is Mercy Canyon, which is located to the east of the existing parking structures located at the north side of the hospital campus. Captured storm drain is conveyed to this location via a 24" RCP storm line, which ends in a surface outfall structure to discharge into the canyon. The second discharge location for this existing hospital campus is located on 6th Ave, at the storm manhole adjacent to the existing Right-of-Way line. Once stormwater is directed to the low point at the entrance along 6th Ave, the collected runoff is funneled to an existing manhole. Stormwater then is discharged out of the site via an 18" CMP City main, which continues its route across 6th Ave. The third discharge location is around the intersection of 4th Ave and Washington St. Stormwater is captured and discharged off-site southward via an existing 24" RCP along Washington.

Form I-3B Page 4 of 11
Description of Proposed Site Development and Drainage Patterns
Project Description / Proposed Land Use and/or Activities: The proposed land use of the Scripps Mercy Campus is listed below: - One medical office building (Medical Office Building North) - Two Hospital towers (Hospital I and Hospital II) - Hospital Support Building with subterranean parking structure - Ambulance drop-off area (Emergency Department) - Central Energy Plant Expansion - Loading dock area - Two Utility Yards, covered in gravel (North of College Building, East of Chapel)
List/describe proposed impervious features of the project (e.g., buildings, roadways, parking lots, courtyards, athletic courts, other impervious features): The total impervious area created/replacing existing impervious area in the proposed condition is 7.61 ac., while the treated impervious area for the project site is 8.35 ac. The proposed BMPs are oversized to treat and detain the stormwater generated within the area of disturbance, accounting for existing impervious and pervious areas that will remain undisturbed within the project limit but still need treatment. Proposed impervious features of the project consists of building and roof surfaces, pavements for both pedestrian and vehicular use, parkings lots, and structures such as stairs and walls.
List/describe proposed pervious features of the project (e.g., landscape areas): Pervious features of this project site include gravel for utility yards and landscaped areas, and stormwater treatment facilities such as biofiltration planters and modular wetland systems.
Does the project include grading and changes to site topography? ☑ Yes ☐ No Description / Additional Information: Grading: Demolition of existing buildings, private roadways, parking lots, and rough grading.

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:

In the northern half of the campus, roof runoff from Medical Office Building I, Medical Office Building II, Lewis St, and western half of Hospital II will be collected and treated through biofiltration planters. The bio-filtered runoff will be collected and conveyed off-site to an existing 24" RCP along 4th Ave, which will ultimately discharge the treated stormwater out to Mercy Canyon.

Roof drains and catch basins throughout site will capture and treat runoff in DMAs by directing stormwater to BMPs for Hospital I, Hospital Support Building (HSB), HSB Parking Structure, and the Loading Dock Area. Stormwater in HSB and HSB Plaza will be treated by the south side of HSPT I and discharge towards the existing storm line on 6th Ave. Runoff generated on HSPT I and Loading Dock will be first detained in large cisterns in order to comply with hydromodification requirements defined by the San Diego Regional Model BMP Design Manual. The flow controlled runoff from cisterns will be treated via a compact biofiltration chamber (Modular Wetland) before discharging to a proposed storm drain cleanout at the Loading Dock.

The primary discharge points of this proposed project site are listed below:
- Mercy Canyon
- 6th Ave

Runoff exceeding the hydromodification threshold will be captured and conveyed off-site via overflow drains within the BMPs, typical throughout the site. The overflow drains discharge at the curb. The design flow to the discharge location was calculated using the Rational Method per the City of San Diego Drainage Design manual. Please refer to Attachment 2d for additional information.

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:

Project Name: Scripps Mercy Hospital

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) Site runoff enters catch basin at the northeastern site corner, discharges into the storm conveyance system which daylights along Cabrillo Fwy, then conveyed in public channels to San Diego River.The project site is part of the San Diego River watershed and discharges into Pacific Ocean via San Diego River.
Provide a summary of all beneficial uses of receiving waters downstream of the project discharge locations AGR, IND, REC1, REC2, WARM, WILD, RARE
Identify all ASBS (areas of special biological significance) receiving waters downstream of the project discharge locations
Provide distance from project outfall location to impaired or sensitive receiving waters The site discharge location is approximately 1.1 miles from San Diego River.
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 There are three Multi Habitat Planning Areas (MHPA) within a mile radius of the project site. First MPHA is located approximately 0.5 miles away in the northwest direction of the site. Located northeast of project site, the second area about 0.3 miles away and on the other side of the 163 Hwy. The third MPHA is approximately 0.7 miles to the southeast of the project site.

Project Name: Scripps Mercy Hospital

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 River (Lower)	Enterococcus	Cadmium
	Fecal Coliform	Nitrogen
	Low Dissolved Oxygen	Phosphorus
	Manganese	Total Dissolved Solids
	Nitrogen	Toxicity
	Phosphorus	
	Total Dissolved Solids	
	Toxicity	

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): 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:

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.   POC #1: Connection to existing 24" RCP storm drain line at 4th Ave, which ultimately discharges out to Mercy Canyon via a surface outfall.   POC #2: Connection to proposed storm drain cleanout at the Loading Dock, which discharges stormwater off-site via a private 18" storm drain line across 6th Ave.
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:
Discussion / Additional Information: (optional)

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.
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.

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:			
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:			
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.			

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:			
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: Mercy Canyon, located at the north portion of the Project Area, will be conserved and undisturbed.			

Project Name: Scripps Mercy Hospital

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:			
4.3.4 Minimize Soil Compaction	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.3.4 not implemented: Geotechnical report will show the minimum soil compaction.			
4.3.5 Impervious Area Dispersion	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.3.5 not implemented: 5-3: Impervious area dispersion credit volume was not incorporated as we are proposing alternative site design BMPs to satisfy treatment and volume retention performance of necessary DMAs.			
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: 6a-1: All captured storm water will be routed to biofiltration basins or modular wetland system, but without installing green roof. 6a-2: All captured storm water will be routed to biofiltration basins or modular wetland system, but without installing green roof. 6b-1: Permeable pavements will not be implemented as collected storm water will be routed to biofiltration basins or modular wetland system.			
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: All storm water will be routed to biofiltration basins or modular wetland system.			
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

Form I-5B Page 4 of 4
Insert Site Map with all site design BMPs identified:
For site map showing site design BMPs, refer to DMA Map (Attachment 1a).

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. PDP Structural BMP was selected by following the procedure on Figure 5.1 and 5.2 of the City of San Diego Storm Water Standards Manual. The site was determined to not be self-mitigating, de minimis, or self-retaining, while harvest and reuse was determined to be infeasible. Due to the underlying underlying Scripps formation layer, the campus is under 'No Infiltration' condition. Thus, biofiltration planters were chosen and sized for the DCV. For DMA 1, a compact biofiltration system (BF-3) Modular Wetland and cisterns was sized per volume sizing factor value in hydromodification requirements. Rest of the DMAs implement Biofiltration planters (BF-1). In total, there will be a total of 11 biofiltration planters implemented, including Modular Wetland system by 6th Ave. DMA 7, 8, and 9 will each have a biofiltration planter along the private drive, Lewis Street, with filtered runoff from each DMA joining site storm main along Lewis Street. Filtered runoff from DMA 7, 8, 9 will merge with storm main along Lewis St before exiting the site via a new manhole connection to an existing 24" RCP public storm main on 4th Ave. (Continue on page 2 as necessary.)	

Form I-6 Page 2 of 24

(Continued from page 1)

In DMA 6A, 6B and 6C, there will each be a connected planter which will convey treated stormwater to the existing 24" public RCP main, located south of the existing Parking Lot 12. The existing public main will ultimately discharge out to Mercy Canyon.

DMA 5, the northern and westen portion of HSPT II, will also have one biofiltration planter, treating and discharging stormwater overflow toward 4th Ave. DMA 4, covering the Ambulance Yard, will have a biofiltration planter along 5th Ave, and also direct runoff north towards the existing 24" RCP main, which will eventually discharge at Mercy Canyon.

DMA 2, corresponding to Hospital Support Building and its plaza area, will collect and drain its stormwater to a biofiltration planter located to the south of HSPT I. Storm drain from this DMA will eventually discharge out to campus storm main.

DMA 3, the eastern and southern portion of HSPT II, will drain captured stormwater southward to a biofiltration planter. Treated runoff will then be conveyed with a stormwater cleanout that will drain treated site runoff to a private 18" storm drain main that connects to an existing storm drain cleanout across 6th Ave.

The structural BMP utilized for DMA 1 is the Modular Wetland system connected downstream of stormwater cisterns. This strategy was selected due to the limited amount of available landscape square footage to satisfy the minimum biofiltration size. A flow control riser will regulate stormwater flow into the proposed cisterns. Then, Modular Wetland receives the low velocity flow from the cisterns and sends stormwater horizontally through filtration chambers for treatment, as shown in CUP-11. Treated stormwater will merge with a private campus storm drain main, and eventually discharge out to POC#2 and across 6th Ave.

For standard details of Modular Wetland system used for BMP 1-1, please refer to Attachment 3. For manual on operation and maintenance please also refer to Attachment 3.

Project Name: Scripps Mercy Hospital

Form I-6 Page 3 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 1-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Project Name: Scripps Mercy Hospital

Form I-6 Page 4 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 1-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): The Modular Wetland sytem implements biofiltration technology via filtration through a patented soil, WetlandMEDIA. Unlike conventional bioplanters, Modular Wetland utilizes horizontal flow to filter runoff through soil. As treated runoff exits the system, the outflow is controlled by the outflow riser. For additional details refer to Attachment 3. The Modular Wetland system planter with a separate storage tank was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Form I-6 Page 5 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 2-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Form I-6 Page 6 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 2-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Project Name: Scripps Mercy Hospital

Form I-6 Page 7 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 3-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Form I-6 Page 8 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 3-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Form I-6 Page 9 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 4-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Project Name: Scripps Mercy Hospital

Form I-6 Page 10 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 4-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Form I-6 Page 11 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 5-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Form I-6 Page 12 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 5-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Project Name: Scripps Mercy Hospital

Form I-6 Page 13 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 6A-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Form I-6 Page 14 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 6A-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Form I-6 Page 15 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 6B-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Project Name: Scripps Mercy Hospital

Form I-6 Page 16 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 6B-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Form I-6 Page 17 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 6C-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Project Name: Scripps Mercy Hospital

Form I-6 Page 18 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 6C-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Project Name: Scripps Mercy Hospital

Form I-6 Page 19 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 7-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Project Name: Scripps Mercy Hospital

Form I-6 Page 20 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 7-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Form I-6 Page 21 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 8-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Form I-6 Page 22 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 8-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

Project Name: Scripps Mercy Hospital

Form I-6 Page 23 of 24 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 9-1	
Construction Plan Sheet No. CUP-11	
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	Jeffrey Gavazza 213-418-0201
Who will be the final owner of this BMP?	Scripps Health
Who will maintain this BMP into perpetuity?	Scripps Health
What is the funding mechanism for maintenance?	Scripps Health

Form I-6 Page 24 of 24 (Copy as many as needed)
Structural BMP ID No. BMP 9-1
Construction Plan Sheet No. CUP-11
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Biofiltration planter was sized using the San Diego Regional Model BMP Design Manual BMP sizing spreadsheet V3.0 for hydromodification compliance. Please refer to Attachment 2.

BMP 1-1

Compact (high rate) Biofiltration BMP Checklist		Form I-10
Compact (high rate) biofiltration BMPs have a media filtration rate greater than 5 in/hr. and a media surface area smaller than 3% of contributing area times adjusted runoff factor. Compact biofiltration BMPs are typically proprietary BMPs that may qualify as biofiltration. A compact biofiltration BMP may satisfy the pollutant control requirements for a DMA onsite in some cases. This depends on the characteristics of the DMA **and** the performance certification/data of the BMP. If the pollutant control requirements for a DMA are met onsite, then the DMA is not required to participate in an offsite storm water alternative compliance program to meet its pollutant control obligations. An applicant using a compact biofiltration BMP to meet the pollutant control requirements onsite must complete Section 1 of this form and include it in the PDP SWQMP. A separate form must be completed for each DMA. In instances where the City Engineer does not agree with the applicant's determination, Section 2 of this form will be completed by the City and returned to the applicant.		
Section 1: Biofiltration Criteria Checklist (Appendix F)		
Refer to Part 1 of the Storm Water Standards to complete this section. When separate forms/worksheets are referenced below, the applicant must also complete these separate forms/worksheets (as applicable) and include in the PDP SWQMP. The criteria numbers below correspond to the criteria numbers in Appendix F.		
Criteria	**Answer**	**Progression**
Criteria 1 and 3: What is the infiltration condition of the DMA? Refer to Section 5.4.2 and Appendix C of the BMP Design Manual (Part 1 of Storm Water Standards) for guidance. Applicant must complete and include the following in the PDP SWQMP submittal to support the feasibility determination: • Infiltration Feasibility Condition Letter; or • Worksheet C.4-1: Form I-8A and Worksheet C.4-2: Form I-8B. Applicant must complete and include all applicable sizing worksheets in the SWQMP submittal	○ Full Infiltration Condition	**Stop**. Compact biofiltration BMP is not allowed.
	○ Partial Infiltration Condition	Compact biofiltration BMP is only allowed, if the target volume retention is met onsite (Refer to Table B.5-1 in Appendix B.5). Use Worksheet B.5-2 in Appendix B.5 to estimate the target volume retention (Note: retention in this context means reduction). If the required volume reduction is achieved **proceed to Criteria 2**. If the required volume reduction is not achieved, compact biofiltration BMP is not allowed. **Stop**.
	◉ No Infiltration Condition	Compact biofiltration BMP is allowed if volume retention criteria in Table B.5-1 in Appendix B.5 for the no infiltration condition is met. Compliance with this criterion must be documented in the PDP SWQMP. If the criteria in Table B.5-1 is met **proceed to Criteria 2**. If the criteria in Table B.5-1 is not met, compact biofiltration BMP is not allowed. **Stop**.

Compact (high rate) Biofiltration BMP Checklist	Form I-10

Provide basis for Criteria 1 and 3:

<u>Feasibility Analysis:</u>

Summarize findings and include either infiltration feasibility condition letter or Worksheet C.4-1: Form I-8A and Worksheet C.4-2: Form I-8B in the PDP SWQMP submittal.

<u>If Partial Infiltration Condition:</u>

Provide documentation that target volume retention is met (include Worksheet B.5-2 in the PDP SWQMP submittal). Worksheet B.5-7 in Appendix B.5 can be used to estimate volume retention benefits from landscape areas.

<u>If No Infiltration Condition:</u>

Provide documentation that the volume retention performance standard is met (include Worksheet B.5-2 in the PDP SWQMP submittal) in the PDP SWQMP submittal. Worksheet B.5-6 in Appendix B.5 can be used to document that the performance standard is met.

Documentation will be provided to show the target volume retention is met. Please refer to Worksheet B.5-2 and B.5-6 in Attachment 1e.

Criteria	Answer	Progression
<u>Criteria 2:</u> Is the compact biofiltration BMP sized to meet the performance standard from the MS4 Permit? Refer to Appendix B.5 and Appendix F.2 of the BMP Design Manual (Part 1 of Storm Water Standards) for guidance.	○ Meets Flow based Criteria	Use guidance from **Appendix F.2.2** to size the compact biofiltration BMP to meet the flow based criteria. Include the calculations in the PDP SWQMP. Use parameters for sizing consistent with manufacturer guidelines and conditions of its third party certifications (i.e. a BMP certified at a loading rate of 1 gpm/sq. ft. cannot be designed using a loading rate of 1.5 gpm/sq. ft.) **Proceed to Criteria 4.**
	◉ Meets Volume based Criteria	Provide documentation that the compact biofiltration BMP has a total static (i.e. non-routed) storage volume, including pore-spaces and pre-filter detention volume (Refer to Appendix B.5 for a schematic) of at least 0.75 times the portion of the DCV not reliably retained onsite. **Proceed to Criteria 4.**
	○ Does not Meet either criteria	**Stop**. Compact biofiltration BMP is not allowed.

Compact (high rate) Biofiltration BMP Checklist	Form I-10

Provide basis for Criteria 2:

Provide documentation that the BMP meets the numeric criteria and is designed consistent with the manufacturer guidelines and conditions of its third-party certification (i.e., loading rate, etc., as applicable).

Criteria	Answer	Progression
Criteria 4: Does the compact biofiltration BMP meet the pollutant treatment performance standard for the projects most significant pollutants of concern? Refer to Appendix B.6 and Appendix F.1 of the BMP Design Manual (Part 1 of Storm Water Standards) for guidance.	◉ Yes, meets the TAPE certification.	Provide documentation that the compact BMP has an appropriate TAPE certification for the projects most significant pollutants of concern. **Proceed to Criteria 5.**
	○ Yes, through other third-party documentation	Acceptance of third-party documentation is at the discretion of the City Engineer. The City engineer will consider, (a) the data submitted; (b) representativeness of the data submitted; and (c) consistency of the BMP performance claims with pollutant control objectives in Table F.1-2 and Table F.1-1 while making this determination. If a compact biofiltration BMP is not accepted, a written explanation/ reason will be provided in Section 2. **Proceed to Criteria 5.**
	○ No	**Stop**. Compact biofiltration BMP is not allowed.

Provide basis for Criteria 4:

Provide documentation that identifies the projects most significant pollutants of concern and TAPE certification or other third party documentation that shows that the compact biofiltration BMP meets the pollutant treatment performance standard for the projects most significant pollutants of concern.

Refer to Attachment 3 for product information of Modular Wetland Systems.

Compact (high rate) Biofiltration BMP Checklist		Form I-10
Criteria	**Answer**	**Progression**
Criteria 5: Is the compact biofiltration BMP designed to promote appropriate biological activity to support and maintain treatment process? Refer to Appendix F of the BMP Design Manual (Part 1 of Storm Water Standards) for guidance.	◉ Yes	Provide documentation that the compact biofiltration BMP support appropriate biological activity. Refer to Appendix F for guidance. **Proceed to Criteria 6.**
	○ No	**Stop**. Compact biofiltration BMP is not allowed.

Provide basis for Criteria 5:

Provide documentation that appropriate biological activity is supported by the compact biofiltration BMP to maintain treatment process.

Refer to the operations and maintenance manual of Modular Wetland Systems in Attachment 3.

Criteria	**Answer**	**Progression**
Criteria 6: Is the compact biofiltration BMP designed with a hydraulic loading rate to prevent erosion, scour and channeling within the BMP?	◉ Yes	Provide documentation that the compact biofiltration BMP is used in a manner consistent with manufacturer guidelines and conditions of its third-party certification. **Proceed to Criteria 7.**
	○ No	**Stop**. Compact biofiltration BMP is not allowed.

Provide basis for Criteria 6:

Provide documentation that the BMP meets the numeric criteria and is designed consistent with the manufacturer guidelines and conditions of its third-party certification (i.e., maximum tributary area, maximum inflow velocities, etc., as applicable).

Compact (high rate) Biofiltration BMP Checklist		Form I-10
Criteria	**Answer**	**Progression**
Criteria 7: Is the compact biofiltration BMP maintenance plan consistent with manufacturer guidelines and conditions of its third-party certification (i.e., maintenance activities, frequencies)?	◉ Yes, and the compact BMP is privately owned, operated and not in the public right of way.	Submit a maintenance agreement that will also include a statement that the BMP will be maintained in accordance with manufacturer guidelines and conditions of third-party certification. **Stop**. The compact biofiltration BMP meets the required criteria.
	○ Yes, and the BMP is either owned or operated by the City or in the public right of way.	Approval is at the discretion of the City Engineer. The city engineer will consider maintenance requirements, cost of maintenance activities, relevant previous local experience with operation and maintenance of the BMP type, ability to continue to operate the system in event that the vending company is no longer operating as a business or other relevant factors while making the determination. **Stop**. Consult the City Engineer for a determination.
	○ No	**Stop**. Compact biofiltration BMP is not allowed.

Provide basis for Criteria 7:

Include copy of manufacturer guidelines and conditions of third-party certification in the maintenance agreement. PDP SWQMP must include a statement that the compact BMP will be maintained in accordance with manufacturer guidelines and conditions of third-party certification.

Compact (high rate) Biofiltration BMP Checklist	Form I-10
Section 2: Verification (For City Use Only)	
Is the proposed compact BMP accepted by the City Engineer for onsite pollutant control compliance for the DMA?	○ Yes ○ No, See explanation below
Explanation/reason if the compact BMP is not accepted by the City for onsite pollutant control compliance:	

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Attachment 1
Backup For PDP Pollutant Control BMPs

This is the cover sheet for Attachment 1.

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Project Name: Scripps Mercy Hospital

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: o Infiltration Feasibility Condition Letter (*Note: must be stamped and signed by licensed geotechnical engineer*) o Form I-8A (optional) o Form I-8B (optional) • Partial Infiltration Condition: o Infiltration Feasibility Condition Letter (*Note: must be stamped and signed by licensed geotechnical engineer*) o Form I-8A o Form I-8B • Full Infiltration Condition: o Form I-8A o Form I-8B o Worksheet C.4-3 o 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

Project Name: Scripps Mercy Hospital

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

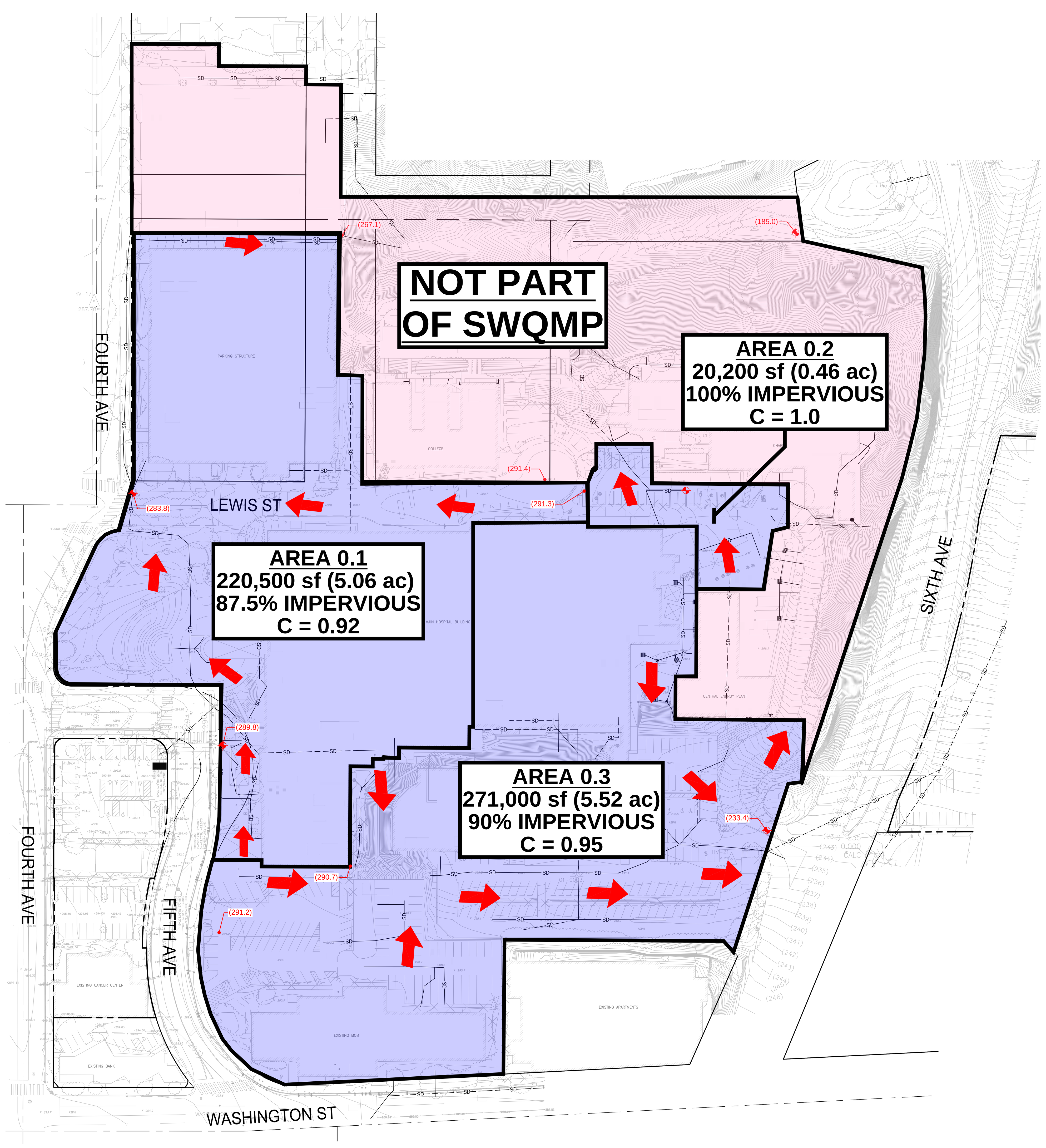
The DMA Exhibit must identify:

- ☑ Underlying hydrologic soil group
- ☑ Approximate depth to groundwater
- ☑ Existing natural hydrologic features (watercourses, seeps, springs, wetlands)
- ☑ Critical coarse sediment yield areas to be protected
- ☑ Existing topography and impervious areas
- ☑ 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
- ☑ 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)
- ☑ Structural BMPs (identify location, type of BMP, size/detail, and include cross-section)

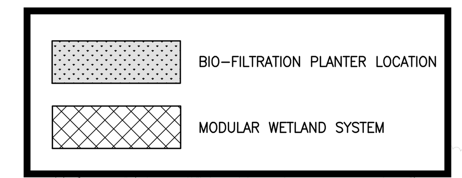
Attachment 1a: DMA Exhibit

BMP selected: Biofiltration (BF-1 & BF-3 w/ Cisterns)

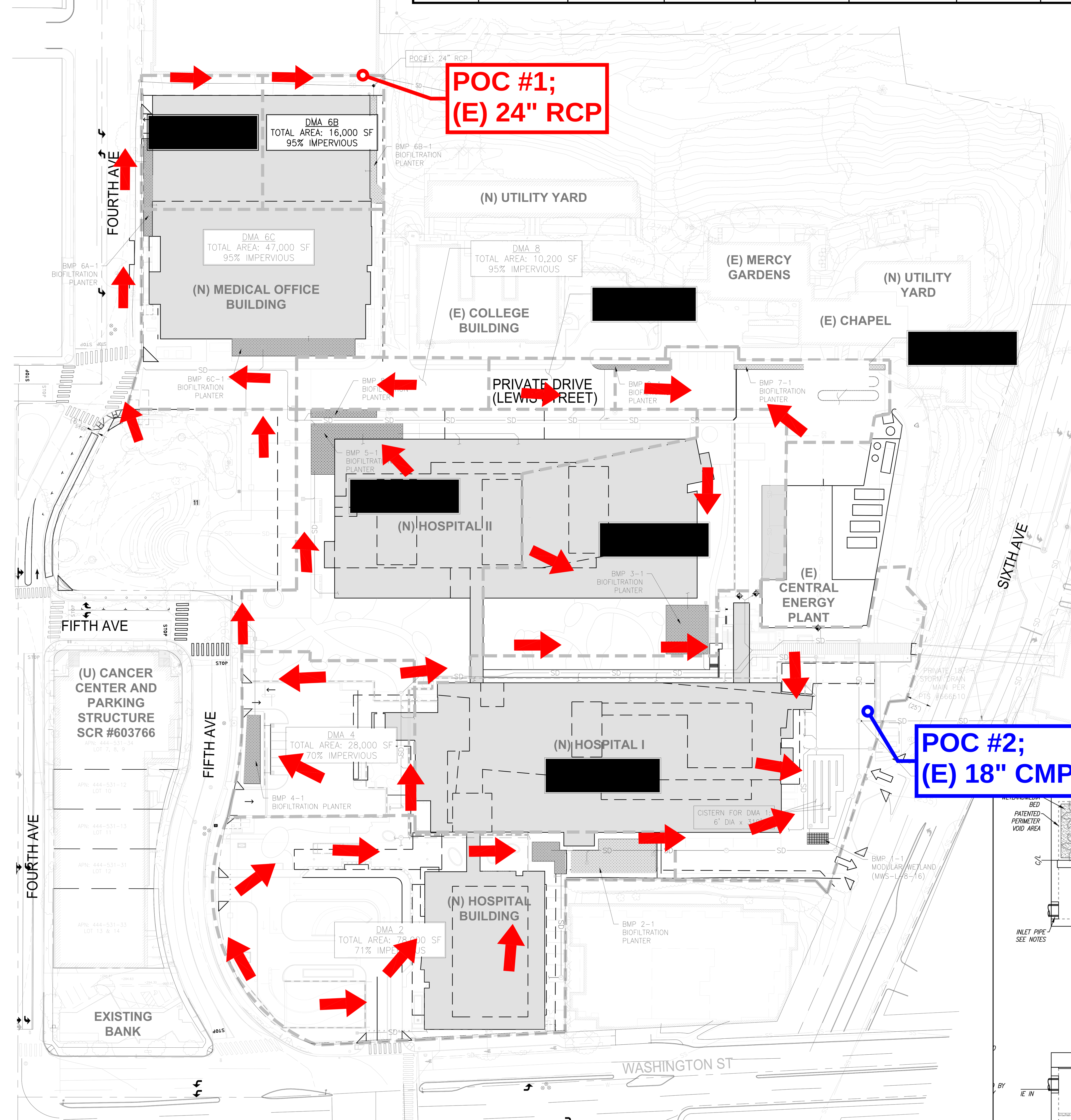
Existing Condition

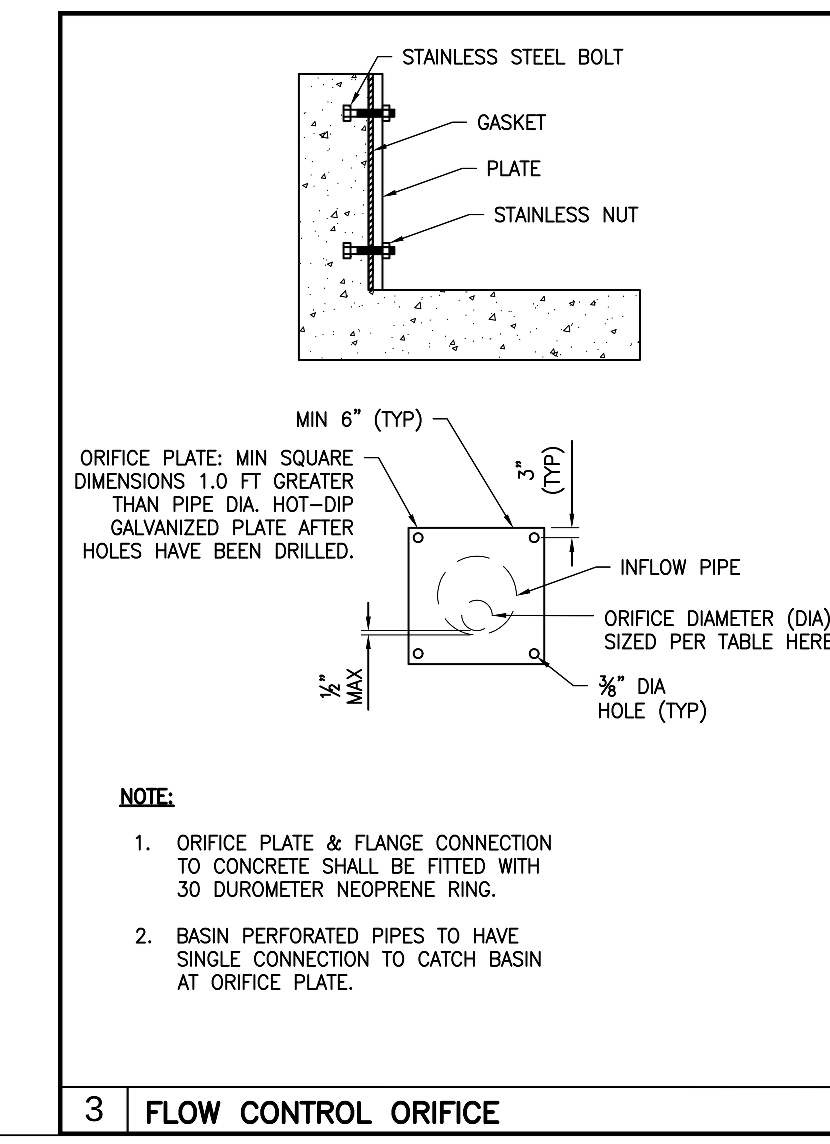


Proposed Condition

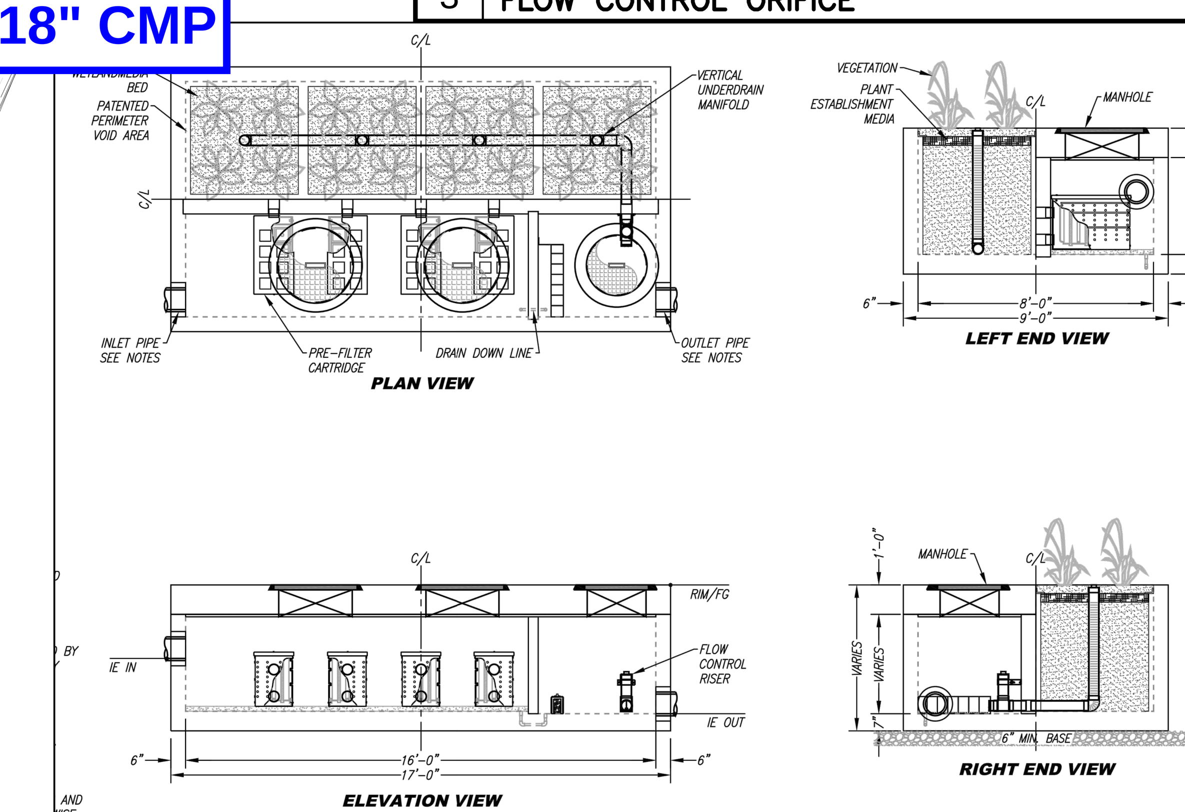


DMA	AREA (AC.)	IMPERVIOUS%	REQ. BMP CAPACITY*	PROP. BMP CAPACITY	BMP ID	ORIFICE SIZE (IN.)	MAX ORIFICE OUFLOW (CFS)	DRAINS TO (POC ID)
DMA 1	2.39	92	8,712 CF	8,760 CF	BMP 1–1	3 1/8"	0.520	#2
DMA 2	1.79	72	2,888 SF	2,900 SF	BMP 2–1	2 1/2"	0.344	#2
DMA 3	1.29	75	1,895 SF	1,900 SF	BMP 3–1	2 1/8"	0.249	#2
DMA 4	0.64	71	970 SF	1,000 SF	BMP 4–1	1 1/2"	0.124	#1
DMA 5	1.52	71	2,190 SF	2,200 SF	BMP 5–1	2 3/8"	0.311	#1
DMA 6A	0.32	95	628 SF	630 SF	BMP 6A–1	1"	0.055	#1
DMA 6B	0.37	95	724 SF	730 SF	BMP 6B–1	1 1/8"	0.065	#1
DMA 6C	1.08	95	2,098 SF	2,100 SF	BMP 6C–1	2"	0.220	#1
DMA 7	0.36	91	712 SF	715 SF	BMP 7–1	1 1/8"	0.070	#1
DMA 8	0.24	95	493 SF	495 SF	BMP 8–1	7/8"	0.042	#1
DMA 9	0.16	90	319 SF	325 SF	BMP 9–1	3/4"	0.031	#1



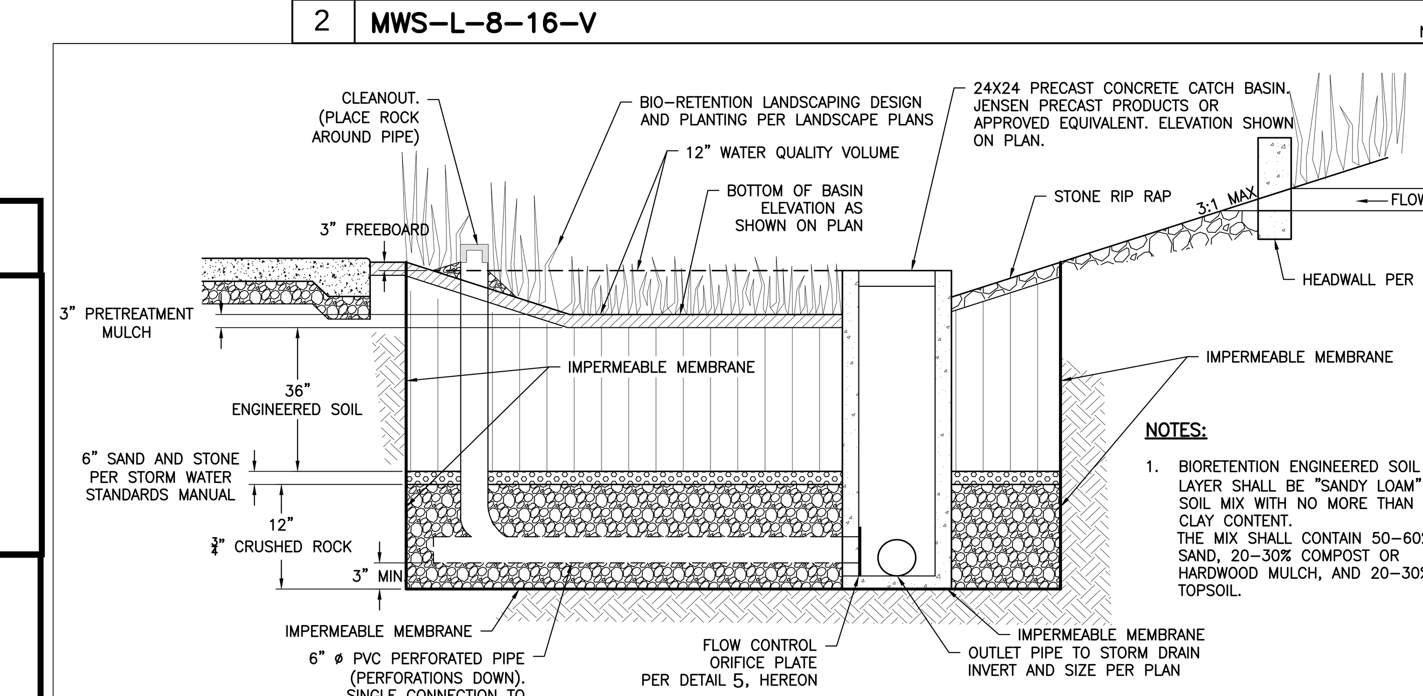


3 FLOW CONTROL ORIFICE



2 MWS–L–8–16–V

N.T.S.



1 TYPICAL BIO-FILTRATION SECTION

N.T.S.

Existing Natural Hydrologic Features

CCSYA: Not Applicable

Appx. Depth to Groundwater: +20 feet
- Boring tests show refusal from 20 to 21 ft. deep
- Only one out of 5 boring logs show groundwater encounter; the perched grounwater table was detected at 21 ft below surface.

Underlying HSG: Group D

Underlying HSG N/A, but judging from SM (silty sand) below artificial fill and infiltration rate ~0" in/hr, HGS Group D was selected, per San Diego Stormwater Design Manual.

Attachment 1b: Summary of DMAs

Project Name: Scripps Mercy Hospital

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	2.39	2.20	92	D	0.84	4287	1-1	Biofiltration	#2
DMA 2	1.79	1.29	72	D	0.67	2586	2-1	Biofiltration	#2
DMA 3	1.29	0.96	75	D	0.70	1,927	3-1	Biofiltration	#2
DMA 4	0.64	0.46	71	D	0.67	924	4-1	Biofiltration	#1
DMA 5	1.52	1.08	71	D	0.67	2,173	5-1	Biofiltration	#1
DMA 6A,B,C	1.77	1.42	95	D	0.85	3,112	6A-1, 6B-1, 6C-1	Biofiltration	#1
DMA 7	0.356	0.324	91	D	0.83	631	7-1	Biofiltration	#1
DMA 8	0.236	0.225	95	D	0.86	436	8-1	Biofiltration	#1
DMA 9	0.161	0.145	90	D	0.82	282	9-1	Biofiltration	#1
SEE DMA EXHIBIT FOR ADDITIONAL DETAILS ON DMA 6A, 6B, AND 6C									
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
11	10.56	8.35	0.791		0.733	16,474	10.56		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

Attachment 1c

Harvest and Use Feasibility Checklist		Worksheet B.3-1 : Form I-7
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] Proposed Landscape Area (A) = 96,180 SF / 43,560 = 2.21 acres Use 1,470 gallons/acre/(36 hours) for Moderate Plant Water Use per table B.3-3: 1,470 gallons/acre/(36 hours) / (7.48 CF/gallon) = 196 CF/acre/(36 hours) Demand = (2.21 acres) x (196) = 433 CF/(36 hours)		
3. Calculate the DCV using worksheet B-2.1. DCV = 16,474 (cubic feet) [Provide a summary of calculations here] For all of the DMAs combined, DCV = C × d × A × 43,560 sf⁄ac × 1/12 ft/in C = 0.733 d = 0.59 A = 10.56 acres DCV = (0.733) × (0.59 in) × (10.56 acres) × (3,630 CF⁄(ac·in)) = 16,474 CF		
3a. Is the 36-hour demand greater than or equal to the DCV? ☐ Yes / ☑ No ⇨ ⇩	3b. Is the 36-hour demand greater than 0.25DCV but less than the full DCV? ☐ Yes / ☑ No ⇨ ⇩	3c. Is the 36-hour demand less than 0.25DCV? ☑ 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.
Is harvest and use feasible based on further evaluation? ☐ Yes, refer to Appendix E to select and size harvest and use BMPs. ☑ No, select alternate BMPs.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
DMA 2		Preliminary Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ⦿ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ◯ Yes; continue to Step 1F. ◯ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ◯ Yes; continue to Step 1G. ◯ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ◯ Yes; answer "Yes" to Criteria 1 Result. ◯ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ◯ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ⦿ No; full infiltration is not required. Skip to Part 1 Result.
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.01 inches per hour.	

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	◯Yes	◯No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◯Yes	◯No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◯Yes	◯No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◯Yes	◯No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◯Yes	◯No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	○ Yes	○ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Summarize findings and basis; provide references to related reports or exhibits.

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	○ Full infiltration Condition ⦿ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:
DMA 2	
Criteria 3 : Infiltration Rate Screening	

3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⦿ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ⦿ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ⦿ No: Skip to Part 2 Result.

Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).

Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.01 inches per hour.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	○ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.	

Part 2 – Partial Infiltration Geotechnical Screening Result[5]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	○ Partial Infiltration Condition ◉ No Infiltration Condition

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]	Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
DMA 2	Preliminary Phase
Criteria 1: Infiltration Rate Screening	

1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ◉ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ○ Yes; continue to Step 1F. ○ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ○ Yes; continue to Step 1G. ○ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ○ Yes; answer "Yes" to Criteria 1 Result. ○ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ○ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ◉ No; full infiltration is not required. Skip to Part 1 Result.

Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.

Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.03 inches per hour.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	○ Yes	○ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions			Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	○ Yes	○ No	
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No	
Summarize findings and basis; provide references to related reports or exhibits.				

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	○ Full infiltration Condition ◉ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
DMA 2		
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⊙ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ⊙ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ⊙ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.03 inches per hour.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	○ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.	

Part 2 – Partial Infiltration Geotechnical Screening Result[5]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	○ Partial Infiltration Condition ◉ No Infiltration Condition

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

Percolation Test Data Sheet

Project: Mercy Replacement Hospital

Project No: 20194096.001A

Tested By: Sean Hanrahan
Date: 5/19/2019
Checked By: S.Tena

Borehole ID: **PERC-1**
Depth of Borehole: 5 feet
Diameter of Borehole: 8 inches
USCS Soil Classification: SC
PVC Pipe Hieght above Surface 0.2 ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	6:43	7:13	30	0.79	0.84	0.05	50.00
2	7:13	7:43	30	0.79	0.80	0.01	250.00
3	7:43	8:13	30	0.79	0.81	0.02	125.00
4	8:13	8:43	30	0.79	0.81	0.02	125.00
5	8:43	9:13	30	0.79	0.81	0.02	125.00
6	9:13	9:43	30	0.79	0.80	0.01	250.00
7	9:43	10:13	30	0.79	0.81	0.02	125.00
8	10:13	10:43	30	0.79	0.81	0.02	125.00
9	10:43	11:13	30	0.79	0.81	0.02	125.00
10	11:13	11:43	30	0.79	0.81	0.02	125.00
11	11:43	12:13	30	0.79	0.82	0.03	83.33
12	12:13	12:43	30	0.79	0.81	0.02	**125.00**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t\ (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	50.50	inches
Hf	50.26	inches
ΔH	0.24	inches
Havg	50.38	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.02**	in/hr

Percolation Test Data Sheet

Project: Mercy Replacement Hospital

Tested By: Sean Hanrahan
Date: 5/19/2019
Checked By: S.Tena

Project No: 20194096.001A

Borehole ID: **PERC-2**
Depth of Borehole: 5 feet
Diameter of Borehole: 8 inches
USCS Soil Classification: SC
PVC Pipe Hieght above Surface 0.7 ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	6:47	7:17	30	0.33	0.42	0.09	27.78
2	7:17	7:47	30	0.33	0.42	0.09	27.78
3	7:47	8:17	30	0.33	0.39	0.06	41.67
4	8:17	8:47	30	0.33	0.39	0.06	41.67
5	8:47	9:17	30	0.33	0.39	0.06	41.67
6	9:17	9:47	30	0.33	0.39	0.06	41.67
7	9:47	10:17	30	0.33	0.38	0.05	50.00
8	10:17	10:47	30	0.33	0.39	0.06	41.67
9	10:47	11:17	30	0.33	0.39	0.06	41.67
10	11:17	11:47	30	0.33	0.39	0.06	41.67
11	11:47	12:17	30	0.33	0.38	0.05	50.00
12	12:17	12:47	30	0.33	0.39	0.06	**41.67**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H \; 60 \; r}{\Delta t \; (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	56.00	inches
Hf	55.28	inches
ΔH	0.72	inches
Havg	55.64	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.05**	in/hr

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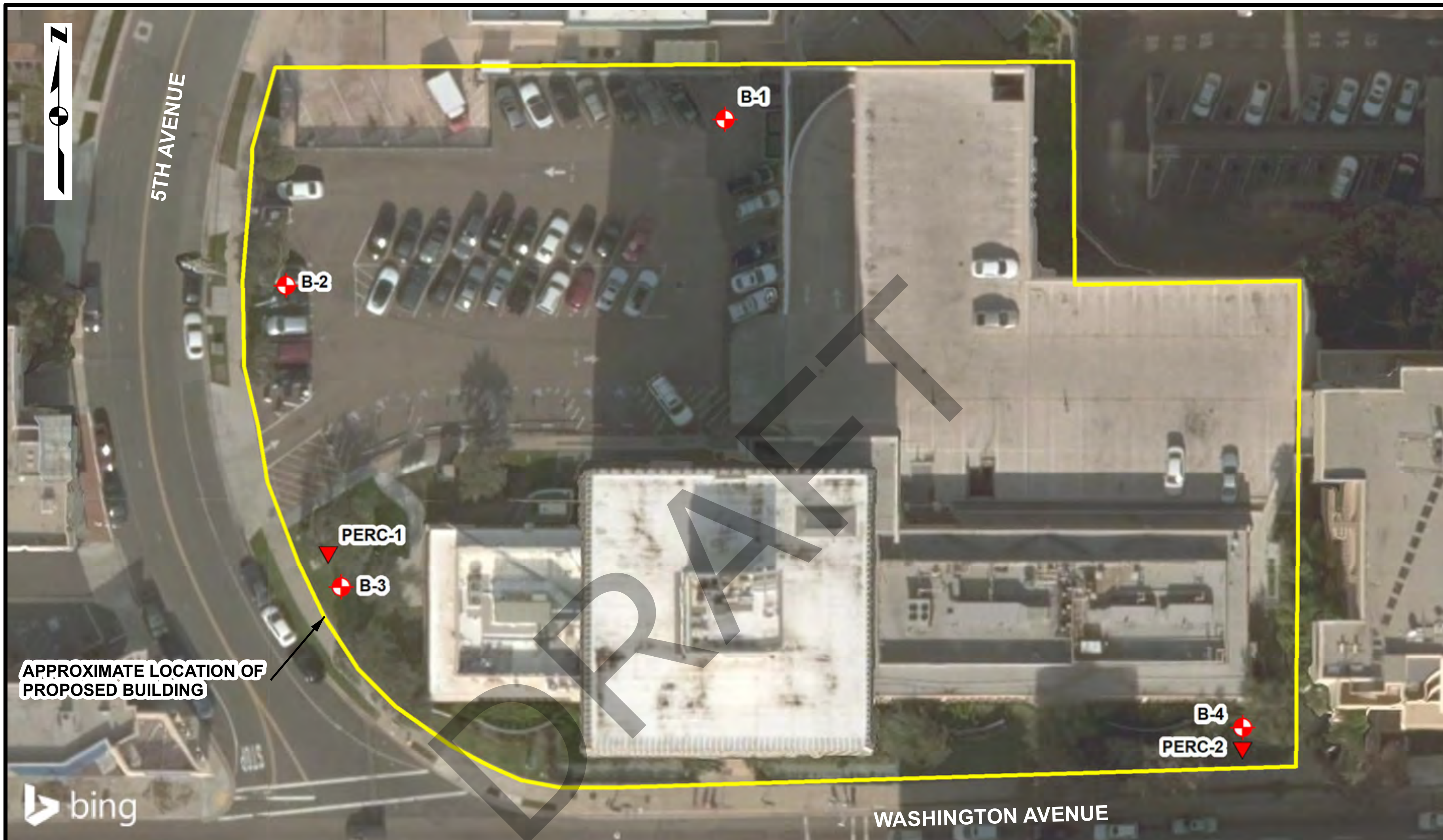

LEGEND

APPROXIMATE LOCATION OF BORING

APPROXIMATE LOCATION OF PERCOLATION TEST

0 25 50 100 Feet

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PROJECT NO. 20194096	EXPLORATION MAP	FIGURE
DRAWN: 5/22/2019		
DRAWN BY: JP		2
CHECKED BY: ST	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	
FILE NAME: 20194096_Site_Merc.mxd		

HSPT I
PERC-1

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
DMA 1		Planning Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ⊙ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ○ Yes; continue to Step 1F. ○ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ○ Yes; continue to Step 1G. ○ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ○ Yes; answer "Yes" to Criteria 1 Result. ○ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ○ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ⦿ No; full infiltration is not required. Skip to Part 1 Result.

Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.

One borehole percolation test was performed and the unfactored infiltration rate was 0.22 inches per hour. The soils at the location were described as silty sand with gravel.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	◯ Yes	◉ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◯ Yes	◉ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◯ Yes	◉ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◯ Yes	◯ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◯ Yes	◯ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	○ Yes	● No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	● Yes	○ No
Summarize findings and basis; provide references to related reports or exhibits. One borehole percolation test was performed and the unfactored infiltration rate was 0.22 inches per hour. The soils at the location were described as silty sand with gravel.			

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	○ Full infiltration Condition ◉ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:
DMA 1	Planning Phase
Criteria 3 : Infiltration Rate Screening	

3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⦿ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ⦿ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ○ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ⦿ Yes; Continue to Criteria 4. ○ No: Skip to Part 2 Result.

Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).

One borehole percolation test was performed and the unfactored infiltration rate was 0.22 inches per hour.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	◉ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◉ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◉ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◉ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◉ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	◉ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	◉ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	◉ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	◉ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	◉ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	◉ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits. One borehole percolation test was performed and the unfactored infiltration rate was 0.22 inches per hour.		
Part 2 – Partial Infiltration Geotechnical Screening Result[5]		**Result**
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.		◉ Partial Infiltration Condition ○ No Infiltration Condition

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

Percolation Test Data Sheet

Project:	Mercy Replacement Hospital	Tested By:	Salvador Tena
		Date:	4/27/2018
Project No:	20183768.001A	Checked By:	

Borehole ID:	**PERC-1**	
Depth of Borehole:	5	feet
Diameter of Borehole:	8	inches
USCS Soil Classification:	SM	
PVC Pipe Hieght above Surface	0	ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	8:48	9:18	30	1.87	1.93	0.06	41.67
2	9:18	9:48	30	1.93	2.00	0.07	35.71
3	9:48	10:18	30	2.00	2.15	0.15	16.67
4	10:18	10:48	30	2.15	2.46	0.31	8.06
5	10:48	11:18	30	1.95	1.97	0.02	125.00
6	11:18	11:48	30	1.97	2.09	0.12	20.83
7	11:48	12:18	30	2.09	3.25	1.16	2.16
8	12:18	12:48	30	1.85	1.92	0.07	35.71
9	12:48	13:18	30	1.92	1.99	0.07	35.71
10	13:18	13:48	30	1.99	2.10	0.11	22.73
11	13:48	14:18	30	2.10	3.24	1.14	2.19
12	14:18	14:48	30	2.03	2.20	0.17	**14.71**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H \; 60 \; r}{\Delta t \; (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	35.64	inches
Hf	33.60	inches
ΔH	2.04	inches
Havg	34.62	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.22**	in/hr

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
DMA 1		Planning Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ○ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ⦿ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ⦿ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ⦿ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ○ Yes; continue to Step 1F. ○ No; conduct appropriate number of tests.	
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ○ Yes; continue to Step 1G. ○ No; select appropriate factor of safety.	
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ○ Yes; answer "Yes" to Criteria 1 Result. ○ No; answer "No" to Criteria 1 Result.	
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ⊙ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ○ No; full infiltration is not required. Skip to Part 1 Result.	
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. One borehole percolation test was performed and the unfactored infiltration rate was 2.47 inches per hour.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]		
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	◉ Yes	○ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◉ Yes	○ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◉ Yes	○ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◉ Yes	○ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◉ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	◉ Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	◉ Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	◉ Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	◉ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	◉ Yes	○ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	◉ Yes	○ No

Summarize findings and basis; provide references to related reports or exhibits.

One borehole percolation test was performed and the unfactored infiltration rate was 2.47 inches per hour.

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	◉ Full infiltration Condition ○ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
DMA 1		Planning Phase
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ◯ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ◯ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ◯ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ◯ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ◯ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ◯ Yes; Continue to Criteria 4. ◯ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	◯ Yes	◯ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◯ Yes	◯ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◯ Yes	◯ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◯ Yes	◯ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◯ Yes	◯ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	◯ Yes	◯ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.		
Part 2 – Partial Infiltration Geotechnical Screening Result[5]		**Result**
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.		○ Partial Infiltration Condition ○ No Infiltration Condition

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

Factor of Safety and Design Infiltration Rate Worksheet				Worksheet D.5-1: Form I-9	
Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
A	Suitability Assessment	Soil assessment methods	0.25	2	0.50
		Predominant soil texture	0.25	3	0.75
		Site soil variability	0.25	2	0.50
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			2.00
B	Design	Level of pretreatment/ expected sediment loads	0.5	1	0.50
		Redundancy/resiliency	0.25	2	0.50
		Compaction during construction	0.25	1	0.25
		Design Safety Factor, $S_B = \Sigma p$			1.25
Combined Safety Factor, $S_{total} = S_A \times S_B$ [Minimum of 2 and Maximum of 9]				2.50	
Observed Infiltration Rate, inch/hr., $K_{observed}$ (corrected for test-specific bias) Note: This worksheet is only applicable when the observed infiltration rate is greater than or equal to 1 inch/hr.				2.50	
Design Infiltration Rate, in/hr., $K_{design} = K_{observed} / S_{total}$ Note: If the estimated design infiltration rate is less than or equal to 0.5 inch/hr. then the applicant may choose to implement partial infiltration BMPs.				1.00	
Supporting Data					
Briefly describe infiltration test and provide reference to test forms:					

Note: Worksheet D.5-1: Form I-9 is only applicable to design BMPs in "full infiltration condition". This form is not applicable for categorization of infiltration feasibility (Worksheet C.4-1: Form I-8) and/or for designing BMPs in "partial infiltration condition" or "no infiltration condition".

Percolation Test Data Sheet

Project:		Tested By:	Salvador Tena
	Mercy Replacement Hospital	Date:	4/27/2018
Project No:	20183768.001A	Checked By:	

Borehole ID:	**PERC-2**	
Depth of Borehole:	5	feet
Diameter of Borehole:	8	inches
USCS Soil Classification:	SM	
PVC Pipe Hieght above Surface	0	ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	9:45	9:55	10	2.32	3.60	1.28	0.65
2	9:55	10:05	10	2.18	2.96	0.78	1.07
3	10:05	10:15	10	2.06	2.77	0.71	1.17
4	10:15	10:25	10	2.14	2.85	0.71	1.17
5	10:25	10:35	10	2.05	2.97	0.92	0.91
6	10:39	10:49	10	2.10	3.12	1.02	0.82
7	10:49	10:59	10	2.01	2.60	0.59	1.41
8				0.00	0.00	0	NA
9				0.00	0.00	0	NA
10				0.00	0.00	0	NA
11				0.00	0.00	0	NA
12				0.00	0.00	0	**NA**

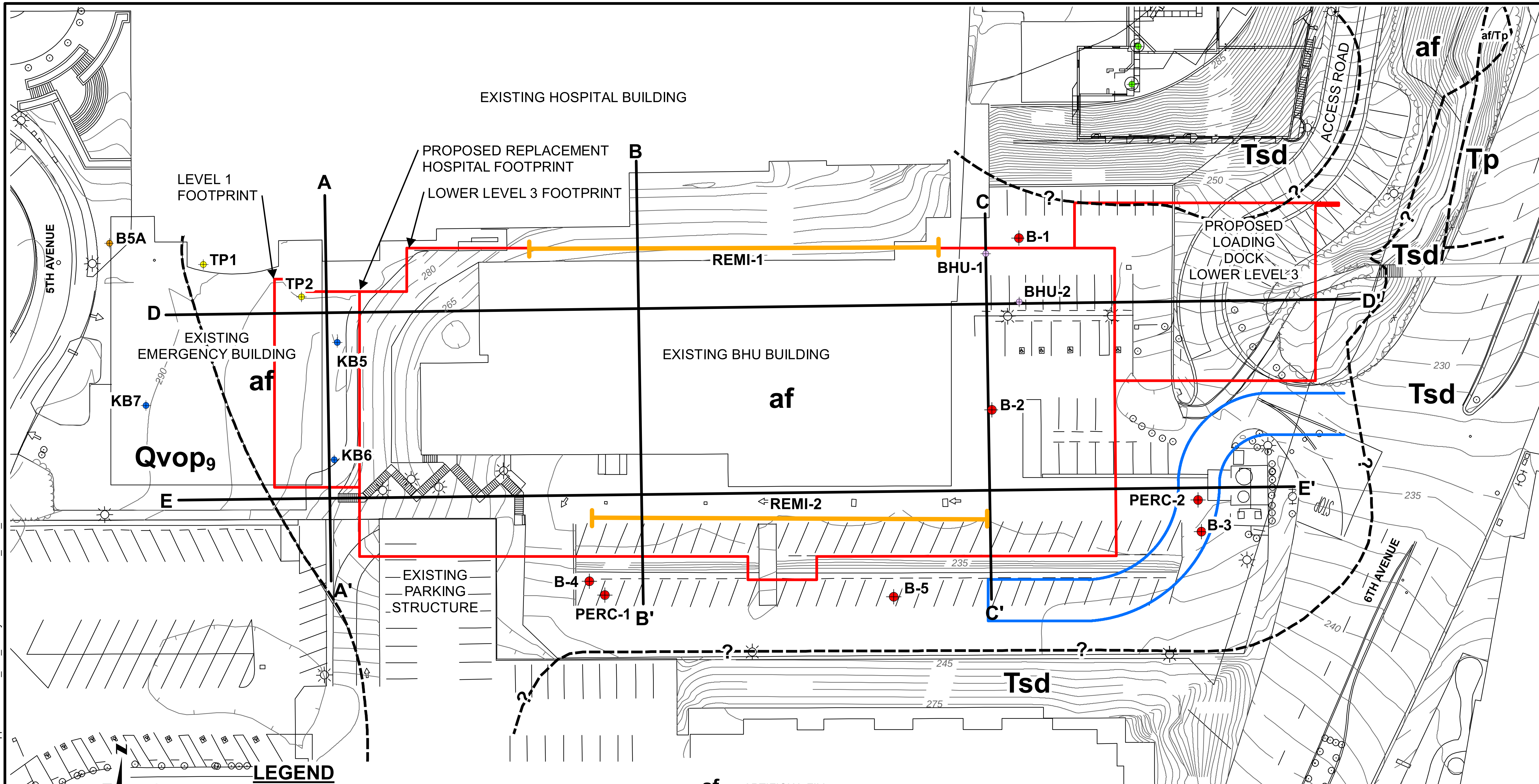
Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H \; 60 \; r}{\Delta t \; (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	35.88	inches
Hf	28.80	inches
ΔH	7.08	inches
Havg	32.34	inches
Δt	10.00	minutes
r	4.00	inches
It	**2.47**	in/hr



LEGEND

- APPROXIMATE BORING LOCATION, KLEINFELDER 2018
- APPROXIMATE BORING LOCATION, KLEINFELDER 2006
- APPROXIMATE BORING LOCATION, KLEINFELDER 2005
- APPROXIMATE BORING LOCATION, BENTON 1994
- APPROXIMATE BORING LOCATION, WOODWARD CLYDE 1986
- APPROXIMATE BORING LOCATION, BENTON 1977
- APPROXIMATE LOCATION OF CROSS SECTION, KLEINFELDER 2018
- APPROXIMATE LOCATION OF ReMi LINE
- PROPOSED REPLACEMENT HOSPITAL FOOTPRINT
- PROPOSED FIRE LANE

af	ARTIFICIAL FILL
$Qvop_9$	VERY OLD PARALIC DEPOSITS
Tsd	SAN DIEGO FORMATION
Tp	POMERADO CONGLOMERATE
- - -	GEOLOGIC CONTACT

N

0 25 50 100 Feet

\\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20183768_001_Mercy\mxd\20183768_Site2_Merc.mxd

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PROJECT NO.	20183768
DRAWN:	4/22/2019
DRAWN BY:	JP
CHECKED BY:	KC
FILE NAME:	20183768_Site2_Merc.mxd

EXPLORATION MAP

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

2

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
DMA 6A, 6B, 6C, 5		Preliminary Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ⦿ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ◯ Yes; continue to Step 1F. ◯ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ◯ Yes; continue to Step 1G. ◯ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ◯ Yes; answer "Yes" to Criteria 1 Result. ◯ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ◯ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ◉ No; full infiltration is not required. Skip to Part 1 Result.
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.02 inches per hour.	

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	◯ Yes	◯ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◯ Yes	◯ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◯ Yes	◯ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◯ Yes	◯ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◯ Yes	◯ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions			Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	◯ Yes	◯No	
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	◯Yes	◯ No	
Summarize findings and basis; provide references to related reports or exhibits.				

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	◯ Full infiltration Condition ⦿ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:
DMA 6A, 6B, 6C, 5	
Criteria 3 : Infiltration Rate Screening	

3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⊙ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ⊙ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ⊙ No: Skip to Part 2 Result.

Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).

Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.02 inches per hour.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	◯ Yes	◯ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◯ Yes	◯ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◯ Yes	◯ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◯ Yes	◯ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◯ Yes	◯ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	◯ Yes	◯ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○Yes	○No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○Yes	○No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.	

Part 2 – Partial Infiltration Geotechnical Screening Result[5]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	○ Partial Infiltration Condition ◉ No Infiltration Condition

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
DMA 6A, 6B, 6C, 5		Preliminary Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ◉ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ◯ Yes; continue to Step 1F. ◯ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ◯ Yes; continue to Step 1G. ◯ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ◯ Yes; answer "Yes" to Criteria 1 Result. ◯ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ◯ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ⦿ No; full infiltration is not required. Skip to Part 1 Result.
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.01 inches per hour.	

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	○Yes	○No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○Yes	○No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○Yes	○No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○Yes	○No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○Yes	○No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	○ Yes	○ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	○ Full infiltration Condition ◉ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
DMA 6A, 6B, 6C, 5		
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⦿ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ⦿ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ⦿ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.01 inches per hour.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	○ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.	

Part 2 – Partial Infiltration Geotechnical Screening Result[5]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	◯ Partial Infiltration Condition ⦿ No Infiltration Condition

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

Percolation Test Data Sheet

Project:	MOB Scripps Mercy Hospital	Tested By:	Salvador Tena
		Date:	5/8/2019
Project No:	20194095.001A	Checked By:	

Borehole ID:	**PERC-1**	
Depth of Borehole:	5	feet
Diameter of Borehole:	8	inches
USCS Soil Classification:	SM	
PVC Pipe Hieght above Surface	0.3	ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	6:43	7:13	30	0.61	0.66	0.05	50.00
2	7:13	7:43	30	0.66	0.71	0.05	50.00
3	7:43	8:13	30	0.71	0.76	0.05	50.00
4	8:13	8:43	30	0.76	0.81	0.05	50.00
5	8:43	9:13	30	0.81	0.85	0.04	62.50
6	9:13	9:43	30	0.85	0.90	0.05	50.00
7	9:43	10:13	30	0.90	0.95	0.05	50.00
8	10:13	10:43	30	0.95	1.00	0.05	50.00
9	10:43	11:13	30	1.00	1.03	0.03	83.33
10	11:13	11:43	30	1.03	1.07	0.04	62.50
11	11:43	12:13	30	1.07	1.10	0.03	83.33
12	12:13	12:43	30	1.10	1.14	0.04	**62.50**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H \; 60 \, r}{\Delta t \; (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	46.80	inches
Hf	46.32	inches
ΔH	0.48	inches
Havg	46.56	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.04**	in/hr

Percolation Test Data Sheet

Project:	MOB Scripps Mercy Hospital	Tested By:	Salvador Tena
		Date:	5/8/2019
Project No:	20194095.001A	Checked By:	

Borehole ID:	**PERC-2**	
Depth of Borehole:	5	feet
Diameter of Borehole:	8	inches
USCS Soil Classification:	SM	
PVC Pipe Hieght above Surface	0.3	ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	6:47	7:17	30	0.74	0.75	0.01	250.00
2	7:17	7:47	30	0.75	0.77	0.02	125.00
3	7:47	8:17	30	0.77	0.78	0.01	250.00
4	8:17	8:47	30	0.78	0.78	0	NA
5	8:47	9:17	30	0.78	0.78	0	NA
6	9:17	9:47	30	0.78	0.79	0.01	250.00
7	9:47	10:17	30	0.79	0.80	0.01	250.00
8	10:17	10:47	30	0.80	0.80	0	NA
9	10:47	11:17	30	0.80	0.81	0.01	250.00
10	11:17	11:47	30	0.81	0.82	0.01	250.00
11	11:47	12:17	30	0.82	0.82	0	NA
12	12:17	12:47	30	0.82	0.83	0.01	**250.00**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t\ (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	50.16	inches
Hf	50.04	inches
ΔH	0.12	inches
Havg	50.10	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.01**	in/hr

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LEGEND

APPROXIMATE LOCATION OF BORING

APPROXIMATE LOCATION OF PERCOLATION TEST

0 25 50 100 Feet

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PROJECT NO.	20194095
DRAWN:	5/22/2019
DRAWN BY:	JP
CHECKED BY:	ST
FILE NAME:	20194095_Site_Merc.mxd

EXPLORATION MAP

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

2

Attachment 1e: Area Weighted Runoff Factor Calculation

B.1.1 Runoff Factor

Estimate the area weighted runoff factor for the tributary area to the BMP using runoff factor (from Table B.1-1) and area of each surface type in the tributary area and Equation B.1-2.

Equation B.1-2: Estimating Runoff Factor for Area

$$C = \frac{\sum C_x A_x}{\sum A_x}$$

where:

C_x = Runoff factor for area X

A_x = Tributary area X (acres)

These runoff factors apply to areas receiving direct rainfall only. For conditions in which runoff is routed onto a surface from an adjacent surface, see Section B.2 for determining composite runoff factors for these areas.

Table B.1-1: Runoff factors for surfaces draining to BMPs – Pollutant Control BMPs

Surface	Runoff Factor
Roofs[1]	0.90
Concrete or Asphalt[1]	0.90
Unit Pavers (grouted)[1]	0.90
Decomposed Granite	0.30
Cobbles or Crushed Aggregate	0.30
Amended, Mulched Soils or Landscape[2]	0.10
Compacted Soil (e.g., unpaved parking)	0.30
Natural (A Soil)	0.10
Natural (B Soil)	0.14
Natural (C Soil)	0.23
Natural (D Soil)	0.30

[1]Surface is considered impervious and could benefit from use of Site Design BMPs and adjustment of the runoff factor per Section B.2.1.

[2]Surface shall be designed in accordance with SD-F (Amended soils) fact sheet in Appendix E

AREA WEIGHTED RUNOFF FACTOR

C_{1_imp} = 0.9 A_{1_imp} = 2.20 ac C_{1_perv} = 0.1 A_{1_perv} = 0.184 ac

C_1 = (0.9 * 1.85 ac) + (0.1 * 0.184 ac) / 2.39 ac = 0.84

C_{2_imp} = 0.9 A_{2_imp} = 1.29 ac C_{2_perv} = 0.1 A_{2_perv} = 0.505 ac

C_2 = (0.9 * 1.29 ac) + (0.1 * 0.505 ac) / 1.79 ac = 0.67

C_{3_imp} = 0.9 A_{3_imp} = 0.96 ac C_{3_perv} = 0.1 A_{3_perv} = 0.321 ac

C_3 = (0.9 * 0.96 ac) + (0.1 * 0.321 ac) / 1.29 ac = 0.70

C_{4_imp} = 0.9 A_{4_imp} = 0.46 ac C_{4_perv} = 0.1 A_{4_perv} = 0.183 ac

C_4 = (0.9 * 0.46 ac) + (0.1 * 0.183 ac) / 0.643 ac = 0.67

C_{5_imp} = 0.9 A_{5_imp} = 1.08 ac C_{5_perv} = 0.1 A_{5_perv} = 0.436 ac

C_5 = (0.9 * 1.08 ac) + (0.1 * 0.436 ac) / 1.52 ac = 0.67

C_{6A_imp} = 0.9 A_{6A_imp} = 0.32 ac C_{6A_perv} = 0.1 A_{6A_perv} = 0.018 ac

C_{6A} = (0.9 * 0.303 ac) + (0.1 * 0.018 ac) / 0.32 ac = 0.85

C_{6_imp} = 0.9 A_{6B_imp} = 0.37 ac C_{6B_perv} = 0.1 A_{6B_perv} = 0.018 ac

C_{6B} = (0.9 * 0.349 ac) + (0.1 * 0.018 ac) / 0.37 ac = 0.86

C_{6C_imp} = 0.9 A_{6C_imp} = 1.01 ac C_{6C_perv} = 0.1 A_{6C_perv} = 0.069 ac

C_{6C} = (0.9 * 1.01 ac) + (0.1 * 0.069 ac) / 1.08 ac = 0.85

C_{7_imp} = 0.9 A_{7_imp} = 0.324 ac C_{7_perv} = 0.1 A_{7_perv} = 0.032 ac

C_7 = (0.9 * 1.05 ac) + (0.1 * 0.041 ac) / 0.356 ac = 0.83

C_{8_imp} = 0.9 A_{8_imp} = 0.225 ac C_{8_perv} = 0.1 A_{8_perv} = 0.011 ac

C_8 = (0.9 * 0.225 ac) + (0.1 * 0.011 ac) /0.236 ac = 0.86

C_{9_imp} = 0.9 A_{9_imp} = 0.145 ac C_{9_perv} = 0.1 A_{9_perv} = 0.016 ac

C_9 = (0.9 * 0.145ac) + (0.1 * 0.016 ac) / 0.161 ac = 0.82

C_{utiliy_yard} = 0.9 A_{utiliy_yard} = 0.0 ac C_{utiliy_yard} = 0.1 A_{utiliy_yard} = 0.413 ac

C_{utiliy_yard} = (0.9 * 0.0ac) + (0.1 * 0.413 ac) / 0.413 ac = 0.10

$A_{\_total}$ = 460,000 sf (10.56 ac)

$C_{\_total} = \sum(C_x * A_x/A_{\_total})$

$C_{\_total}$ = 0.733

Attachment 1e: Worksheet B-2.1

DMA 1

	Design Capture Volume	Worksheet B.2-1		
1	85^{th} percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	2.39	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.84	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	4287	cubic-feet

Attachment 1e: Worksheet B-2.1

DMA 2

	Design Capture Volume	Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	1.79	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.67	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	2586	cubic-feet

Attachment 1e: Worksheet B-2.1

DMA 3

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	1.29	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.70	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	1,927	cubic-feet

Attachment 1e: Worksheet B-2.1

DMA 4

	Design Capture Volume	Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	0.64	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.67	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	924	cubic-feet

Attachment 1e: Worksheet B-2.1

DMA 5

	Design Capture Volume	Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	1.52	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.67	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	2173	cubic-feet

Attachment 1e: Worksheet B-2.1 DMA 6A

	Design Capture Volume	Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	0.32	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.85	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	566	cubic-feet

Attachment 1e: Worksheet B-2.1 DMA 6B

	Design Capture Volume	Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	0.37	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.86	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	647	cubic-feet

Attachment 1e: Worksheet B-2.1 DMA 6C

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	1.08	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.85	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	1900	cubic-feet

Attachment 1e: Worksheet B-2.1

DMA 7

	Design Capture Volume	Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	0.356	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.83	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	631	cubic-feet

Attachment 1e: Worksheet B-2.1

DMA 8

	Design Capture Volume	Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	0.236	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.86	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	436	cubic-feet

Attachment 1e: Worksheet B-2.1

DMA 9

	Design Capture Volume	Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.59	inches
2	Area tributary to BMP (s)	A=	0.161	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.82	unitless
4	Trees Credit Volume Note: In the SWQMP list the number of trees, size of each tree, amount of soil volume installed for each tree, contributing area to each tree and the inlet opening dimension for each tree.	TCV=	0	cubic-feet
5	Rain barrels Credit Volume Note: In the SWQMP list the number of rain barrels, size of each rain barrel and the use of the captured storm water runoff.	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	282	cubic-feet

DMA 2

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 2-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	

1	Area draining to the BMP	78,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.674	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	2585	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	3877	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	834	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]	1939	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	902	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]	1577	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	1577	sq. ft.
23	Provided BMP Footprint	2900	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 3

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 3-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	
1	Area draining to the BMP	56,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.7	
3	85^{th} percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	1927	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	2891	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	622	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]	1446	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	672	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]	1176	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	1176	sq. ft.
23	Provided BMP Footprint	1900	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 4

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 4-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	
1	Area draining to the BMP	28,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.671	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	924	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	1386	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	298	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]	693	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	322	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]	564	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	564	sq. ft.
23	Provided BMP Footprint	1000	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 5

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 5-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	
1	Area draining to the BMP	66,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.67	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	2174	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	3261	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	701	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]	1631	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	758	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]	1327	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	1327	sq. ft.
23	Provided BMP Footprint	2190	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 6A

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 6A-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	
1	Area draining to the BMP	14,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.822	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	566	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	849	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	183	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]	424	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	197	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]	345	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	345	sq. ft.
23	Provided BMP Footprint	630	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 6B

The City of SAN DIEGO

	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 6B-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	
1	Area draining to the BMP	16,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.822	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	647	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	970	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	209	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]	485	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	226	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]	395	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	395	sq. ft.
23	Provided BMP Footprint	730	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 6C

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 6C-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	
1	Area draining to the BMP	47,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.822	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	1900	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	2849	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	613	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]	1425	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	663	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]	1159	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	1159	sq. ft.
23	Provided BMP Footprint	2,100	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 7

The City of SAN DIEGO

	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 7-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	
1	Area draining to the BMP	15,500	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.8277	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	631	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	946	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	203	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]	473	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]	385	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	385	sq. ft.
23	Provided BMP Footprint	715	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 8

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 8-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	

1	Area draining to the BMP	10,300	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.861	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	436	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	654	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	141	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]	327	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	152	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]	266	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	266	sq. ft.
23	Provided BMP Footprint	495	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 9

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 9-1	
Sizing Method for Pollutant Removal Criteria		**Worksheet B.5-1**	

1	Area draining to the BMP	7,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.82	
3	85th percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	282	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	12	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	45	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	9	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)]	25.8	inches
15	Total Depth Treated [Line 13 + Line 14]	55.8	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	423	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	91	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]	212	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	98	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]	172	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	172	sq. ft.
23	Provided BMP Footprint	325	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

DMA 1

The City of SAN DIEGO	Project Name	Scripps Mercy Hospital	
	BMP ID	BMP 1-1	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	

1	Area draining to the BMP	104,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.8385	
3	85^{th} percentile 24-hour rainfall depth	0.59	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	4288	cu. ft.
Volume Retention Requirement			
5	Measured infiltration rate in the DMA Note: When mapped hydrologic soil groups are used enter 0.10 for NRCS Type D soils and for NRCS Type C soils enter 0.30 When in no infiltration condition and the actual measured infiltration rate is unknown enter 0.0 if there are geotechnical and/or groundwater hazards identified in Appendix C	0.1	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.05	in/hr.
8	Average annual volume reduction target (Figure B.5-2) When Line 7 > 0.01 in/hr. = Minimum (40, 166.9 x Line 7 +6.62) When Line 7 ≤ 0.01 in/hr. = 3.5%	15.0	%
9	Fraction of DCV to be retained (Figure B.5-3) When Line 8 > 8% = $0.0000013 \times \text{Line } 8^3 - 0.000057 \times \text{Line } 8^2 + 0.0086 \times \text{Line } 8 - 0.014$ When Line 8 ≤ 8% = 0.023	0.106	
10	Target volume retention [Line 9 x Line 4]	454	cu. ft.

DMA 1

The City of SAN DIEGO

		Project Name	Scripps Mercy Hospital				
		BMP ID	BMP 1-1				
Volume Retention for No Infiltration Condition					Worksheet B.5-6		
1	Area draining to the biofiltration BMP					104,000	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)					0.8385	
3	Effective impervious area draining to the BMP [Line 1 x Line 2]					87204	sq. ft.
4	Required area for Evapotranspiration [Line 3 x 0.03]					2616	sq. ft.
5	Biofiltration BMP Footprint					0	sq. ft.
Landscape Area (must be identified on DS-3247)							
		Identification	1	2	3	4	5
6	Landscape area that meet the requirements in SD-B and SD-F Fact Sheet (sq. ft.)						
7	Impervious area draining to the landscape area (sq. ft.)						
8	Impervious to Pervious Area ratio [Line 7/Line 6]		0.00	0.00	0.00	0.00	0.00
9	Effective Credit Area If (Line 8 >1.5, Line 6, Line 7/1.5]		0	0	0	0	0
10	Sum of Landscape area [sum of Line 9 Id's 1 to 5]					0	sq. ft.
11	Provided footprint for evapotranspiration [Line 5 + Line 10]					0	sq. ft.
Volume Retention Performance Standard							
12	Is Line 11 ≥ Line 4?		No, Proceed to Line 13				
13	Fraction of the performance standard met through the BMP footprint and/or landscaping [Line 11/Line 4]					0	
14	Target Volume Retention [Line 10 from Worksheet B.5.2]					454	cu. ft.
15	Volume retention required from other site design BMPs [(1-Line 13) x Line 14]					454	cu. ft.
Site Design BMP							
	Identification	Site Design Type				Credit	
16	1	(3) 6' Dia. x 74L cisterns				8760	cu. ft.
	2						cu. ft.
	3						cu. ft.
	4						cu. ft.
	5						cu. ft.
	Sum of volume retention benefits from other site design BMPs (e.g. trees; rain barrels etc.). [sum of Line 16 Credits for Id's 1 to 5] Provide documentation of how the site design credit is calculated in the PDP SWQMP.					8760	cu. ft.
17	Is Line 16 ≥ Line 15?		Volume Retention Performance Standard is Met				

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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: Scripps Mercy Hospital

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

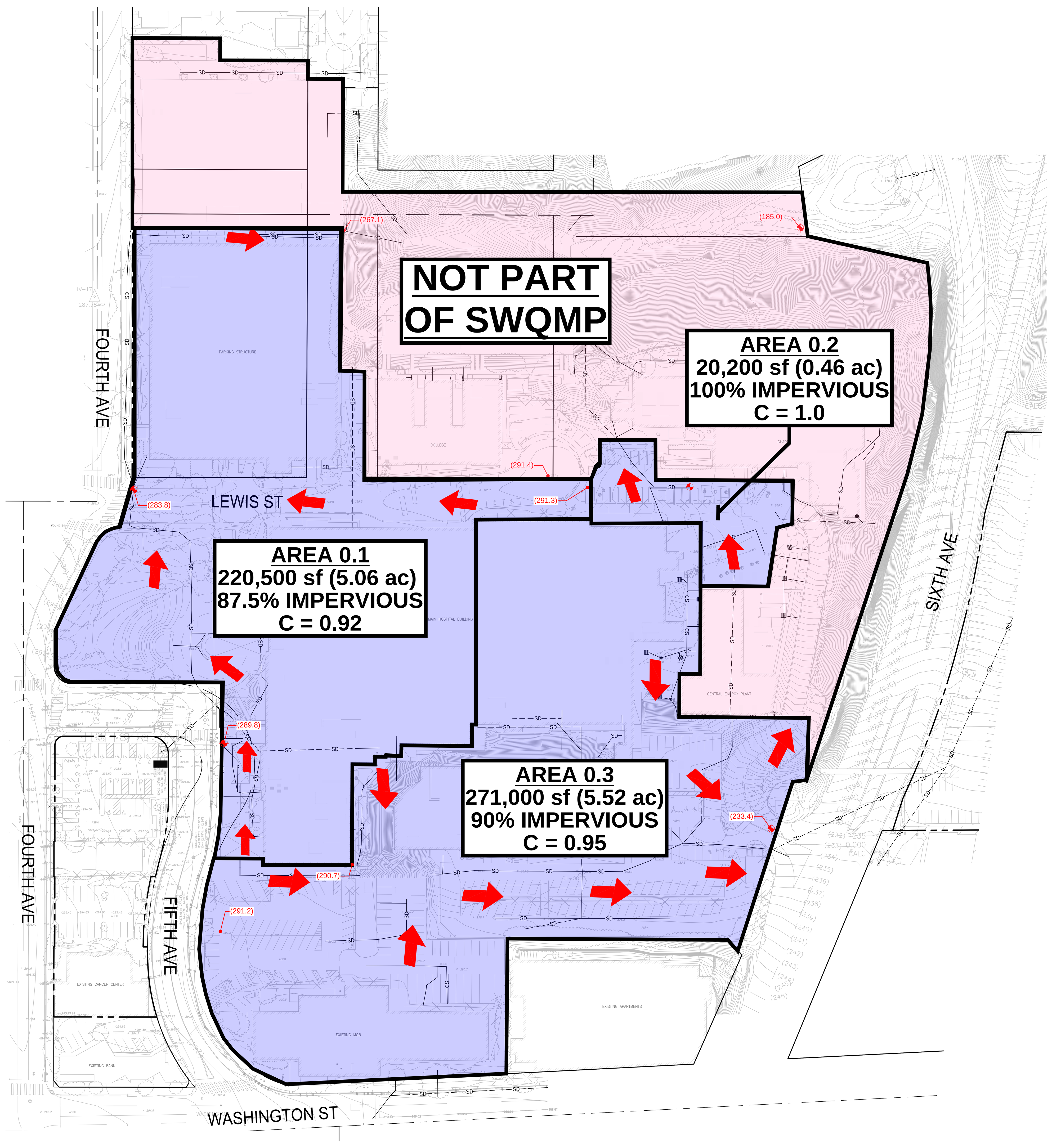
The Hydromodification Management Exhibit must identify:

- ☑ 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).

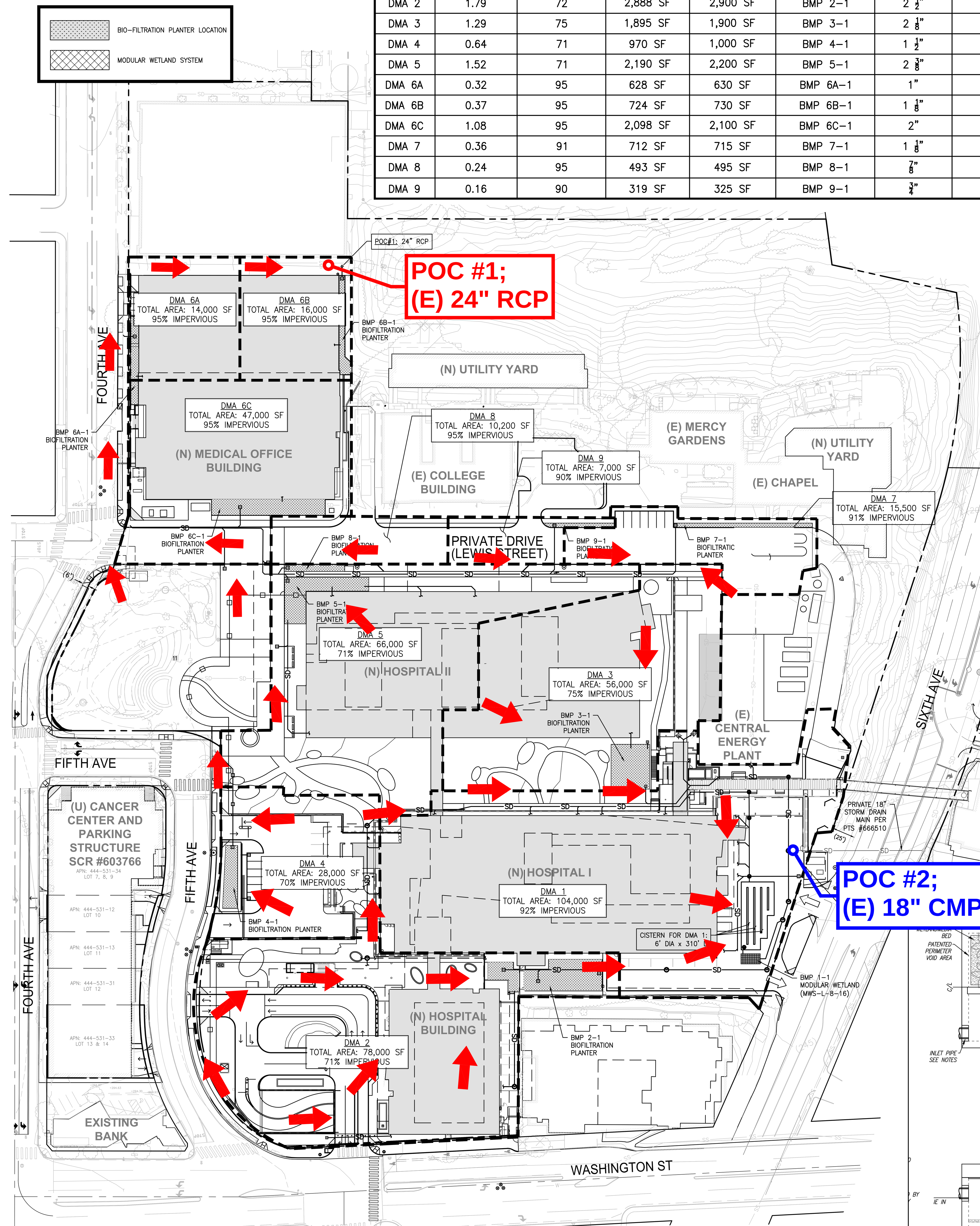
Attachment 2a: HMP Exhibit

BMP selected: Biofiltration (BF-1 & BF-3 w/ Cisterns)

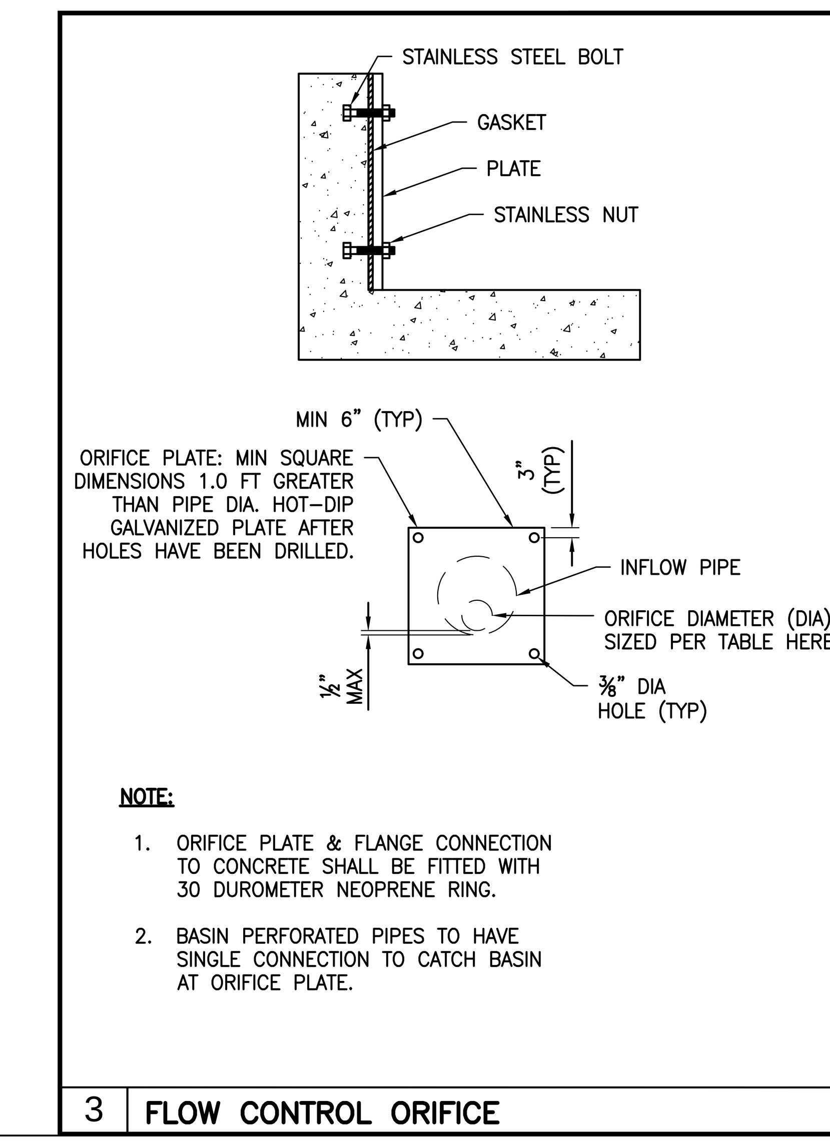
Existing Condition

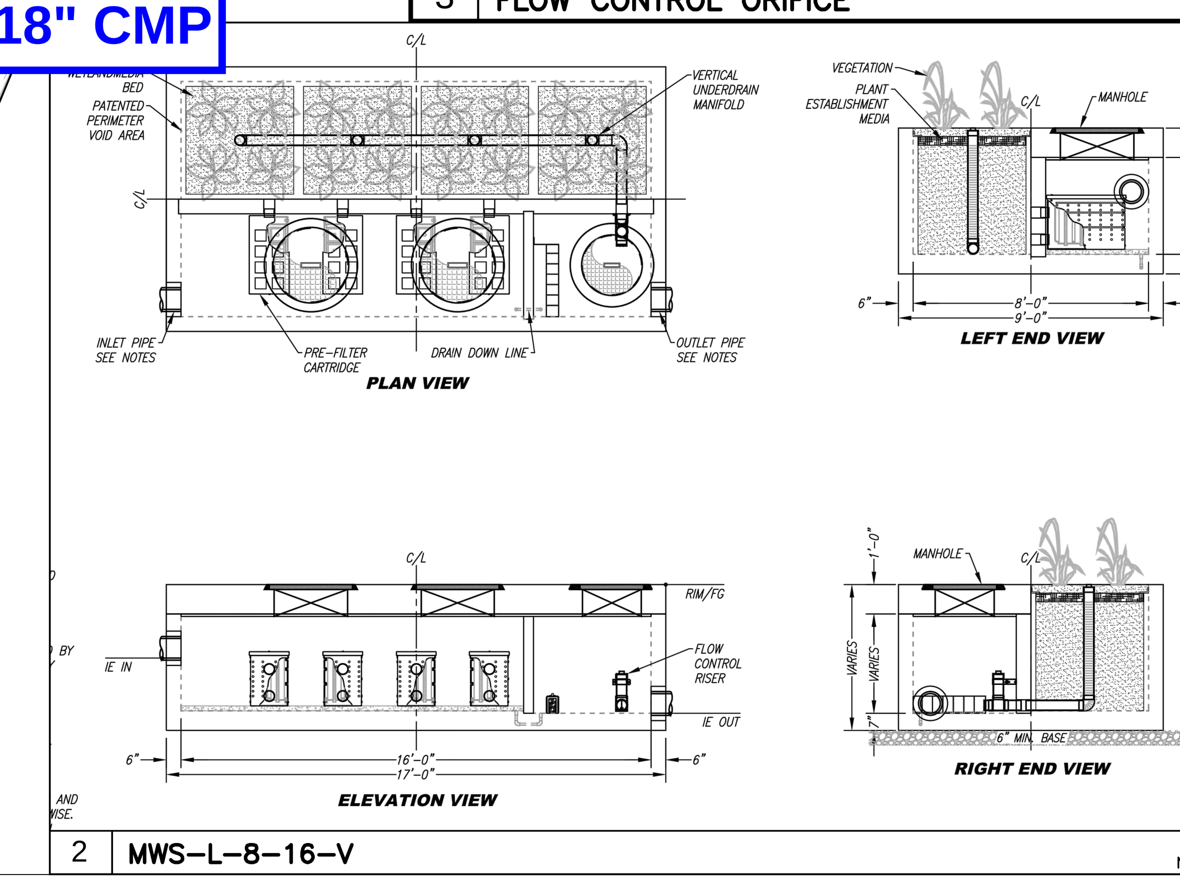


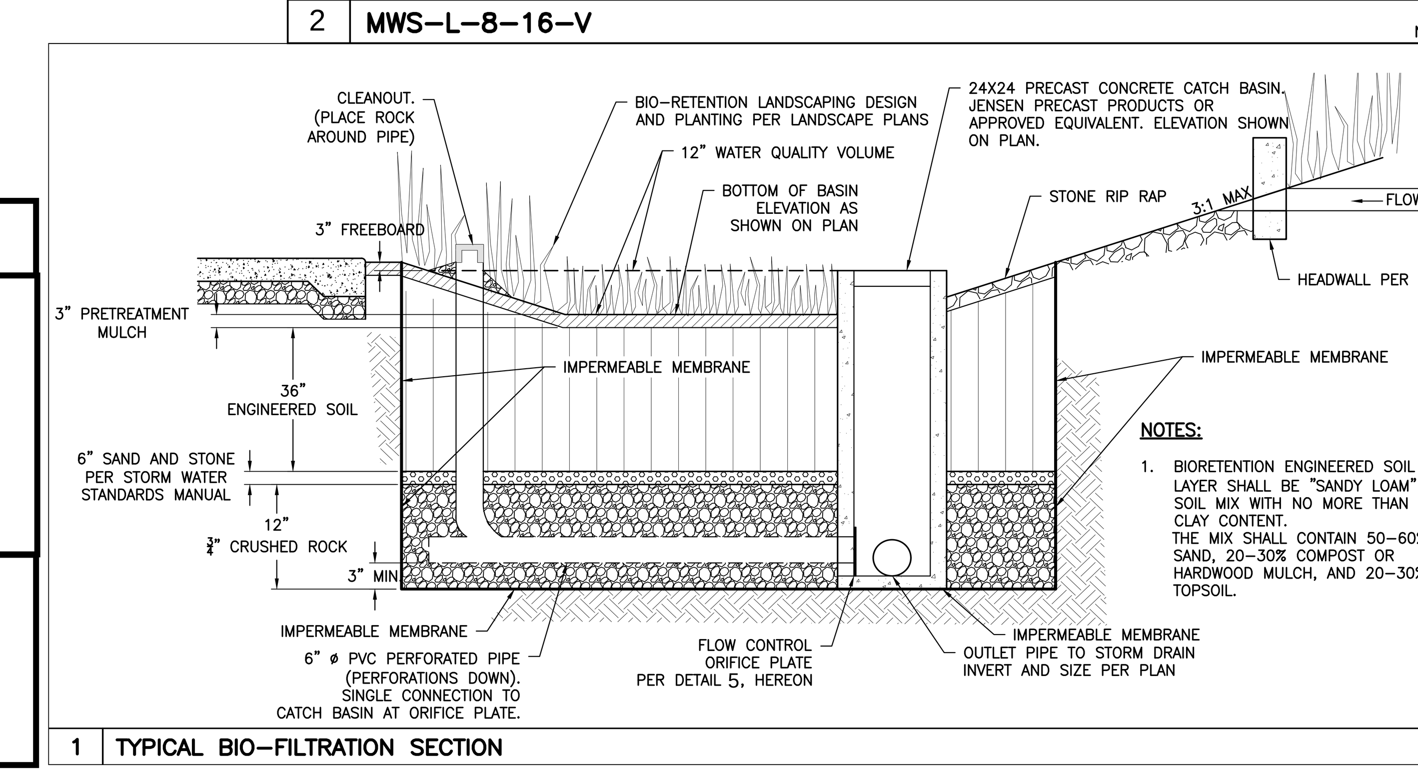
Proposed Condition



DMA SUMMARY TABLE								
DMA	AREA (AC.)	IMPERVIOUS%	REQ. BMP CAPACITY*	PROP. BMP CAPACITY	BMP ID	ORIFICE SIZE (IN.)	MAX ORIFICE OUFLOW (CFS)	DRAINS TO (POC ID)
DMA 1	2.39	92	8,712 CF	8,760 CF	BMP 1-1	3 1/8"	0.520	#2
DMA 2	1.79	72	2,888 SF	2,900 SF	BMP 2-1	2 1/2"	0.344	#2
DMA 3	1.29	75	1,895 SF	1,900 SF	BMP 3-1	2 1/8"	0.249	#2
DMA 4	0.64	71	970 SF	1,000 SF	BMP 4-1	1 1/2"	0.124	#1
DMA 5	1.52	71	2,190 SF	2,200 SF	BMP 5-1	2 3/8"	0.311	#1
DMA 6A	0.32	95	628 SF	630 SF	BMP 6A-1	1"	0.055	#1
DMA 6B	0.37	95	724 SF	730 SF	BMP 6B-1	1 1/8"	0.065	#1
DMA 6C	1.08	95	2,098 SF	2,100 SF	BMP 6C-1	2"	0.220	#1
DMA 7	0.36	91	712 SF	715 SF	BMP 7-1	1 1/8"	0.070	#1
DMA 8	0.24	95	493 SF	495 SF	BMP 8-1	7/8"	0.042	#1
DMA 9	0.16	90	319 SF	325 SF	BMP 9-1	3/4"	0.031	#1







Existing Natural Hydrologic Features

CCSYA: Not Applicable

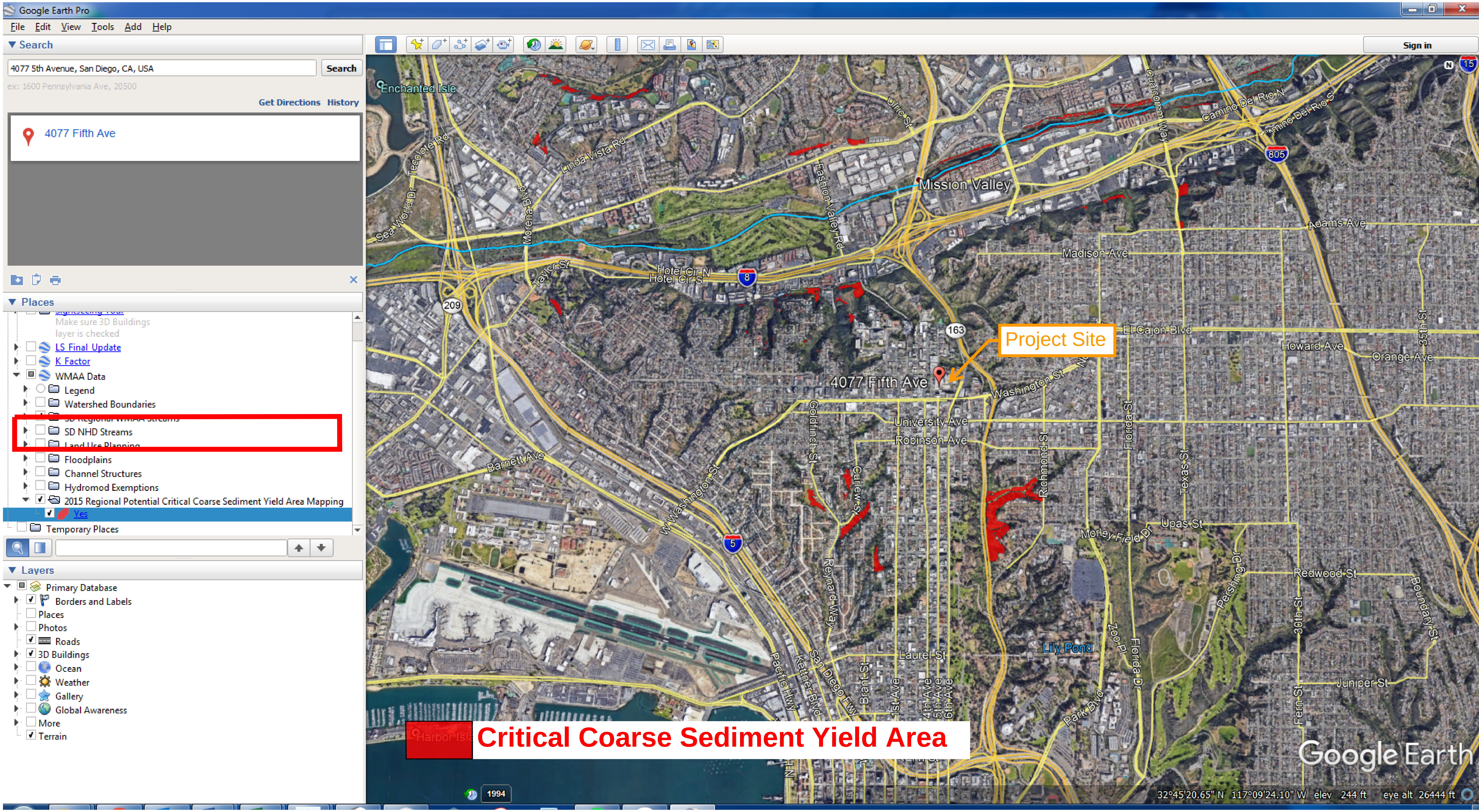
Appx. Depth to Groundwater: +20 feet

- Boring tests show refusal from 20 to 21 ft. deep
- Only one out of 5 boring logs show groundwater encounter; the perched grounwater table was detected at 21 ft below surface.

Underlying HSG: Group D

Underlying HSG N/A, but judging from SM (silty sand) below artificial fill and infiltration rate ~0" in/hr, HGS Group D was selected, per San Diego Stormwater Design Manual.

Attachment 2b: Critical Coarse Sediment Yield Area Exhibit



Attachment 2d

DMA 1

BMP Sizing Spreadsheet V3.0			
Project Name:	Enter Project Name	Hydrologic Unit:	Enter Hydrologic Unit
Project Applicant:	Enter Appplicant Name	Rain Gauge:	Lindbergh
Jurisdiction:	Enter Jurisdiction	Total Project Area:	Enter Total Project Area
Parcel (APN):	Enter Parcel Number(s)	Low Flow Threshold:	0.1Q2
BMP Name:	BMP-1	BMP Type:	Cistern
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	NA

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Volume	Volume (CF)
DMA 1_Impervious	96,000	D	Moderate	Concrete	1.0	0.09	8640
DMA 1_Pervious	8,000	D	Moderate	Landscape	0.1	0.09	72
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	104,000					Minimum BMP Size	8712
						Proposed BMP Size*	8760

Standard Cistern Depth (Overflow Elevation)	3.5	ft
Provided Cistern Depth (Overflow Elevation)	3.5	ft
Minimum Required Cistern Footprint)	2489	CF

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table refe

Describe the BMP's in sufficient detail in your PDP SWQMP to demonstrate the area, volume, and other criteria can be met within the constraints of the site.

BMP's must be adapted and applied to the conditions specific to the development project such as unstable slopes or the lack of available head.
Designated Staff have final review and approval authority over the project design.

This BMP Sizing Spreadsheet has been updated in conformance with the San Diego Region Model BMP Design Manual, April 2018. For questions or concerns please contact the jurisdiction in whi

Attachment 2d

DMA 1

BMP Sizing Spreadsheet V3.0			
Project Name:	Enter Project Name	Hydrologic Unit:	Enter Hydrologic Unit
Project Applicant:	Enter Appplicant Name	Rain Gauge:	Lindbergh
Jurisdiction:	Enter Jurisdiction	Total Project Area:	Enter Total Project Area
Parcel (APN):	Enter Parcel Number(s)	Low Flow Threshold:	0.1Q2
BMP Name	BMP-1	BMP Type:	Cistern

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio	DMA Area (ac)	Orifice Flow - $\%Q_2$	Orifice Area
		Soil Type	Slope	(cfs/ac)		(cfs)	(in^2)
DMA 1_Impervious	Lindbergh	D	Moderate	0.437	2.204	0.482	**7.11**
DMA 1_Pervious	Lindbergh	D	Moderate	0.437	0.184	0.040	**0.59**

3.50	**0.522**	**7.70**	**3.13**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

Provide Hand Calc.	**0.520**	**7.67**	**3.125**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	Provide Hand Calculation

DMA 2

Attachment 2d

BMP Sizing Spreadsheet V3.0			
Project Name:	Enter Project Name	Hydrologic Unit:	Enter Hydrologic Unit
Project Applicant:	Enter Appplicant Name	Rain Gauge:	Lindbergh
Jurisdiction:	Enter Jurisdiction	Total Project Area:	Enter Total Project Area
Parcel (APN):	Enter Parcel Number(s)	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 2-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 2_Impervious	55,500	D	Moderate	Concrete	1.0	0.05	2775
DMA 2_Pervious	22,500	D	Moderate	Landscape	0.1	0.05	113
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	78,000					Minimum BMP Size	2888
						Proposed BMP Size*	2900

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table refe

Describe the BMP's in sufficient detail in your PDP SWQMP to demonstrate the area, volume, and other criteria can be met within the constraints of the site.

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Attachment 2d

DMA 2

BMP Sizing Spreadsheet V3.0			
Project Name:	Enter Project Name	Hydrologic Unit:	Enter Hydrologic Unit
Project Applicant:	Enter Appplicant Name	Rain Gauge:	Lindbergh
Jurisdiction:	Enter Jurisdiction	Total Project Area:	Enter Total Project Area
Parcel (APN):	Enter Parcel Number(s)	Low Flow Threshold:	0.1Q2
BMP Name	BMP 2-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio (cfs/ac)	DMA Area (ac)	Orifice Flow - %Q_2 (cfs)	Orifice Area (in^2)
		Soil Type	Slope				
DMA 2_Impervious	Lindbergh	D	Moderate	0.437	1.274	0.278	**3.97**
DMA 2_Pervious	Lindbergh	D	Moderate	0.437	0.517	0.113	**1.61**

3.75	**0.391**	**5.58**	**2.66**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.353	**0.380**	**5.41**	**2.625**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	2.3

Attachment 2d

DMA 3

BMP Sizing Spreadsheet V3.0			
Project Name:	Enter Project Name	Hydrologic Unit:	Enter Hydrologic Unit
Project Applicant:	Enter Appplicant Name	Rain Gauge:	Lindbergh
Jurisdiction:	Enter Jurisdiction	Total Project Area:	Enter Total Project Area
Parcel (APN):	Enter Parcel Number(s)	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 3-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 3_Impervious	42,000	D	Flat	Concrete	1.0	0.05	1995
DMA 3_Pervious	14,000	D	Flat	Landscape	0.1	0.05	35
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	56,000					Minimum BMP Size	2030
						Proposed BMP Size*	2050

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

Describe the BMP's in sufficient detail in your PDP SWQMP to demonstrate the area, volume, and other criteria can be met within the constraints of the site.

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DMA 3

Attachment 2d

BMP Sizing Spreadsheet V3.0			
Project Name:	Enter Project Name	Hydrologic Unit:	Enter Hydrologic Unit
Project Applicant:	Enter Appplicant Name	Rain Gauge:	Lindbergh
Jurisdiction:	Enter Jurisdiction	Total Project Area:	Enter Total Project Area
Parcel (APN):	Enter Parcel Number(s)	Low Flow Threshold:	0.1Q2
BMP Name	BMP 3-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio (cfs/ac)	DMA Area (ac)	Orifice Flow - %Q_2 (cfs)	Orifice Area (in^2)
		Soil Type	Slope				
DMA 3_Impervious	Lindbergh	D	Flat	0.429	0.964	0.207	**2.95**
DMA 3_Pervious	Lindbergh	D	Flat	0.429	0.321	0.069	**0.98**

3.75	**0.276**	**3.93**	**2.24**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.232	**0.249**	**3.55**	**2.125**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	2.5

DMA 4

Attachment 2d

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 4-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 4_Impervious	20,000	D	Flat	Concrete	1.0	0.05	950
DMA 4_Pervious	8,000	D	Flat	Landscape	0.1	0.05	20
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	28,000					Minimum BMP Size	970
						Proposed BMP Size*	1000

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

Describe the BMP's in sufficient detail in your PDP SWQMP to demonstrate the area, volume, and other criteria can be met within the constraints of the site.

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Attachment 2d

DMA 4

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name	BMP 4-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio	DMA Area (ac)	Orifice Flow - %Q_2	Orifice Area
		Soil Type	Slope	(cfs/ac)		(cfs)	(in^2)
DMA 4_Impervious	Lindbergh	D	Flat	0.429	0.459	0.098	**1.40**
DMA 4_Pervious	Lindbergh	D	Flat	0.429	0.184	0.039	**0.56**

3.75	**0.138**	**1.97**	**1.58**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.115	**0.124**	**1.77**	**1.500**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	2.4

Attachment 2d

DMA 5

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 5-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 5_Impervious	46,000	D	Flat	Concrete	1.0	0.05	2185
DMA 5_Pervious	20,000	D	Flat	Landscape	0.1	0.05	50
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	66,000					Minimum BMP Size	2235
						Proposed BMP Size*	2250

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

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Attachment 2d

DMA 5

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name	BMP 5-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio (cfs/ac)	DMA Area (ac)	Orifice Flow - $\%Q_2$ (cfs)	Orifice Area (in^2)
		Soil Type	Slope				
DMA 5_Impervious	Lindbergh	D	Flat	0.429	1.056	0.227	**3.23**
DMA 5_Pervious	Lindbergh	D	Flat	0.429	0.459	0.098	**1.40**

3.75	**0.325**	**4.63**	**2.43**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.289	**0.311**	**4.43**	**2.375**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	2.2

DMA 6A

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 6A-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 6A_Impervious	13,200	D	Flat	Concrete	1.0	0.05	627
DMA 6A_Pervious	800	D	Flat	Landscape	0.1	0.05	2
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	14,000					Minimum BMP Size	629
						Proposed BMP Size*	630

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

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DMA 6A

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name	BMP 6A-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio (cfs/ac)	DMA Area (ac)	Orifice Flow - $\%Q_2$ (cfs)	Orifice Area (in^2)
		Soil Type	Slope				
DMA 6A_Impervious	Lindbergh	D	Flat	0.429	0.303	0.065	**0.93**
DMA 6A_Pervious	Lindbergh	D	Flat	0.429	0.018	0.004	**0.06**

3.75	**0.069**	**0.98**	**1.12**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.051	**0.055**	**0.79**	**1.000**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	3.4

DMA 6B

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 6B-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 6B_Impervious	15,200	D	Flat	Concrete	1.0	0.05	722
DMA 6B_Pervious	800	D	Flat	Landscape	0.1	0.05	2
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	16,000					Minimum BMP Size	724
						Proposed BMP Size*	730

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

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DMA 6B

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name	BMP 6B-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio	DMA Area (ac)	Orifice Flow - %Q_2	Orifice Area
		Soil Type	Slope	(cfs/ac)		(cfs)	(in^2)
DMA 6B_Impervious	Lindbergh	D	Flat	0.429	0.349	0.075	**1.07**
DMA 6B_Pervious	Lindbergh	D	Flat	0.429	0.018	0.004	**0.06**

3.75	**0.079**	**1.12**	**1.20**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.065	**0.070**	**0.99**	**1.125**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	3.1

DMA 6C

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 6C-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 6C_Impervious	44,000	D	Flat	Concrete	1.0	0.05	2090
DMA 6C_Pervious	3,000	D	Flat	Landscape	0.1	0.05	8
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	47,000					Minimum BMP Size	2098
						Proposed BMP Size*	2100

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

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DMA 6C

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name	BMP 6C-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio (cfs/ac)	DMA Area (ac)	Orifice Flow - $\%Q_2$ (cfs)	Orifice Area (in^2)
		Soil Type	Slope				
DMA 6C_Impervious	Lindbergh	D	Flat	0.429	1.010	0.217	**3.09**
DMA 6C_Pervious	Lindbergh	D	Flat	0.429	0.069	0.015	**0.21**

3.75	**0.231**	**3.30**	**2.05**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.205	**0.220**	**3.14**	**2.000**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	2.8

DMA 7

Attachment 2d

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 10-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 10_Impervious	14,100	D	Flat	Concrete	1.0	0.05	705
DMA 10_Pervious	1,400	D	Moderate	Landscape	0.1	0.05	7
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	15,500					Minimum BMP Size	712
						Proposed BMP Size*	715

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

Describe the BMP's in sufficient detail in your PDP SWQMP to demonstrate the area, volume, and other criteria can be met within the constraints of the site.

BMP's must be adapted and applied to the conditions specific to the development project such as unstable slopes or the lack of available head.
Designated Staff have final review and approval authority over the project design.

This BMP Sizing Spreadsheet has been updated in conformance with the San Diego Region Model BMP Design Manual, April 2018. For questions or concerns please contact the jurisdiction in which your project is located.

DMA 7

Attachment 2d

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name	BMP 10-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio	DMA Area (ac)	Orifice Flow - $\%Q_2$	Orifice Area
		Soil Type	Slope	(cfs/ac)		(cfs)	(in^2)
DMA 10_Impervious	Lindbergh	D	Flat	0.429	0.324	0.069	**0.99**
DMA 10_Pervious	Lindbergh	D	Moderate	0.437	0.032	0.007	**0.10**

3.75	**0.076**	**1.09**	**1.18**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.065	**0.070**	**0.99**	**1.125**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	3.1

Attachment 2d

DMA 8

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 8-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 8_Impervious	9,800	D	Flat	Concrete	1.0	0.05	490
DMA 8_Pervious	500	D	Moderate	Landscape	0.1	0.05	3
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	10,300					Minimum BMP Size	493
						Proposed BMP Size*	495

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

Describe the BMP's in sufficient detail in your PDP SWQMP to demonstrate the area, volume, and other criteria can be met within the constraints of the site.

BMP's must be adapted and applied to the conditions specific to the development project such as unstable slopes or the lack of available head.
Designated Staff have final review and approval authority over the project design.

This BMP Sizing Spreadsheet has been updated in conformance with the San Diego Region Model BMP Design Manual, April 2018. For questions or concerns please contact the jurisdiction in which your project is located.

Attachment 2d

DMA 8

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name	BMP 8-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio (cfs/ac)	DMA Area (ac)	Orifice Flow - $\%Q_2$ (cfs)	Orifice Area (in^2)
		Soil Type	Slope				
DMA 8_Impervious	Lindbergh	D	Flat	0.429	0.225	0.048	**0.69**
DMA 8_Pervious	Lindbergh	D	Moderate	0.437	0.011	0.003	**0.04**

3.75	**0.051**	**0.72**	**0.96**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.039	**0.042**	**0.60**	**0.875**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	3.5

Attachment 2d

DMA 9

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name:	BMP 9-1	BMP Type:	Biofiltration
BMP Native Soil Type:	N/A - Impervious Liner	BMP Infiltration Rate (in/hr):	N/A

Areas Draining to BMP						HMP Sizing Factors	Minimum BMP Size
DMA Name	Area (sf)	Pre Project Soil Type	Pre-Project Slope	Post Project Surface Type	Area Weighted Runoff Factor (Table G.2-1)[1]	Surface Area	Surface Area (SF)
DMA 9_Impervious	6,300	D	Flat	Concrete	1.0	0.05	315
DMA 9_Pervious	700	D	Moderate	Landscape	0.1	0.05	4
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
						0	0
BMP Tributary Area	7,000					Minimum BMP Size	319
						Proposed BMP Size*	325

* Assumes standard configuration

Surface Ponding Depth	12.00	in
Bioretention Soil Media Depth	18.00	in
Filter Coarse	6.00	in
Gravel Storage Layer Depth	12	in
Underdrain Offset	3.0	in

Notes:

1. Runoff factors which are used for hydromodification management flow control (Table G.2-1) are different from the runoff factors used for pollutant control BMP sizing (Table B.1-1). Table references are taken from the San Diego Region Model BMP Design Manu

Describe the BMP's in sufficient detail in your PDP SWQMP to demonstrate the area, volume, and other criteria can be met within the constraints of the site.

BMP's must be adapted and applied to the conditions specific to the development project such as unstable slopes or the lack of available head.
Designated Staff have final review and approval authority over the project design.

This BMP Sizing Spreadsheet has been updated in conformance with the San Diego Region Model BMP Design Manual, April 2018. For questions or concerns please contact the jurisdiction in which your project is located.

DMA 9

Attachment 2d

BMP Sizing Spreadsheet V3.0			
Project Name:	Scripps Mercy Hospital	Hydrologic Unit:	0
Project Applicant:	Scripps Health	Rain Gauge:	Lindbergh
Jurisdiction:	0	Total Project Area:	774,000
Parcel (APN):	444-710-25-00	Low Flow Threshold:	0.1Q2
BMP Name	BMP 9-1	BMP Type:	Biofiltration

DMA Name	Rain Gauge	Pre-developed Condition		Unit Runoff Ratio	DMA Area (ac)	Orifice Flow - $\%Q_2$	Orifice Area
		Soil Type	Slope	(cfs/ac)		(cfs)	(in^2)
DMA 9_Impervious	Lindbergh	D	Flat	0.429	0.145	0.031	**0.44**
DMA 9_Pervious	Lindbergh	D	Moderate	0.437	0.016	0.004	**0.05**

3.75	**0.035**	**0.49**	**0.79**
Max Orifice Head (feet)	**Max Tot. Allowable Orifice Flow (cfs)**	**Max Tot. Allowable Orifice Area (in^2)**	**Max Orifice Diameter (in)**

0.029	**0.031**	**0.44**	**0.750**
Average outflow during surface drawdown (cfs)	Max Orifice Outflow (cfs)	Actual Orifice Area (in^2)	Selected Orifice Diameter (in)

Drawdown (Hrs)	3.1

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Attachment 3
Structural BMP Maintenance Information

This is the cover sheet for Attachment 3.

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Project Name: Scripps Mercy Hospital

Indicate which Items are Included:

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

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
- ☑ 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:
Scripps Health
10140 Campus Point Drive, Suite #210
San Diego, CA 92121

(THIS SPACE IS FOR RECORDER'S USE ONLY)

STORM WATER MANAGEMENT AND DISCHARGE CONTROL MAINTENANCE AGREEMENT

APPROVAL NUMBER:	ASSESSORS PARCEL NUMBER:	PROJECT NUMBER:
	444-710-25, 444-733-25, -26	658548

This agreement is made by and between the City of San Diego, a municipal corporation [City] and Scripps Health, the owner or duly authorized representative of the owner [Property Owner] of property located at

4077 FIFTH AVENUE SAN DIEGO, CA 92103
(Property Address)

and more particularly described as: ______________________________

(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): 658548.

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): 658548.

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): 658548.

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) 658548.

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): ______________________________

____
(Owner Signature)

____
(Print Name and Title)

____
(Company/Organization Name)

____
(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

LEGAL DESCRIPTIONS

540 LEWIS STREET:

REAL PROPERTY IN THE CITY OF SAN DIEGO, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, DESCRIBED AS FOLLOWS:

PARCEL A:

PARCEL 1 AS SHOWN ON THAT CERTAIN "PARCEL MAP NO. 18598" FILED FOR RECORD IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO, CALIFORNIA ON DECEMBER 8, 2000 AS INSTRUMENT NO. 2000-0669213 OF OFFICIAL RECORDS. TOGETHER WITH THAT PORTION OF 5TH AVENUE VACATED PURSUANT TO "RESOLUTION NUMBER R-303733 ADOPTED ON MAY 20, 2008" AND RECORDED ON JUNE 6, 2008 AS INSTRUMENT NO. 2008-0305191 OF OFFICIAL RECORDS.

PARCEL B:

PARCEL 2 OF PARCEL MAP 18598, IN THE CITY OF SAN DIEGO, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, FILED IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO, DECEMBER 8, 2000.

PARCEL B1:

NON-EXCLUSIVE EASEMENT FOR VEHICULAR INGRESS AND EGRESS AND AN EXCLUSIVE EASEMENT FOR VEHICULAR PARKING BOTH LOCATED ON PARCEL 1 AS SHOWN ON THAT CERTAIN PARCEL MAP NO. 18598 FILED FOR RECORD IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO, CALIFORNIA ON DECEMBER 8, 2000 AS INSTRUMENT NO. 2000-0669213 OF OFFICIAL RECORDS, AND MORE PARTICULARLY DESCRIBED AS FOLLOWS: BEGINNING AT THE SOUTHWEST COMER OF PARCEL 2 OF SAID PARCEL MAP NO. 18598; THENCE NORTH 89°30'47" EAST, 61.83 FEET; THENCE SOUTH 00°29'13" EAST, 22.65 FEET; THENCE NORTH 89°30'47" EAST, 21.05 FEET; THENCE SOUTH 89°48'12" EAST, 32.11 FEET; THENCE NORTH 89°30'47" EAST, 24.39 FEET; THENCE NORTH 00°29'25" WEST, 0.77 FEET; THENCE NORTH 89°41'41" EAST, 43.83 FEET; THENCE SOUTH 00°00'00" EAST, 3.24 FEET; THENCE SOUTH 89°20'47" EAST, 16.85 FEET; THENCE SOUTH 01°55'06" WEST, 7.73 FEET; THENCE SOUTH 89°26'31" WEST, 17.31 FEET; THENCE SOUTH 62°50'47" EAST, 10.19 FEET; THENCE SOUTH 00°14'21" EAST, 28.04 FEET; THENCE NORTH 89°30'46" EAST, 9.94 FEET TO THE BEGINNING OF A NON-TANGENT 112.00 FOOT RADIUS CURVE, CONCAVE TO THE WEST, A RADIAL TO SAID POINT BEARS NORTH 60°20'01" EAST; THENCE SOUTHERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 39°02'05" AN ARC DISTANCE OF 76.30 FEET; THENCE SOUTH 09°22'06" WEST, 151.45 FEET TO THE BEGINNING OF A NON-TANGENT 76.00 FOOT RADIUS CURVE, CONCAVE TO THE NORTHWEST, A RADIAL TO SAID POINT BEARS SOUTH 80°37'58" EAST; THENCE SOUTHWESTERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 58°06'12" AN ARC DISTANCE OF 77.07 FEET, TO THE BEGINNING OF A NON-TANGENT 40.00 FOOT RADIUS CURVE, CONCAVE TO THE EAST, A RADIAL TO SAID POINT BEARS NORTH 22°31'46" WEST; THENCE SOUTHERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 158°25'34" AN ARC DISTANCE OF 110.60 FEET; THENCE NON-TANGENT TO SAID CURVE, SOUTH 17°38'59" WEST, 37.22 FEET; THENCE NORTH 40°28'08" WEST, 10.68 FEET TO THE BEGINNING OF A TANGENT 12.00 FOOT RADIUS CURVE, CONCAVE TO THE SOUTHWEST; THENCE NORTHWESTERLY, ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 28°41'16" AN ARC DISTANCE OF 6.01 FEET, TO THE BEGINNING OF A NON-TANGENT 66.00 FOOT RADIUS CURVE, CONCAVE TO THE NORTHEAST, A RADIAL TO SAID POINT BEARS SOUTH 20°52'25" WEST; THENCE NORTHWESTERLY, ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 42°53'20" AN ARC DISTANCE OF 49.40 FEET; THENCE NORTH 26°14'15 WEST, 2.50 FEET TO THE BEGINNING OF A NON-TANGENT 66.00 FOOT RADIUS CURVE, CONCAVE TO THE EAST, A RADIAL TO SAID POINT BEARS SOUTH 63°45'45" WEST; THENCE NORTHERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 93°42'29" AN ARC DISTANCE OF 107.94 FEET TO THE BEGINNING OF A NON-TANGENT 50.00 FOOT RADIUS CURVE, CONCAVE TO THE NORTHWEST, A RADIAL TO SAID POINT BEARS SOUTH 22°31'46" EAST; THENCE NORTHEASTERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 58°06'14" AN ARC DISTANCE OF 50.71 FEET; THENCE NON-TANGENT TO SAID CURVE, NORTH 09°22'06" EAST, 151.45 FEET TO THE BEGINNING OF A TANGENT 86.00 FOOT RADIUS CURVE, CONCAVE TO THE WEST; THENCE NORTHERLY, ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 23°13'57" AN ARC DISTANCE OF 34.87 FEET, TO THE BEGINNING OF A COMPOUND 21.29 FOOT RADIUS CURVE, CONCAVE TO THE SOUTHWEST, A RADIAL TO SAID POINT BEARS NORTH 76°08'09" EAST; THENCE NORTHWESTERLY, ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 51°11'03" AN ARC DISTANCE OF 19.02 FEET, TO THE BEGINNING OF A NON-TANGENT 163.26 FOOT RADIUS CURVE, CONCAVE TO THE SOUTH, A RADIAL TO SAID POINT BEARS NORTH 05°28'17" WEST; THENCE WESTERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 06°27'35" AN ARC DISTANCE OF 18.41 FEET; THENCE NON-TANGENT TO SAID CURVE, NORTH 90°00'00" WEST, 20.16 FEET; THENCE NORTH 18°57'00" WEST, 3.48 FEET; THENCE NORTH 66°55'41" WEST, 18.13 FEET TO THE BEGINNING OF A NON-TANGENT 5.23 FOOT RADIUS CURVE, CONCAVE TO THE SOUTHWEST, A RADIAL TO SAID POINT BEARS NORTH 88°32'47" EAST; THENCE NORTHWESTERLY, ALONG SAID CURVE THROUGH A CENTRAL ANGLE OF 78°34'49" AN ARC DISTANCE OF 7.18 FEET; THENCE NON-TANGENT TO SAID CURVE, SOUTH 88°14'36" WEST, 44.95 FEET; THENCE NORTH 89°32'05" WEST, 21.99 FEET; THENCE SOUTH 89°30'47" WEST, 140.58 FEET; THENCE NORTH 37°59'16" WEST, 15.15 FEET; THENCE SOUTH 84°53'27" WEST, 73.93 FEET TO THE BEGINNING OF A TANGENT 30.00 FOOT RADIUS CURVE, CONCAVE TO THE SOUTHEAST; THENCE SOUTHWESTERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE

LEGAL DESCRIPTIONS

continued

OF 41 °36'19" AN ARC DISTANCE OF 21.78 FEET, TO THE BEGINNING OF A REVERSE 20.00 FOOT RADIUS CURVE, CONCAVE TO THE NORTHWEST, A RADIAL TO SAID POINT BEARS SOUTH 46°42'52" EAST; THENCE SOUTHWESTERLY, ALONG SAID CURVE THROUGH A CENTRAL ANGLE OF 46°36'01" AN ARC DISTANCE OF 16.27 FEET; THENCE SOUTH 89°53'09" WEST, 64.30 FEET TO THE BEGINNING OF A TANGENT 64.00 FOOT RADIUS CURVE, CONCAVE TO THE NORTH; THENCE WESTERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 31°25'24" AN ARC DISTANCE OF 35.10 FEET; THENCE NON-TANGENT TO SAID CURVE, SOUTH 89°53'09" WEST, 118.48 FEET TO THE BEGINNING OF A TANGENT 22.50 FOOT RADIUS CURVE, CONCAVE TO THE SOUTHEAST; THENCE SOUTHWESTERLY, ALONG SAID CURVE THROUGH A CENTRAL ANGLE OF 63°17'24" AN ARC DISTANCE OF 24.85 FEET; THENCE NON-TANGENT TO SAID CURVE, SOUTH 89°53'09" WEST, 145.49 FEET; THENCE NORTH 11°24'39" EAST, 41.44 FEET; THENCE NORTH 89°53'09" EAST 209.60 FEET; THENCE NORTH 84°53'27" EAST, 214.54 FEET TO THE BEGINNING OF A TANGENT 21.50 FOOT RADIUS CURVE, CONCAVE TO THE NORTHWEST; THENCE NORTHEASTERLY ALONG SAID CURVE, THROUGH A CENTRAL ANGLE OF 51°55'41" AN ARC DISTANCE OF 19.49 FEET; THENCE NORTH 32°57'46" EAST, 23.37 FEET; THENCE NORTH 00°29'13" WEST, 53.09 FEET; THENCE NORTH 89°30'47" EAST, 37.80 FEET; THENCE SOUTH 00°29'13" EAST, 19.42 FEET TO THE BEGINNING OF A NON-TANGENT 90.00 FOOT RADIUS CURVE, CONCAVE TO THE SOUTHWEST, A RADIAL TO SAID POINT BEARS NORTH 15°59'30' EAST; THENCE SOUTHEASTERLY ALONG SAID CURVE THROUGH A CENTRAL ANGLE OF 31 °28'53" AN ARC DISTANCE OF 49.45 FEET; THENCE NON-TANGENT TO SAID CURVE, NORTH 89°30'47" EAST, 16.00 FEET TO THE WESTERLY LINE OF PARCEL 2; THENCE SOUTH 00°29'13" EAST, 8.75 FEET TO THE POINT OF BEGINNING.

PARCEL B2:

NON-EXCLUSIVE EASEMENT FOR VEHICULAR INGRESS AND EGRESS AND AN EXCLUSIVE EASEMENT FOR VEHICULAR PARKING BOTH LOCATED ON PARCEL 1 AS SHOWN ON THAT CERTAIN PARCEL MAP NO. 18598, FILED FOR RECORD IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, CALIFORNIA ON DECEMBER 8, 2000 AS INSTRUMENT NO. 2000-0669213 OF OFFICIAL RECORDS, AND MORE PARTICULARLY DESCRIBED AS FOLLOWS: BEGINNING AT THE SOUTHERLY TERMINUS OF THE MOST EASTERLY LINE OF PARCEL 2 OF SAID PARCEL MAP NO. 18598; THENCE SOUTH 00°09'49" EAST, 0.78 FEET; THENCE NORTH 89°50'11" EAST, 19.31 FEET; THENCE NORTH 00°06'41" WEST, 10.59 FEET; THENCE NORTH 89°30'51" EAST, 7.63 FEET; THENCE NORTH 00°29'13" WEST, 20.69 FEET; THENCE NORTH 88°18'18" WEST, 27.02 FEET; THENCE SOUTH 00°29'00" EAST, 31.42 FEET TO THE POINT OF BEGINNING.

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

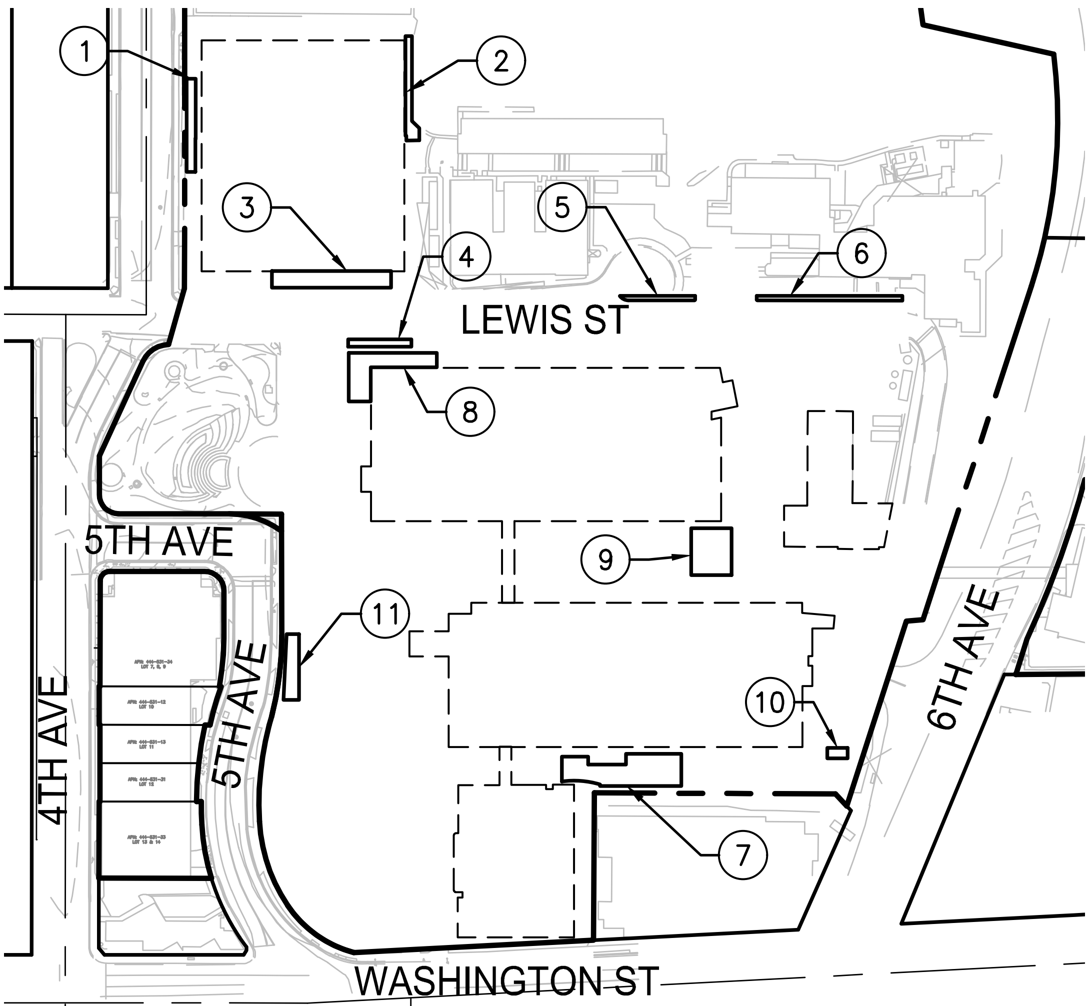
STORM WATER MANAGEMENT AND DISCHARGE CONTROL MAINTENANCE AGREEMENT APPROVAL NO.:

O&M RESPONSIBLE PARTY DESIGNEE: PROPERTY OWNER – SCRIPPS HEALTH

BMP DESCRIPTION	INSPECTION FREQUENCY	MAINTENANCE FREQUENCY	MAINTENANCE METHOD	QTY.	INCLUDED IN O&M MANUAL				SHEET NUMBER(S)
POLLUTANT CONTROL BMP(S):									
FLOW–THROUGH PLANTER	WEEKLY	WEEKLY	REMOVE TRASH/DEBRIS	10	X	YES		NO	CUP–11
FLOW–THROUGH PLANTER	WEEKLY	ANNUALLY	EVALUATE, PRUNE, REPLACE PLANT MATERIAL AS NEEDED	10	X	YES		NO	CUP–11
MODULAR WETLAND SYSTEM	WEEKLY	ANNUALLY	REMOVE TRASH FROM SCREENING DEVICE	1	X	YES		NO	CUP–11
MODULAR WETLAND SYSTEM	WEEKLY	BI–ANNUALLY	REMOVE SEDIMENT FROM SEPAR. CHMBR.	1	X	YES		NO	CUP–11
MODULAR WETLAND SYSTEM	WEEKLY	BI–ANNUALLY	REPLACE CARTRIDGE FILTER MEDIA	1	X	YES		NO	CUP–11
MODULAR WETLAND SYSTEM	WEEKLY	BI–ANNUALLY	REPLACE DRAIN DOWN FILTER MEDIA	1	X	YES		NO	CUP–11
MODULAR WETLAND SYSTEM	WEEKLY	ANNUALLY	TRIM VEGETATION	1	X	YES		NO	CUP–11

HMP EXEMPT	NO

EXHIBIT A: SITE AND VICINITY MAP WITH BMP LOCATIONS

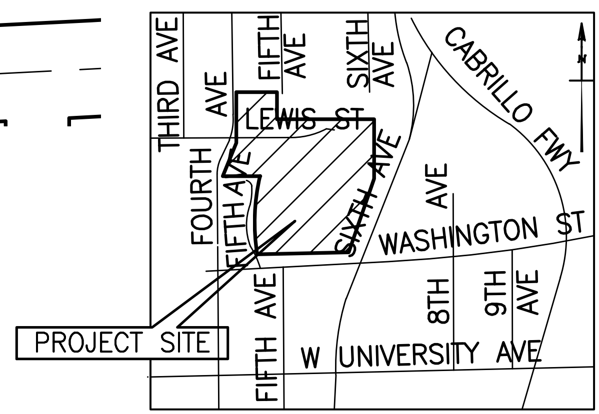


TREATMENT SYSTEMS

1. BMP 6A–1: 630 SF BIOFILTRATION
2. BMP 6B–1: 730 SF BIOFILTRATION
3. BMP 6C–1: 2100 SF BIOFILTRATION
4. BMP 8–1: 495 SF BIOFILTRATION
5. BMP 9–1: 339 SF BIOFILTRATION
6. BMP 7–1: 720 SF BIOFILTRATION
7. BMP 2–1: 2,900 SF BIOFILTRATION
8. BMP 5–1: 2,200 SF BIOFILTRATION
9. BMP 3–1: 1,900 SF BIOFILTRATION
10. BMP 1–1: MODULAR WETLAND SYSTEM (L–8–16–V)
11. BMP 4–1: 1,005 SF BIOFILTRATION

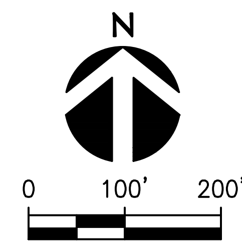
LEGEND

- PROPERTY LINE
- BUILDING OUTLINE
- BIOFILTRATION BASIN
- MODULAR WETLAND SYSTEM

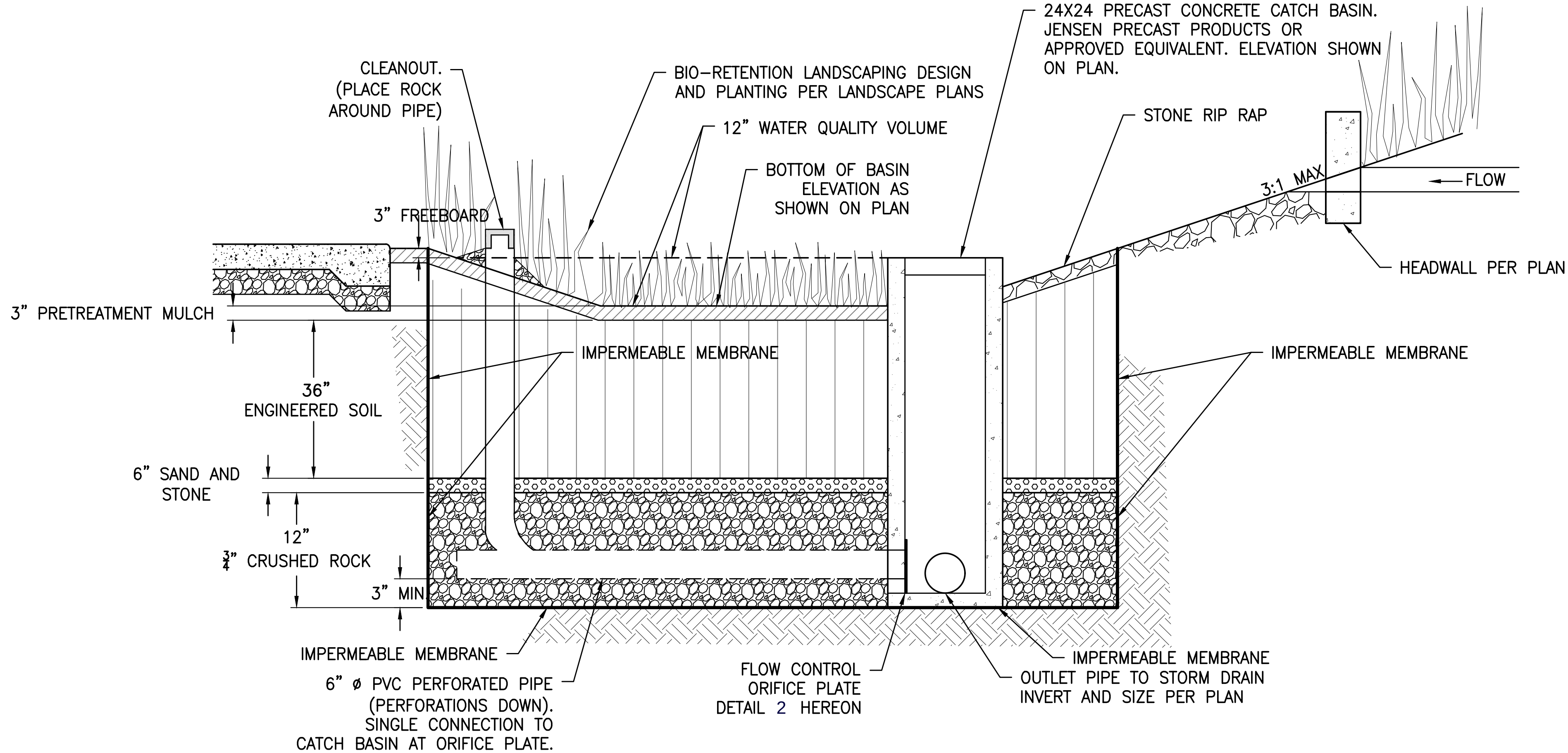


VICINITY MAP

NO SCALE



SCALE: 1"=200'

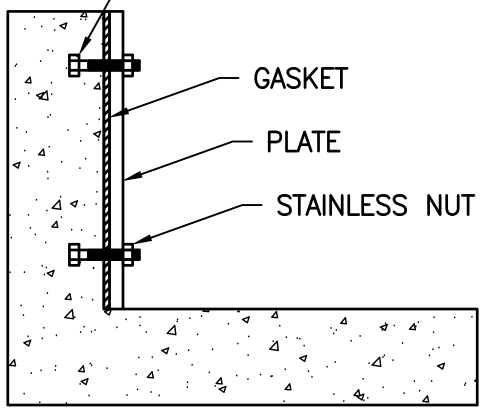


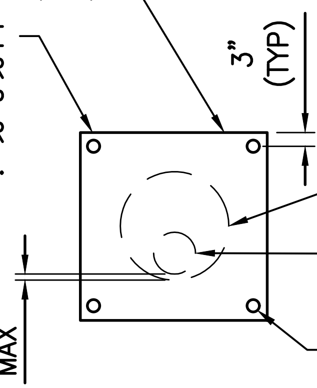
NOTES:

1. BIORETENTION ENGINEERED SOIL LAYER SHALL BE "SANDY LOAM" SOIL MIX WITH NO MORE THAN 5% CLAY CONTENT. THE MIX SHALL CONTAIN 50–60% SAND, 20–30% COMPOST OR HARDWOOD MULCH, AND 20–30% TOPSOIL.

1 BIO–FILTRATION SECTION

N.T.S.





NOTE:

1. ORIFICE PLATE & FLANGE CONNECTION TO CONCRETE SHALL BE FITTED WITH 30 DUROMETER NEOPRENE RING.

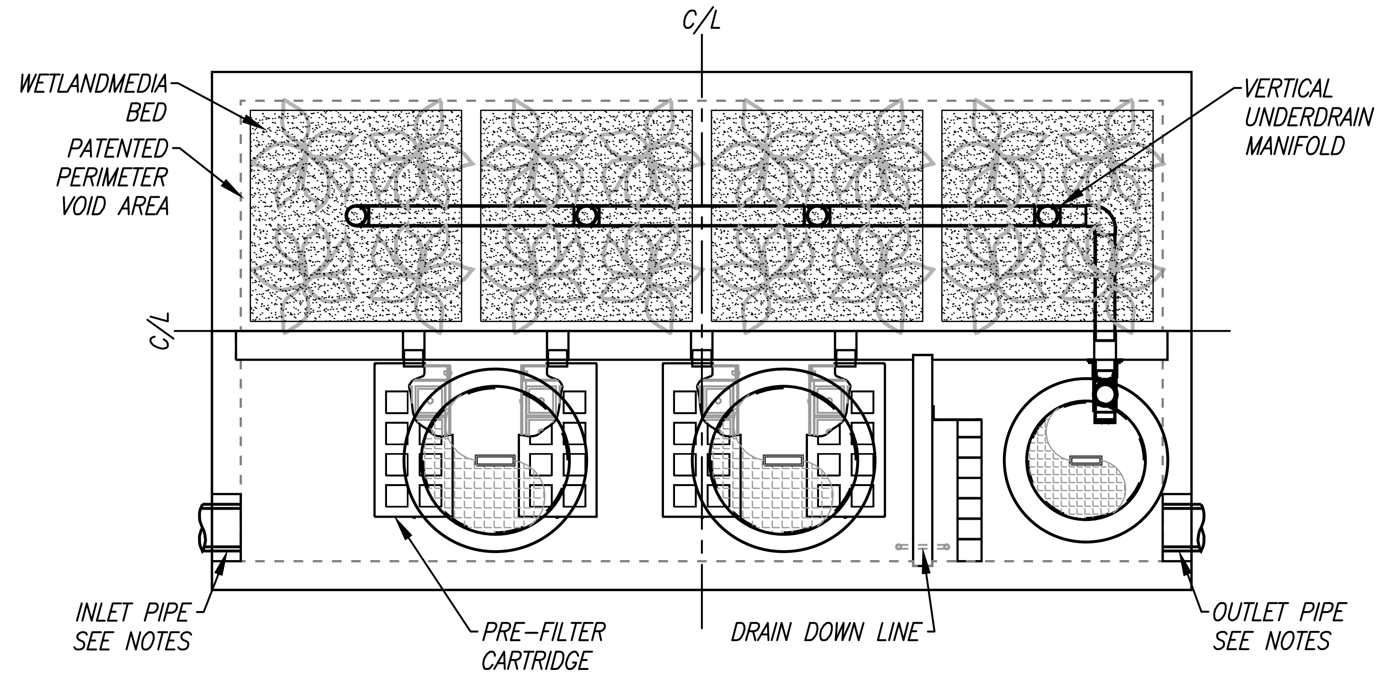
2. BASIN PERFORATED PIPES TO HAVE SINGLE CONNECTION TO CATCH BASIN AT ORIFICE PLATE.

2 **FLOW CONTROL ORIFICE**

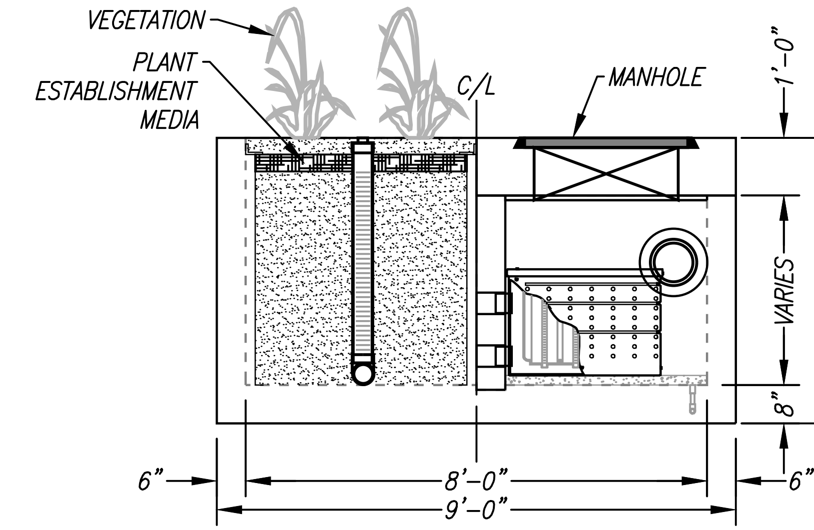
N.T.S.

SITE SPECIFIC DATA			
PROJECT NUMBER			
PROJECT NAME			
PROJECT LOCATION			
STRUCTURE ID			
TREATMENT REQUIRED			
VOLUME BASED (CF)		FLOW BASED (CFS)	
N/A		0.693	
PEAK BYPASS REQUIRED (CFS) – IF APPLICABLE			
PIPE DATA	I.E.	MATERIAL	DIAMETER
INLET PIPE 1			
INLET PIPE 2	N/A	N/A	N/A
OUTLET PIPE			
	PRETREATMENT	BIOFILTRATION	DISCHARGE
RIM ELEVATION			
SURFACE LOAD	PEDESTRIAN		
FRAME & COVER	2EA ø30"	OPEN PLANTER	ø24"
NOTES:			

* PRELIMINARY NOT FOR CONSTRUCTION



PLAN VIEW



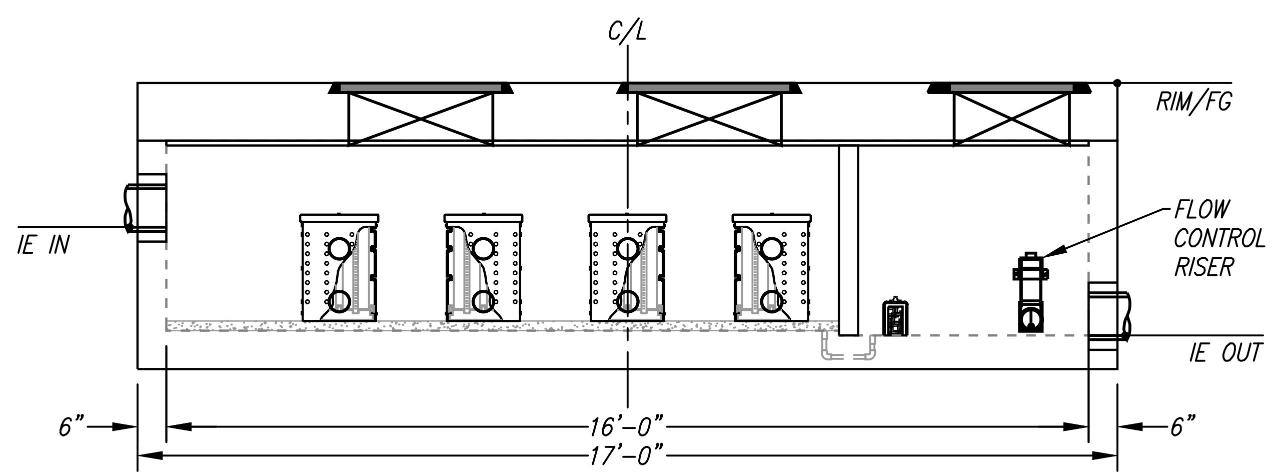
LEFT END VIEW

INSTALLATION NOTES

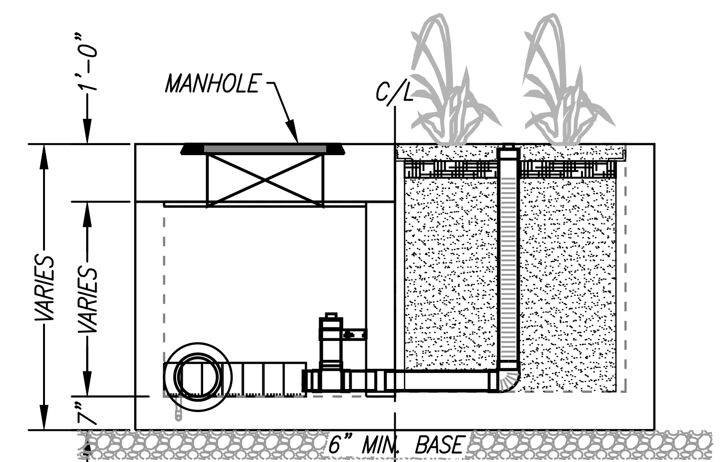
1. CONTRACTOR TO PROVIDE ALL LABOR, EQUIPMENT, MATERIALS AND INCIDENTALS REQUIRED TO OFFLOAD AND INSTALL THE SYSTEM AND APPURTENANCES IN ACCORDANCE WITH THIS DRAWING AND THE MANUFACTURERS SPECIFICATIONS, UNLESS OTHERWISE STATED IN MANUFACTURERS CONTRACT.
2. UNIT MUST BE INSTALLED ON LEVEL BASE. MANUFACTURER RECOMMENDS A MINIMUM 6" LEVEL ROCK BASE UNLESS SPECIFIED BY THE PROJECT ENGINEER. CONTRACTOR IS RESPONSIBLE TO VERIFY PROJECT ENGINEERS RECOMMENDED BASE SPECIFICATIONS.
4. CONTRACTOR TO SUPPLY AND INSTALL ALL EXTERNAL CONNECTING PIPES. ALL PIPES MUST BE FLUSH WITH INSIDE SURFACE OF CONCRETE. (PIPES CANNOT INTRUDE BEYOND FLUSH). INVERT OF OUTFLOW PIPE MUST BE FLUSH WITH DISCHARGE CHAMBER FLOOR. ALL PIPES SHALL BE SEALED WATER TIGHT PER MANUFACTURERS STANDARD CONNECTION DETAIL.
5. CONTRACTOR RESPONSIBLE FOR INSTALLATION OF ALL RISERS, MANHOLES, AND HATCHES. CONTRACTOR TO GROUT ALL MANHOLES AND HATCHES TO MATCH FINISHED SURFACE UNLESS SPECIFIED OTHERWISE.
6. VEGETATION SUPPLIED AND INSTALLED BY OTHERS. ALL UNITS WITH VEGETATION MUST HAVE DRIP OR SPRAY IRRIGATION SUPPLIED AND INSTALLED BY OTHERS.
7. CONTRACTOR RESPONSIBLE FOR CONTACTING BIO CLEAN FOR ACTIVATION OF UNIT. MANUFACTURERS WARRANTY IS VOID WITH OUT PROPER ACTIVATION BY A BIO CLEAN REPRESENTATIVE.

GENERAL NOTES

1. MANUFACTURER TO PROVIDE ALL MATERIALS UNLESS OTHERWISE NOTED.
2. ALL DIMENSIONS, ELEVATIONS, SPECIFICATIONS AND CAPACITIES ARE SUBJECT TO CHANGE. FOR PROJECT SPECIFIC DRAWINGS DETAILING EXACT DIMENSIONS, WEIGHTS AND ACCESSORIES PLEASE CONTACT BIO CLEAN.



ELEVATION VIEW



RIGHT END VIEW

TREATMENT FLOW (CFS)	0.693
OPERATING HEAD (FT)	
PRETREATMENT LOADING RATE (GPM/SF)	2.0
WETLAND MEDIA LOADING RATE (GPM/SF)	1.0

MODULAR WETLANDS

THIS PRODUCT MAY BE PROTECTED BY ONE OR MORE OF THE FOLLOWING US PATENTS: 7,425,262; 7,470,362; 7,674,378; 8,303,816; RELATED FOREIGN PATENTS OR OTHER PATENTS PENDING

Bio Clean
A Forterra Company

MWS-L-8-16-V
STORMWATER BIOFILTRATION SYSTEM
STANDARD DETAIL

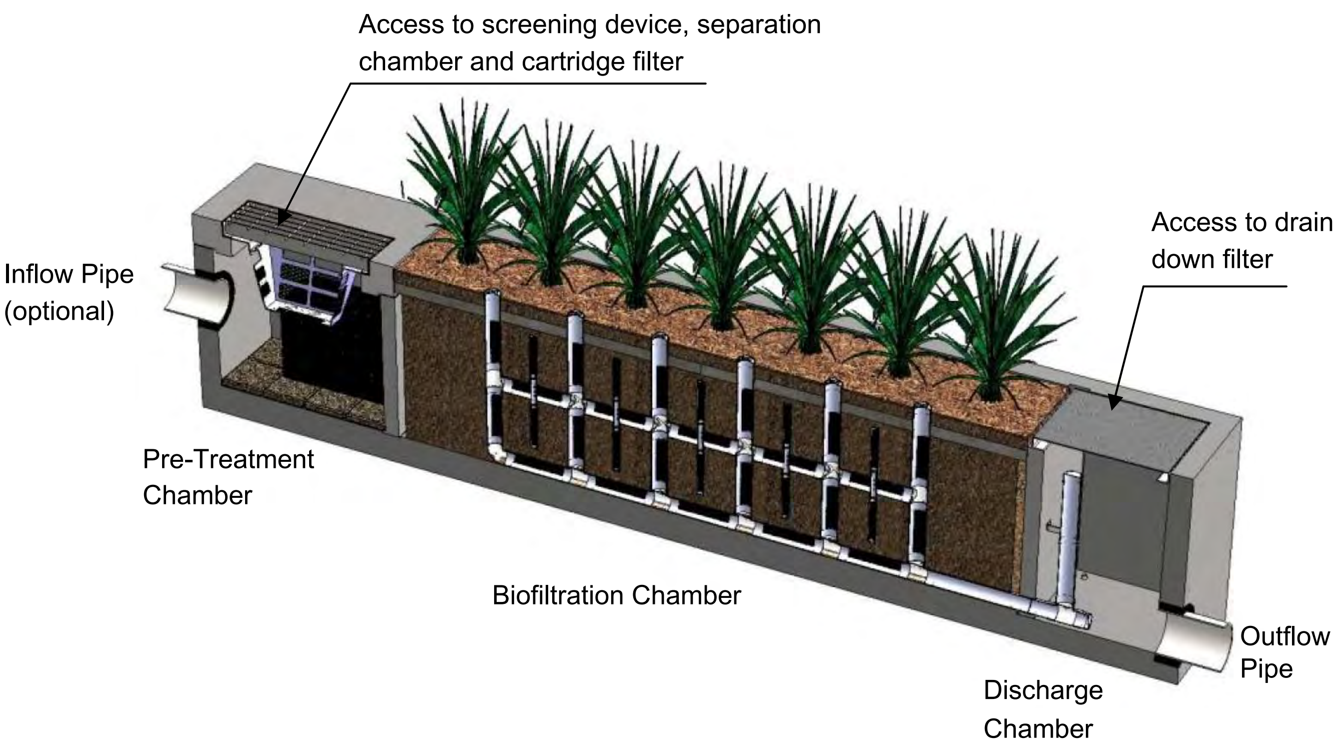
5/23/19TOLFE

Maintenance Guidelines for
Modular Wetland System - Linear

Maintenance Summary

- Remove Trash from Screening Device – average maintenance interval is 6 to 12 months.
    - (*5 minute average service time*).
- Remove Sediment from Separation Chamber – average maintenance interval is 12 to 24 months.
    - (*10 minute average service time*).
- Replace Cartridge Filter Media – average maintenance interval 12 to 24 months.
    - (*10-15 minute per cartridge average service time*).
- Replace Drain Down Filter Media – average maintenance interval is 12 to 24 months.
    - (*5 minute average service time*).
- Trim Vegetation – average maintenance interval is 6 to 12 months.
    - (*Service time varies*).

System Diagram



Maintenance Procedures

Screening Device

1. Remove grate or manhole cover to gain access to the screening device in the Pre-Treatment Chamber. Vault type units do not have screening device. Maintenance can be performed without entry.
2. Remove all pollutants collected by the screening device. Removal can be done manually or with the use of a vacuum truck. The hose of the vacuum truck will not damage the screening device.
3. Screening device can easily be removed from the Pre-Treatment Chamber to gain access to separation chamber and media filters below. Replace grate or manhole cover when completed.

Separation Chamber

1. Perform maintenance procedures of screening device listed above before maintaining the separation chamber.
2. With a pressure washer spray down pollutants accumulated on walls and cartridge filters.
3. Vacuum out Separation Chamber and remove all accumulated pollutants. Replace screening device, grate or manhole cover when completed.

Cartridge Filters

1. Perform maintenance procedures on screening device and separation chamber before maintaining cartridge filters.
2. Enter separation chamber.
3. Unscrew the two bolts holding the lid on each cartridge filter and remove lid.
4. Remove each of 4 to 8 media cages holding the media in place.
5. Spray down the cartridge filter to remove any accumulated pollutants.
6. Vacuum out old media and accumulated pollutants.
7. Reinstall media cages and fill with new media from manufacturer or outside supplier. Manufacturer will provide specification of media and sources to purchase.
8. Replace the lid and tighten down bolts. Replace screening device, grate or manhole cover when completed.

Drain Down Filter

1. Remove hatch or manhole cover over discharge chamber and enter chamber.
2. Unlock and lift drain down filter housing and remove old media block. Replace with new media block. Lower drain down filter housing and lock into place.
3. Exit chamber and replace hatch or manhole cover.

Maintenance Notes

1. Following maintenance and/or inspection, it is recommended the maintenance operator prepare a maintenance/inspection record. The record should include any maintenance activities performed, amount and description of debris collected, and condition of the system and its various filter mechanisms.

2. The owner should keep maintenance/inspection record(s) for a minimum of five years from the date of maintenance. These records should be made available to the governing municipality for inspection upon request at any time.

3. Transport all debris, trash, organics and sediments to approved facility for disposal in accordance with local and state requirements.

4. Entry into chambers may require confined space training based on state and local regulations.

5. No fertilizer shall be used in the Biofiltration Chamber.

6. Irrigation should be provided as recommended by manufacturer and/or landscape architect. Amount of irrigation required is dependent on plant species. Some plants may require irrigation.

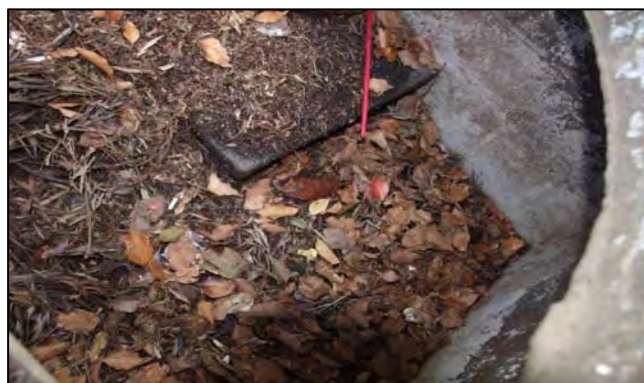
Maintenance Procedure Illustration

Screening Device

The screening device is located directly under the manhole or grate over the Pre-Treatment Chamber. It's mounted directly underneath for easy access and cleaning. Device can be cleaned by hand or with a vacuum truck.

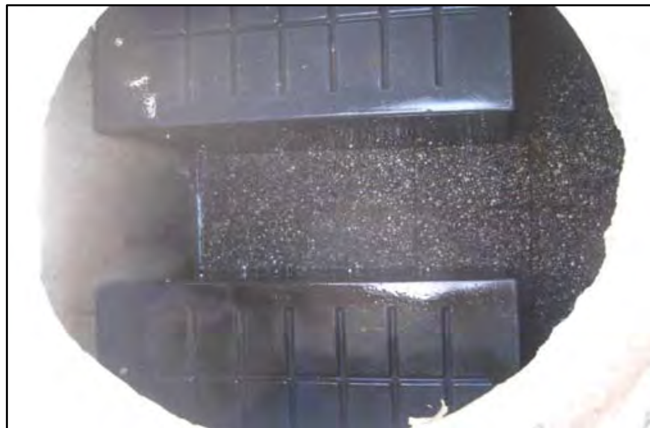
Separation Chamber

The separation chamber is located directly beneath the screening device. It can be quickly cleaned using a vacuum truck or by hand. A pressure washer is useful to assist in the cleaning process.



Cartridge Filters

The cartridge filters are located in the Pre-Treatment chamber connected to the wall adjacent to the biofiltration chamber. The cartridges have removable tops to access the individual media filters. Once the cartridge is open media can be easily removed and replaced by hand or a vacuum truck.



Drain Down Filter

The drain down filter is located in the Discharge Chamber. The drain filter unlocks from the wall mount and hinges up. Remove filter block and replace with new block.

Trim Vegetation

Vegetation should be maintained in the same manner as surrounding vegetation and trimmed as needed. No fertilizer shall be used on the plants. Irrigation per the recommendation of the manufacturer and or landscape architect. Different types of vegetation requires different amounts of irrigation.

Inspection Form

Modular Wetland System, Inc.
P. 760.433-7640
F. 760-433-3176
E. Info@modularwetlands.com

www.modularwetlands.com

Inspection Report
Modular Wetlands System

Project Name ______________________

Project Address ______________________ (city) (Zip Code)

Owner / Management Company ______________________

Contact ______________________ Phone ( ) –

Inspector Name ______________________ Date ___/___/_____ Time ________ AM / PM

Type of Inspection ☐ Routine ☐ Follow Up ☐ Complaint ☐ Storm Storm Event in Last 72-hours? ☐ No ☐ Yes

Weather Condition ______________________ Additional Notes ______________________

For Office Use Only
(Reviewed By)
(Date) Office personnel to complete section to the left.

Inspection Checklist

Modular Wetland System Type (Curb, Grate or UG Vault): __________ Size (22', 14' or etc.): __________

Structural Integrity:	Yes	No	Comments
Damage to pre-treatment access cover (manhole cover/grate) or cannot be opened using normal lifting pressure?			
Damage to discharge chamber access cover (manhole cover/grate) or cannot be opened using normal lifting pressure?			
Does the MWS unit show signs of structural deterioration (cracks in the wall, damage to frame)?			
Is the inlet/outlet pipe or drain down pipe damaged or otherwise not functioning properly?			
Working Condition:			
Is there evidence of illicit discharge or excessive oil, grease, or other automobile fluids entering and clogging the unit?			
Is there standing water in inappropriate areas after a dry period?			
Is the filter insert (if applicable) at capacity and/or is there an accumulation of debris/trash on the shelf system?			
Does the depth of sediment/trash/debris suggest a blockage of the inflow pipe, bypass or cartridge filter? If yes specify which one in the comments section. Note depth of accumulation in in pre-treatment chamber.			Depth:
Does the cartridge filter media need replacement in pre-treatment chamber and/or discharge chamber?			Chamber:
Any signs of improper functioning in the discharge chamber? Note issues in comments section.			
Other Inspection Items:			
Is there an accumulation of sediment/trash/debris in the wetland media (if applicable)?			
Is it evident that the plants are alive and healthy (if applicable)? Please note Plant Information below.			
Is there a septic or foul odor coming from inside the system?			

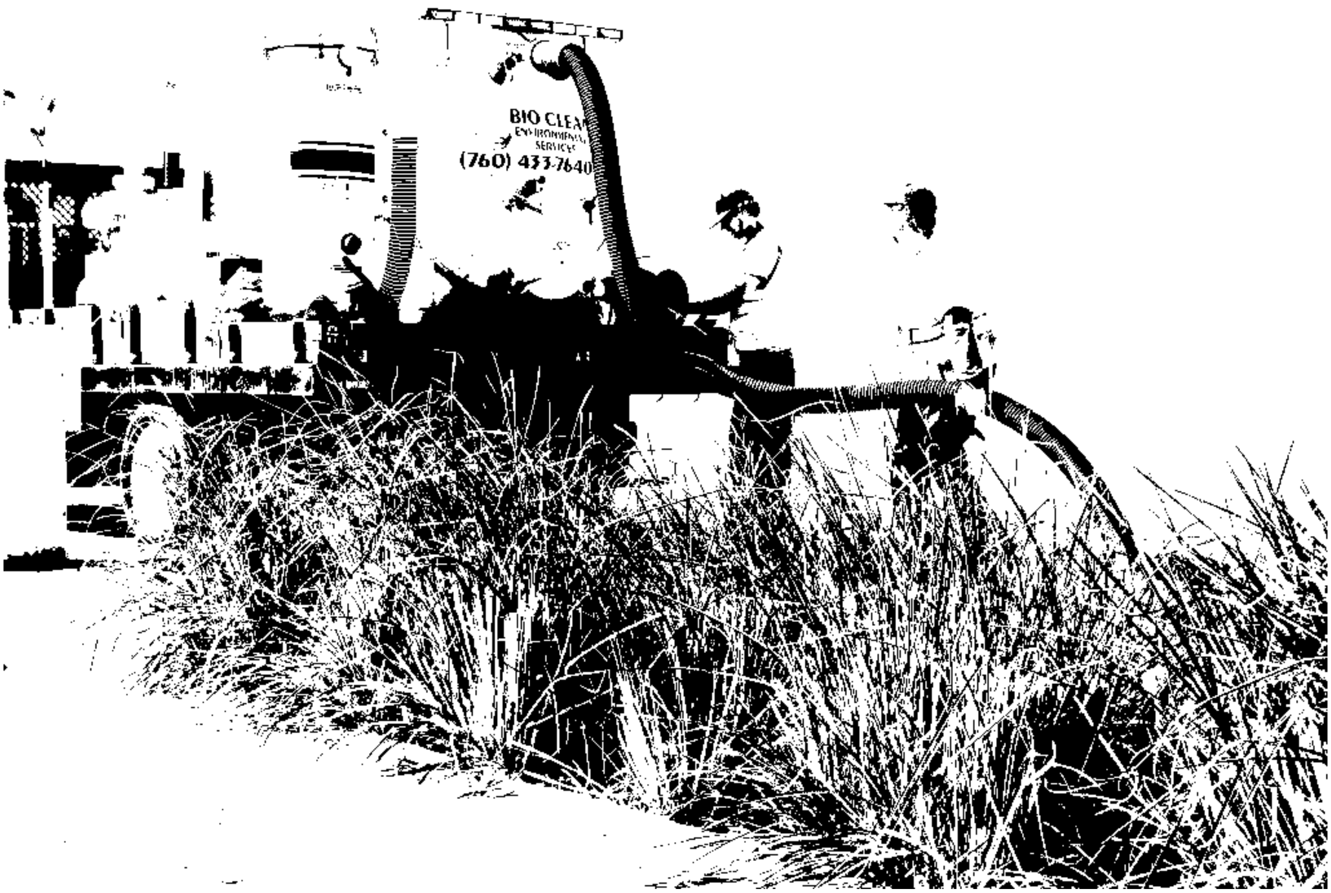
Waste:	Yes	No
Sediment / Silt / Clay		
Trash / Bags / Bottles		
Green Waste / Leaves / Foliage		

Recommended Maintenance	
No Cleaning Needed	
Schedule Maintenance as Planned	
Needs Immediate Maintenance	

Plant Information	
Damage to Plants	
Plant Replacement	
Plant Trimming	

Additional Notes: ______________________

Maintenance Report



Modular Wetland System, Inc.
P. 760.433-7640
F. 760-433-3176
E. Info@modularwetlands.com

www.modularwetlands.com

Cleaning and Maintenance Report
Modular Wetlands System

Project Name ______________________

Project Address ______________________
(city) (Zip Code)

Owner / Management Company ______________________

Contact ______________________ Phone ( ) –

Inspector Name ______________________ Date ___/___/_____ Time __________ AM / PM

Type of Inspection ☐ Routine ☐ Follow Up ☐ Complaint ☐ Storm   Storm Event in Last 72-hours? ☐ No ☐ Yes

Weather Condition ______________________ Additional Notes ______________________

For Office Use Only
(Reviewed By)
(Date)
Office personnel to complete section to the left.

Site Map #	GPS Coordinates of Insert	Manufacturer / Description / Sizing	Trash Accumulation	Foliage Accumulation	Sediment Accumulation	Total Debris Accumulation	Condition of Media 25/50/75/100 (will be changed @ 75%)	Operational Per Manufactures' Specifications (If not, why?)
	Lat: Long:	MWS Catch Basins						
		MWS Sedimentation Basin						
		Media Filter Condition						
		Plant Condition						
		Drain Down Media Condition						
		Discharge Chamber Condition						
		Drain Down Pipe Condition						
		Inlet and Outlet Pipe Condition						

Comments:

Attachment 4
Copy of Plan Sheets Showing Permanent Storm Water BMPs

This is the cover sheet for Attachment 4.

Project Name: Scripps Mercy Hospital

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

The plans must identify:

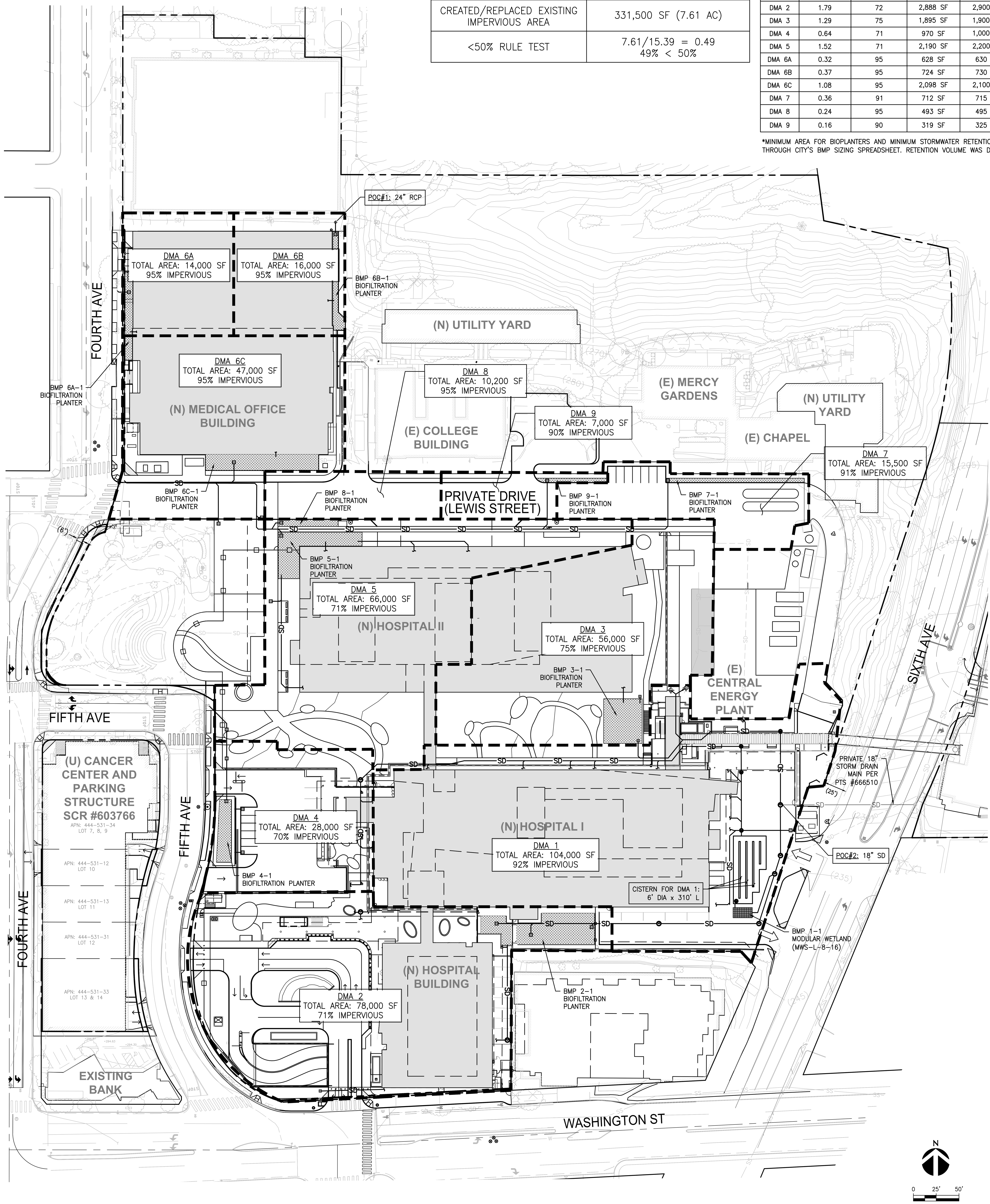
- ☑ Structural BMP(s) with ID numbers matching Form I-6 Summary of PDP Structural BMPs
- ☑ The grading and drainage design shown on the plans must be consistent with the delineation of DMAs shown on the DMA exhibit
- ☑ Details and specifications for construction of structural BMP(s)
- ☑ Signage indicating 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)
- ☑ 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
- ☑ 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
- ☑ When proprietary BMPs are used, site specific cross section with outflow, inflow and model number shall be provided. Broucher photocopies are not allowed.

PROJECT AREA (EXISTING IMPERVIOUS AREA)	670,400 SF (15.39 AC)
AREA TO BE DISTURBED BY PROJECT	460,000 SF (10.56 AC)
CREATED/REPLACED EXISTING IMPERVIOUS AREA	331,500 SF (7.61 AC)
<50% RULE TEST	7.61/15.39 = 0.49 49% < 50%

DMA SUMMARY TABLE

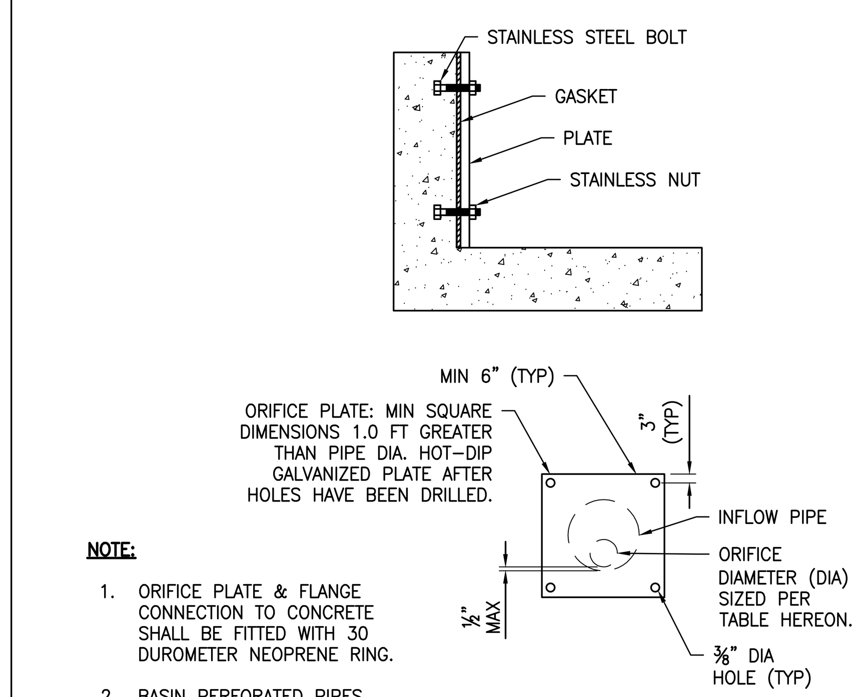
DMA	AREA (AC.)	IMPERVIOUS%	REQ. BMP CAPACITY*	PROP. BMP CAPACITY	BMP ID	ORIFICE SIZE (IN.)	MAX ORIFICE OUFLOW (CFS)	DRAINS TO (POC ID)
DMA 1	2.39	92	8,712 CF	8,760 CF	BMP 1–1	3 1/8"	0.520	#2
DMA 2	1.79	72	2,888 SF	2,900 SF	BMP 2–1	2 1/2"	0.344	#2
DMA 3	1.29	75	1,895 SF	1,900 SF	BMP 3–1	2 1/8"	0.249	#2
DMA 4	0.64	71	970 SF	1,000 SF	BMP 4–1	1 1/2"	0.124	#1
DMA 5	1.52	71	2,190 SF	2,200 SF	BMP 5–1	2 3/8"	0.311	#1
DMA 6A	0.32	95	628 SF	630 SF	BMP 6A–1	1"	0.055	#1
DMA 6B	0.37	95	724 SF	730 SF	BMP 6B–1	1 1/8"	0.065	#1
DMA 6C	1.08	95	2,098 SF	2,100 SF	BMP 6C–1	2"	0.220	#1
DMA 7	0.36	91	712 SF	715 SF	BMP 7–1	1 1/8"	0.070	#1
DMA 8	0.24	95	493 SF	495 SF	BMP 8–1	7/8"	0.042	#1
DMA 9	0.16	90	319 SF	325 SF	BMP 9–1	3/4"	0.031	#1

*MINIMUM AREA FOR BIOPLANTERS AND MINIMUM STORMWATER RETENTION VOLUME FOR MODULAR WETLAND SYSTEMS. BIOFILTRATION WAS SIZED THROUGH CITY'S BMP SIZING SPREADSHEET. RETENTION VOLUME WAS DETERMINED THROUGH HYDROLOGIC CONTINUOUS SOFTWARE, SDHM 3.1.



LEGEND:

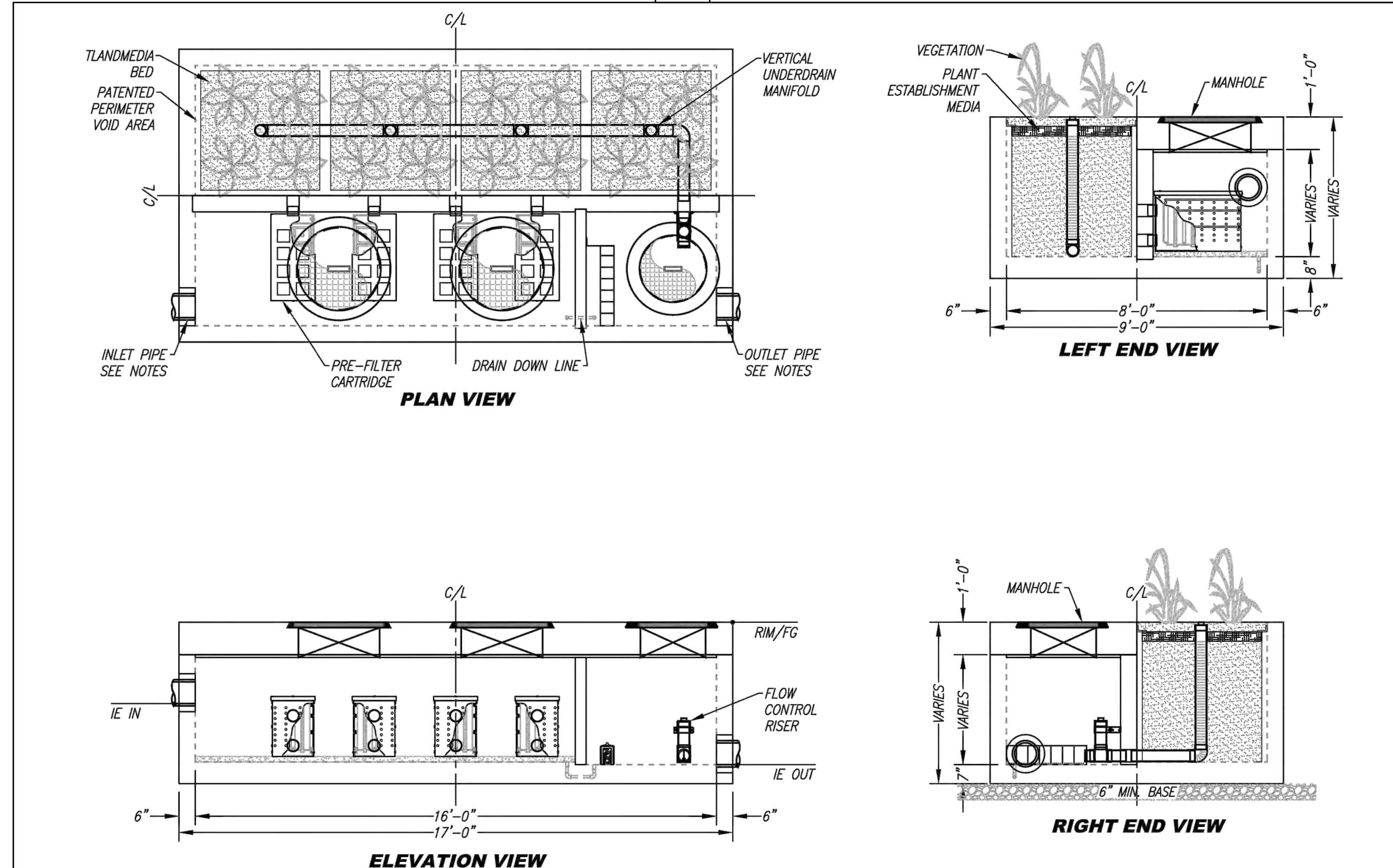
- PROPERTY LINE
- TRIBUTARY AREA
- BIO-FILTRATION PLANTER LOCATION
- MODULAR WETLAND SYSTEM



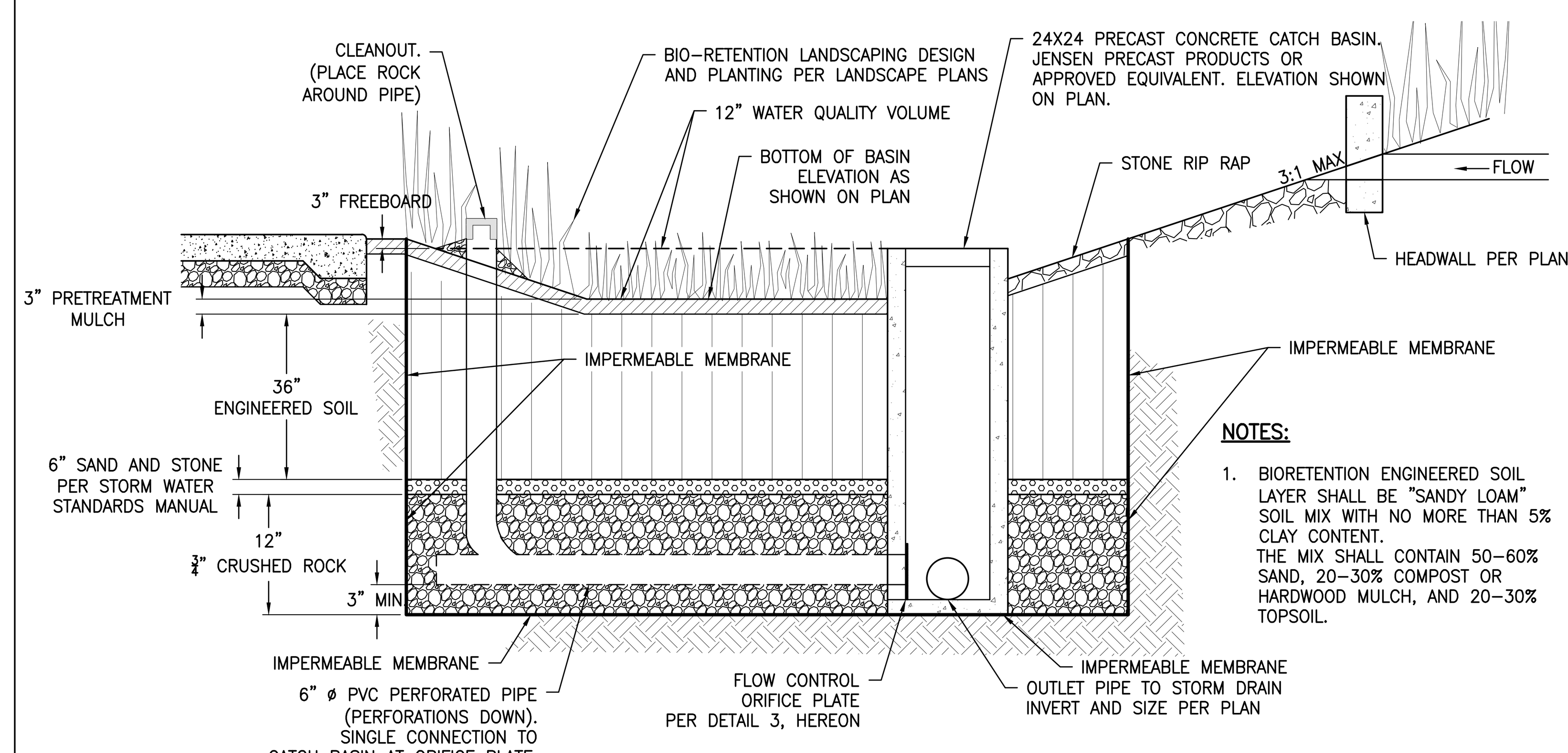
NOTE:

1. ORIFICE PLATE & FLANGE CONNECTION TO CONCRETE SHALL BE FITTED WITH 30 DUROMETER NEOPRENE RING.
2. BASIN PERFORATED PIPES TO HAVE SINGLE CONNECTION TO CATCH BASIN AT ORIFICE PLATE.

3 FLOW CONTROL ORIFICE PLATE N.T.S.



2 MWS-L-8-16-V N.T.S.



NOTES:

1. BIORETENTION ENGINEERED SOIL LAYER SHALL BE "SANDY LOAM" SOIL MIX WITH NO MORE THAN 5% CLAY CONTENT. THE MIX SHALL CONTAIN 50-60% SAND, 20-30% COMPOST OR HARDWOOD MULCH, AND 20-30% TOPSOIL.

1 TYPICAL BIO-FILTRATION SECTION N.T.S.

CO ARCHITECTS
5055 Wilshire Boulevard, 9th Floor
Los Angeles, California 90036
323.525.0500 phone, 323.525.0955 fax

PROFESSIONAL STAMP

REGISTERED PROFESSIONAL ENGINEER JEFFREY B. GANZA No. 59894 Exp. 3/31/23 CIVIL STATE OF CALIFORNIA

AGENCY STAMP

kpff
700 South Flower Street Suite 2100 Los Angeles, CA 90017
O: 213.418.0201
F: 213.266.5294
www.kpff.com

REVISIONS

1	4/10/20	REV 1
2	8/14/20	REV 2
3	11/18/20	REV 3
4	3/26/21	REV 4
5	6/11/21	REV 5
6	3/4/22	REV 6
7	5/27/22	REV 7

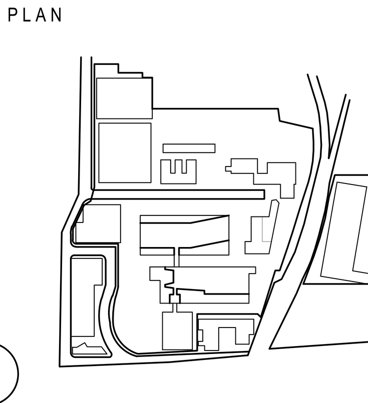
Scripps

CONDITIONAL USE PERMIT MERCY CAMPUS

4077 Fifth Ave, San Diego, CA 92103

SCRIPPS# 35-16060B DESIGN TEAM # 18003.000

KEY PLAN



STORMWATER QUALITY MANAGEMENT PLAN

SCALE:
DATE OF ISSUE: 5/27/22

CUP-11

Attachment 5
Drainage Report

Attach project's drainage report. Refer to Drainage Design Manual to determine the reporting requirements.

DRAINAGE STUDY FOR SCRIPPS MERCY HOSPITAL

CONDITIONAL USE PERMIT
(PTS# 658548)

SAN DIEGO, CALIFORNIA

June 2022

Prepared for:
SCRIPPS HEALTH
10140 Campus Point Drive, Suite 210
San Diego, California 92121
(858) 678-7080

Prepared By:

KPFF CONSULTING ENGINEERS
LA: 700 South Flower Street, Suite 2100
Los Angeles, CA 90017

SD: 3131 Camino Del Rio North, Suite 1080
San Diego, CA 92108

KPFF Job #1700865
(213) 418 – 0201

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1. Project Location and Scope

1.1 Project Location

The 17.7-acres Scripps Mercy Memorial Campus is located at the northeasterly corner of Washington Street and Fifth Ave, in the City of San Diego, California. The CUP project site is generally bound by Mercy Canyon to the north, Washington Street to the south, Fourth Avenue to the west, and Sixth Avenue to the east. Access to the project site is provided off of Lewis Street, Fifth Avenue, and Sixth Avenue. A site vicinity map is shown in Figure 1 below.

Figure 1-1: Site Vicinity Map

1.2 Scope of Report

This report will focus on identifying the hydrologic and hydraulic effects of the proposed development, by studying the 10-year and 100-year flow rates for the pre and post development conditions. This report will not discuss water quality measures or best management practices for stormwater mitigation. For information regarding best management practice requirements and implementation, refer to the project Storm Water Quality Management Plan (SWQMP).

No surface waters are present on the project site or nearby, and site runoff is captured and discharged into an onsite private storm drain system. As such, the project is not anticipated to require a separate CA Regional Water Quality Control Board approval under Federal Clean Water Act Section 401/404.

2. Study Objectives

The specific objectives of this drainage study are:

- Calculate the pre and post development peak flow rates for the 10-year and 100-year storm events.

- Determine the capacity of the proposed off-site storm drain infrastructure under post development conditions.

- Calculate the effects of the post development conditions on the existing hydrology and hydraulics for the 50-year storm events.

- Identify pre and post development areas of concern.

3. Project Description

3.1 Project Site Information

The existing site elevation varies from roughly 289 feet along the northern boundary (Lewis Street) to approximately 233 feet along the southeasterly boundary (Sixth Avenue).

The Federal Emergency Management Agency (FEMA) has not mapped any Special Flood Hazard Areas (SFHAs) for the project site. The FEMA Map for the project site is provided in Appendix A.

3.2 Pre-Development Conditions

The existing site infrastructure includes a college building, parking structures, surface parking lots, medical office buildings, emergency department facilities, and the main hospital building. In the pre developed condition, the site consists of approximately of 74% impervious surface, with no expected off-site drainage. The pre development condition is divided into 3 basins per existing grading and site features: Basin 1, Basin 2, & Basin 3.

Basin 1 consists of the drainage produced from the two multi-level parking structure on the northern part of the site along Fourth Avenue, Lewis Street, emergency department, college building, the main hospital building, and Mercy Canyon. Stormwater from Basin 1 is collected within two catch basins on the west end of Lewis Street and connects to a 24" RCP running along Fourth Avenue, then between the two parking structures. The 24" RCP discharges as a surface outfall into Mercy Canyon on the northern part of the project site. Refer to Figure 3-1 for a view of the existing catch basins on Lewis Street.

Basin 2 contains the drainage produced from the behavioral health clinic, central energy plant, 550 MOB parking structure, surface parking lots, and a portion of the main hospital building. Stormwater from Basin 2 is collected in downspouts from buildings and surface area drains in the parking lots and landscape areas. The collected runoff leaves the site via an 18" RCP, which travels north in Sixth Avenue.

Basin 3 consists of the drainage produced from the 550 Medical Office Building (MOB) and surrounding landscape area. Drainage from the building is collected in the building downspouts and northern street gutter on Washington Street. Refer to Figure 3-2 for a view of the catch basin on Washington Street.

Figure 3-1: Site Photo – two catch basins on west end of Lewis Street

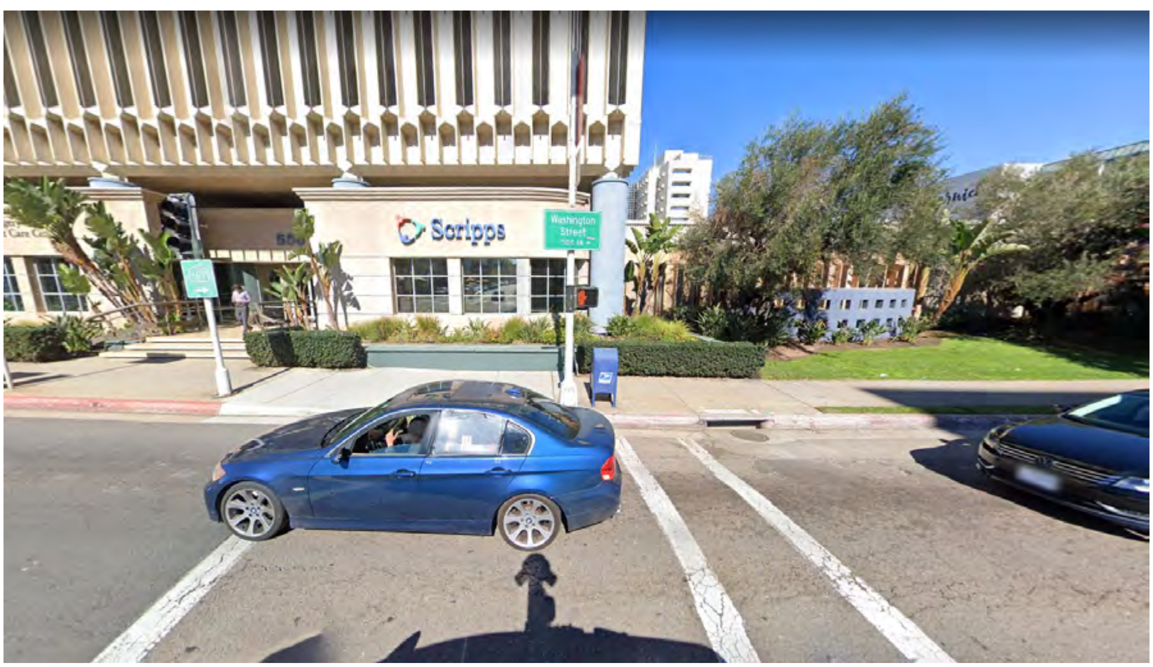


Figure 3-2: Site Photo – catch basin on north side of Washington Street

3.3 Post Development Conditions

The post development will consist of two phases of construction. Phase 1 will commence with the demolition of 550 MOB, the underground parking garage, and the Behavioral Health Unit. Phase 1 demolition will be followed by the construction of MOB, Replacement Hospital 1, and Hospital Support Building (HSB). Phase 2 will commence with the demolition of the existing hospital, and parking structure at the northeast corner of Fourth Avenue and Lewis Street. Phase 2 demolition is then followed by the construction of Replacement Hospital 2. In the post development condition, the site consists of approximately 67% impervious surface; a 7% reduction in imperviousness when compared to the pre development conditions. The post development condition is divided into 2 basins per the proposed grading and site features: Basin 1, Basin 2.

Basin 1 entails the drainage produced from the existing north parking structure, proposed Medical Office Building, west side of Replacement Hospital 2, existing college building, existing Mercy Manor, and surface runoff from Lewis Street. Stormwater from Basin 1 passes through biofiltration planters scattered onsite. Treated stormwater from Basin 1 will discharge to an existing 24” RCP public main on 4^{th} Ave, ultimately leading to a surface outfall to Mercy Canyon in the northern part of the site.

Basin 2 consists of the drainage produced from the proposed Replacement Hospital 1 and 2, HSB & HSB Plaza, and proposed loading dock. Stormwater from Basin 2 passes through biofiltration planter, both traditional and compact form, then discharges into a private 18” storm drain main across Sixth Ave, which will replace an existing public 18” RCP storm drain main.

4. Methodology

4.1 Hydrology

The hydrology calculations are based on the City of San Diego Drainage Design Manual (January 2017). The project site is less than one square mile, and therefore the Rational Method was used to calculate the peak flow rate for the 10-year and 100-year storm events. The Rational Method calculates peak flow rate (Q) as a function of runoff coefficient (C), rainfall intensity (I), and drainage area (A):

$$Q = C * I * A$$

Table A-1: Runoff Coefficients for Rational Method in the Drainage Design Manual is used to compute the runoff coefficients for the development conditions given the site’s imperviousness, soil type, and land use. The site’s imperviousness was determined by calculating the impervious area in the pre and post development conditions. Per the Drainage Design Manual, all sites are assumed to be made up of Type D soil. The project’s land use could be considered Commercial; however Industrial land use was assumed as a conservative approach to calculating the site’s peak flow rate.

Rainfall intensities were determined from Figure A-1: Intensity-Duration-Frequency Design Chart in the Drainage Design Manual. The design chart takes into consideration the time of concentration (Tc) and storm event frequency to calculate the rainfall intensity.

Drainage area was determined by inspecting the existing and proposed conditions and delineating areas according to grading and site features. The Pre-Development Drainage Condition and Post Development Drainage Condition maps can be found in Appendix B and C.

4.2 Hydraulics

The hydraulic calculation was conducted using Flowmaster software. Please refer to Appendix D for Hydraulic Calculations. The private storm drain within the project limit are designed to convey the peak runoff rate for a 50-year storm. The hydraulic calculations for 2 segments of storm drain pipes are summarized in Table 4-1.

Pipe ID	Size	Slope	Q_{50} (cfs)	Q_{full} (cfs)
SD 1	24”	2%	17.14	41.59
SD 2	18”	3%	18.35	23.65

Table 4-1: Hydraulic Calculation Summary

5. Results and Conclusions

5.1 Results

Table 5-1 and Table 5-2 summarize the hydrology results of the pre and post development conditions given the 10-year storm event frequency. The proposed development will increase the amount of pervious area and thus reduce the project site peak flow runoff. As seen in Table 5-1 and Table 5-2, the peak flow runoff rate for the 10-year storm event decreased from 37.6 cfs to 33.5 cfs in the pre and post development conditions. This represents a roughly 12% decrease in the peak runoff flow rate.

Pre-Development (10-Year)										
Drainage Area No.	Area (acres)	Runoff Coefficient (C)(1)	Time of Concentration (T_c)				Tc (min) (2)	I_{10} (in/hr) (3)	V_{10} (ft/s)	Q_{10} (cfs)
			US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)				
BASIN 1	11.50	0.68	291.4	283.8	475	1.6	14.2	2.2	12.59	17.1
BASIN 2	5.72	0.96	291.3	290.4	120	0.8	5.0	3.4	18.15	18.7
BASIN 3	0.55	0.96	290.5	290.0	100	0.5	5.0	3.4	5.18	1.8
Total	**17.77**	-	-	-	-	-	-	-	-	**37.6**

Table 5-1: Hydrologic Summary for Pre-Development (10-Year)

Post Development (10-Year)										
Drainage Area No.	Area (acres)	Runoff Coefficient (C)(1)	Time of Concentration (T_c)				Tc (min) (2)	I_{10} (in/hr) (3)	V_{10} (ft/s)	Q_{10} (cfs)
			US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)				
BASIN 1	12.42	0.62	291.4	283.8	450.0	1.7	15.3	2.2	12.57	17.0
BASIN 2	5.35	0.91	290.0	265.5	160.0	15.3	5.0	3.4	14.47	16.5
Total	**17.77**	-	-	-	-	-	-	-	-	**33.5**

Table 5-2: Hydrologic Summary for Post Development (10-Year)

Notes:

(1) Runoff Coefficient (C) was calculated using Table A-1 Runoff Coefficients for Rational Method of the City of San Diego Drainage Design Manual. Refer to Appendix A for additional information.
(2) Time of Concentration (Tc) was determined by using Figure A-4 Rational Formula - Overland Time of Flow Nomograph
(3) Intensity (I) of rain fall was obtained from the "Rainfall Intensity-Duration-Frequency Curves for County of San Diego" found in Appendix A of the City of San Diego Drain Design Manual

A similar decrease in the peak flow runoff rate is experienced in the 100-year storm event, which can be seen in Table 5-3 and Table 5-4. In the pre and post development conditions, the peak runoff rate decreased from 49.8 cfs to 43.8 cfs. This represents an overall 13% decrease in the peak runoff flow rate.

Pre-Development (100-Year)										
Drainage Area No.	Area (acres)	Runoff Coefficient (C)(1)	Time of Concentration (T_c)				Tc (min) (2)	I_{10} (in/hr) (3)	V_{10} (ft/s)	Q_{100} (cfs)
			US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)				
BASIN 1	11.50	0.68	291.4	283.8	475	1.6	14.2	3.0	13.61	23.3
BASIN 2	5.72	0.96	291.3	290.4	120	0.8	5.0	4.4	19.16	24.2
BASIN 3	0.55	0.96	290.5	290.0	100	0.5	5.0	4.4	5.57	2.3
Total	**17.77**	-	-	-	-	-	-	-		**49.8**

Table 5-3: Hydrologic Summary for Pre-Development (100-Year)

Post Development (100-Year)										
Drainage Area No.	Area (acres)	Runoff Coefficient (C)(1)	Time of Concentration (T_c)				Tc (min) (2)	I_{10} (in/hr) (3)	V_{10} (ft/s)	Q_{100} (cfs)
			US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)				
BASIN 1	12.42	0.62	291.4	283.8	450.0	1.7	15.3	2.9	13.48	22.4
BASIN 2	5.35	0.91	290.0	265.5	160.0	15.3	5.0	4.4	15.16	21.4
Total	**17.77**	-	-	-	-	-	-	-		**43.8**

Table 5-4: Hydrologic Summary for Post Development (100-Year)

Notes:

(1) Runoff Coefficient (C) was calculated using Table A-1 Runoff Coefficients for Rational Method of the City of San Diego Drainage Design Manual. Refer to Appendix A for additional information.
(2) Time of Concentration (Tc) was determined by using Figure A-4 Rational Formula - Overland Time of Flow Nomograph
(3) Intensity (I) of rain fall was obtained from the "Rainfall Intensity-Duration-Frequency Curves for County of San Diego" found in Appendix A of the City of San Diego Drain Design Manual

5.2 Conclusions

As evidenced by the decreased peak flow values in 10-year and 100-year storm, under the Post Development conditions the project site will not be negatively impacted in terms of hydrology or hydraulics. Proposed landscape area and various post construction BMPs identified in the project SWQMP will further alleviate the effects of additional hydrological or hydraulic demands which is typically expected from development.

6. References

City of San Diego, 2017. City of San Diego (January 2017). Drainage Design Manual.

Federal Emergency Management Agency (FEMA), 2012. FEMA (May 16, 2012). FEMA Flood Map Service Center. City of San Diego.

Appendix A Project Site Information

Vicinity Map

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations (BFEs) shown on this map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the Summary of Stillwater Elevations table in the Flood Insurance Study report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures for this jurisdiction.

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) Zone 11. The **horizontal datum** was NAD83, GRS1980 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov/ or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at **(301) 713-3242** or visit its website at http://www.ngs.noaa.gov/.

Base map information shown on this FIRM was provided in digital format by the USDA National Agriculture Imagery Program (NAIP). This information was photogrammetrically compiled at a scale of 1:24,000 from aerial photography dated 2009.

This map reflects more detailed and up-to-date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables in the Flood Insurance Study report (which contains authoritative hydraulic data) may reflect stream channel distances that differ from what is shown on this map.

Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

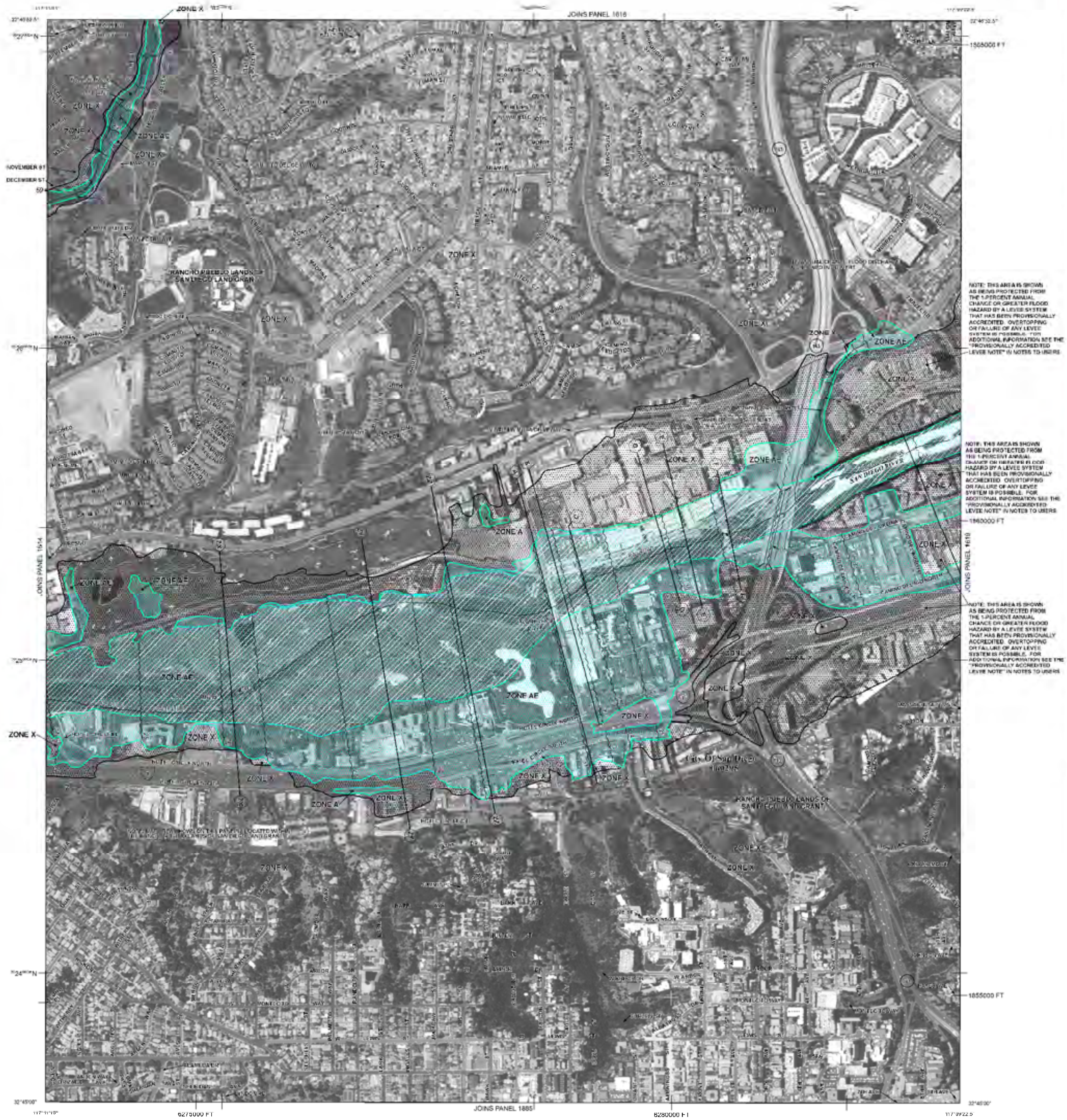
Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

Contact the **FEMA Map Service Center** at 1-877-FEMA MAP (1-877-336-2627) for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, a Flood Insurance Study report, and/or digital versions of this map. The FEMA Map Service Center may also be reached by Fax at 1-800-358-9620 and its website at http://msc.fema.gov/.

If you have **questions about this map** or questions concerning the National Flood Insurance Program in general, please call **1-877-FEMA MAP** (1-877-336-2627) or visit the FEMA website at http://www.fema.gov/business/nfip/.

The **"profile base lines"** depicted on this map represent the hydraulic modeling baselines that match the flood profiles in the FIS report. As a result of improved topographic data, the "profile base line", in some cases, may deviate significantly from the channel centerline or appear outside the SFHA.

Provisionally Accredited Levee Notes to Users: Check with your local community to obtain more information, such as the estimated level of protection provided (which may exceed the 1-percent-annual-chance level) and Emergency Action Plan, on the levee system(s) shown as providing protection for areas on this panel. To maintain accreditation, the levee owner or community is required to submit the data and documentation necessary to comply with Section 65.10 of the NFIP regulations by July 23, 2009. If the community or owner does not provide the necessary data and documentation or if the data and documentation provided indicate the levee system does not comply with Section 65.10 requirements, FEMA will revise the flood hazard and risk information for this area to reflect de-accreditation of the levee system. To mitigate flood risk in residual risk areas, property owners and residents are encouraged to consider flood insurance and floodproofing or other protective measures. For more information on flood insurance, interested parties should visit the FEMA Website at http://www.fema.gov/business/nfip/index.shtm.



LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Areas to be protected from 1% annual chance flood event by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

	1% annual chance floodplain boundary
	0.2% annual chance floodplain boundary
	Floodway boundary
	Zone D boundary
	CBRS and OPA boundary
	Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths, or flood velocities.
~~ 513 ~~~	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988

(A)——(A)	Cross section line
(23)- - - -(23)	Transect line
97°07'30", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
[illegible]	1000-meter Universal Transverse Mercator grid ticks, zone 11
6000000 FT	5000-foot grid values: California State Plane coordinate system, Zone VI (FIPSZONE = 406), Lambert projection
DX5510	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTYWIDE
FLOOD INSURANCE RATE MAP
June 19, 1997

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL
May 16, 2012 - to update corporate limits, to add roads and road names, to incorporate previously issued Letters of Map Revision, and to update map elevations to North American Vertical Datum of 1988.

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 500'

250 0 250 500 750 1000 FEET

150 0 150 300 METERS

PANEL 1618G

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

FIRM

FLOOD INSURANCE RATE MAP

SAN DIEGO COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 1618 OF 2375

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN DIEGO, CITY OF	060295	1618	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER 06073C1618G

MAP REVISED MAY 16, 2012

Federal Emergency Management Agency

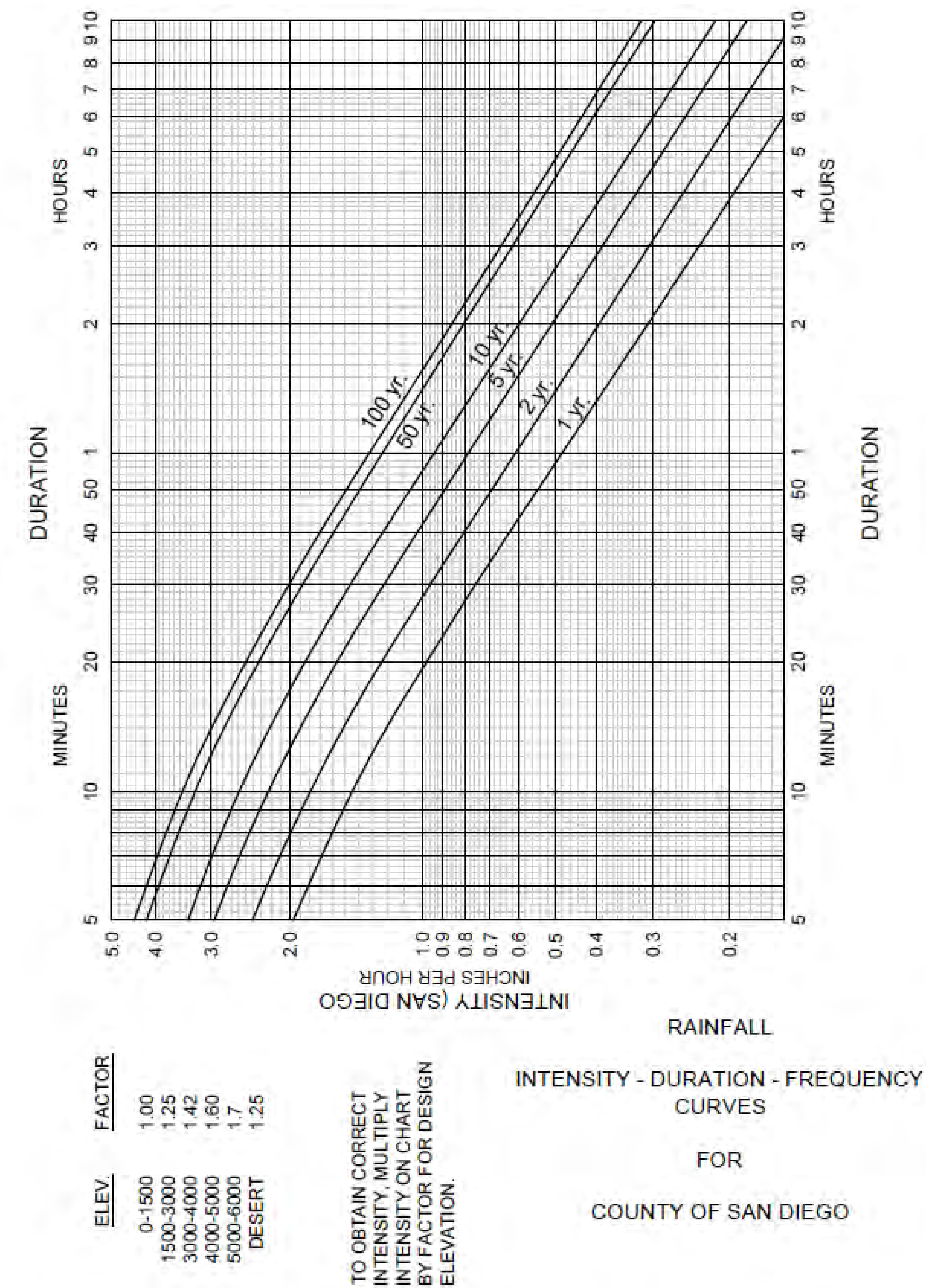
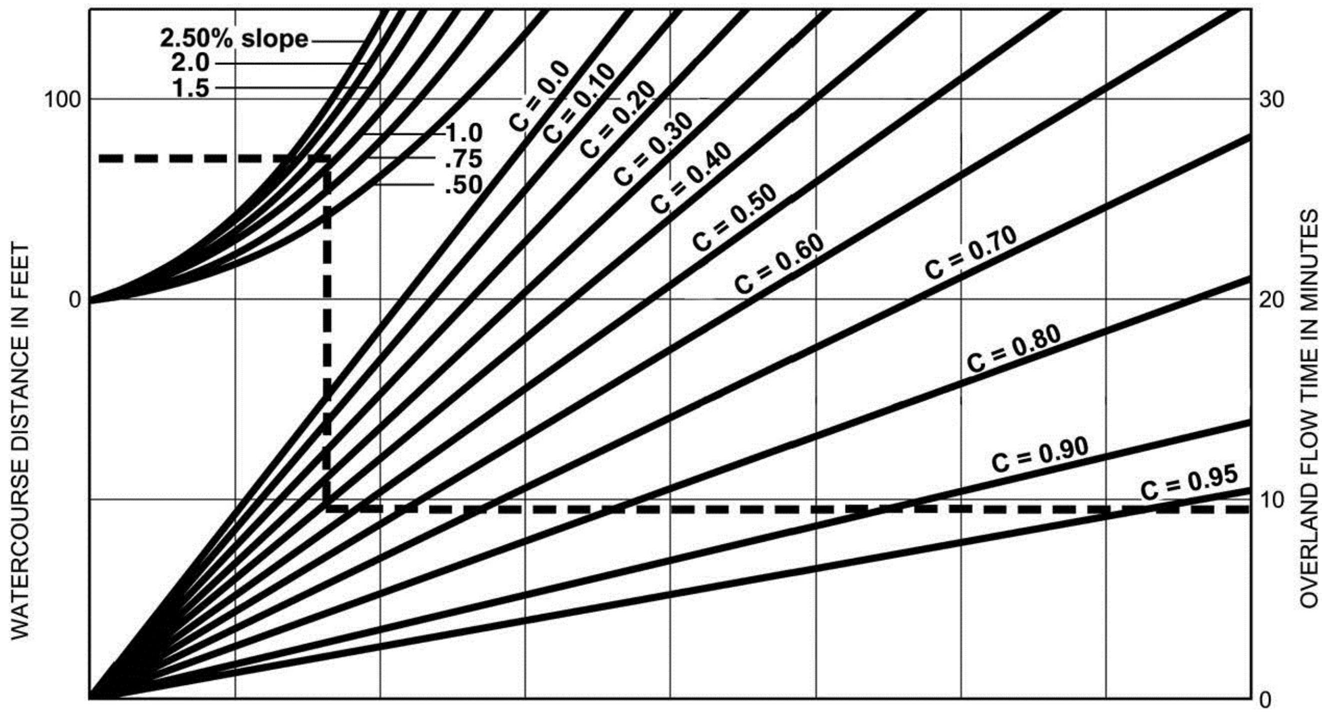


Figure A-1. Intensity-Duration-Frequency Design Chart



EXAMPLE:

Given: Watercourse Distance (D) = 70 Feet

Slope (s) = 1.3%

Runoff Coefficient (C) = 0.41

Overland Flow Time (T) = 9.5 Minutes

$$T = \frac{1.8\,(1.1-C)\sqrt{D}}{\sqrt[3]{s}}$$

SOURCE: Airport Drainage, Federal Aviation Administration, 1965

Figure A-4. Rational Formula – Overland Time of Flow Nomograph

Note: Use formula for watercourse distances in excess of 100 feet.

Table A-1. Runoff Coefficients for Rational Method

Land Use	Runoff Coefficient (C)
	Soil Type [(1)]
Residential:	
Single Family	0.55
Multi-Units	0.70
Mobile Homes	0.65
Rural (lots greater than ½ acre)	0.45
Commercial [(2)]	
80% Impervious	0.85
Industrial [(2)]	
90% Impervious	0.95

Note:
(1) Type D soil to be used for all areas.
(2) Where actual conditions deviate significantly from the tabulated imperviousness values of 80% or 90%, the values given for coefficient C, may be revised by multiplying 80% or 90% by the ratio of actual imperviousness to the tabulated imperviousness. However, in case shall the final coefficient be less than 0.50. For example: Consider commercial property on D soil.

Actual imperviousness			=	50%
Tabulated imperviousness			=	80%
Revised C	=	(50/80) x 0.85	=	0.53

The values in Table A–1 are typical for urban areas. However, if the basin contains rural or agricultural land use, parks, golf courses, or other types of nonurban land use that are expected to be permanent, the appropriate value should be selected based upon the soil and cover and approved by the City.

A.1.3. Rainfall Intensity

The rainfall intensity (I) is the rainfall in inches per hour (in/hr.) for a duration equal to the T_C for a selected storm frequency. Once a particular storm frequency has been selected for design and a T_C calculated for the drainage area, the rainfall intensity can be determined from the Intensity-Duration-Frequency Design Chart (Figure A-1).

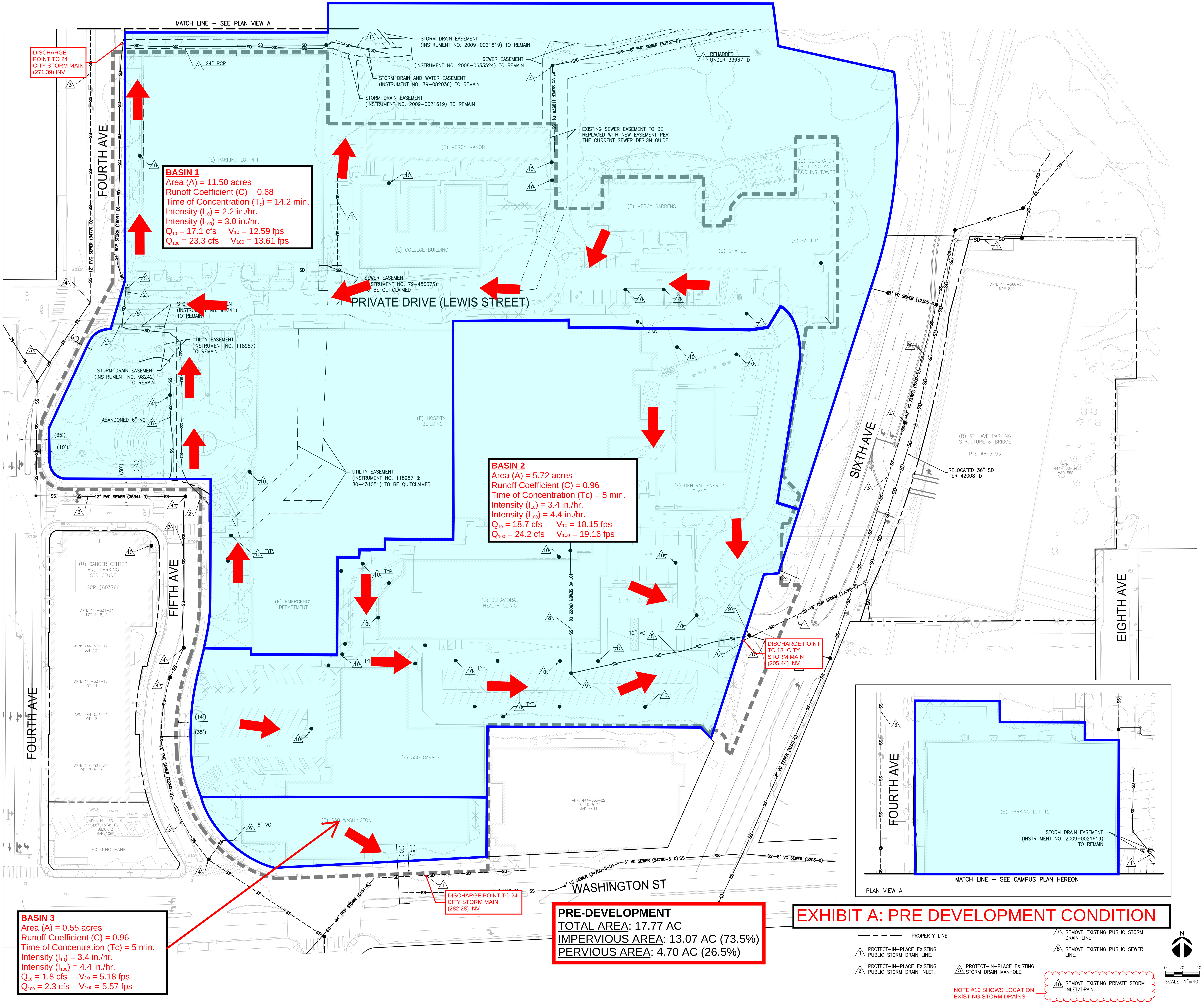
Appendix A: Revised C Value Calculation

EXAMPLE From Table A-1:

Actual imperviousness		=	50%
Tabulated imperviousness		=	80%
Revised C	= (50/80) x 0.85	=	0.53

Pre-Development Condition	Post-Development Condition
Basin 1: Area: 11.50 ac Actual Imperviousness: 64% Tabulated Imperviousness: 90% Revised C: (64/90) x 0.95 = **0.68**	**Basin 1:** Area: 12.42 ac Actual Imperviousness: 59% Tabulated Imperviousness: 90% Revised C: (59/90) x 0.95 = **0.62**
Basin 2: Area: 5.72 ac Actual Imperviousness: 91% Tabulated Imperviousness: 90% Revised C: (91/90) x 0.95 = **0.96**	**Basin 2:** Area: 5.35 ac Actual Imperviousness: 86% Tabulated Imperviousness: 90% Revised C: (86/90) x 0.95 = **0.91**
Basin 3: Area: 0.55 ac Actual Imperviousness: 91% Tabulated Imperviousness: 90% Revised C: (91/90) x 0.95 = **0.96**	

Appendix B Pre Development Hydrologic Work Map & Calculations



CO ARCHITECTS

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Los Angeles, California 90036
323.525.0500 phone, 323.525.0955 fax

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700 South Flower Street
Suite 2100
Los Angeles, CA 90017
O: 213.418.0201
F: 213.266.5294
www.kpff.com

REVISIONS

1	4/10/20	REV 1
2	8/14/20	REV 2
3	11/18/20	REV 3
4	3/26/21	REV 4
5	6/11/21	REV 5
6	3/4/22	REV 6

Scripps

CONDITIONAL USE PERMIT MERCY CAMPUS
4077 Fifth Ave, San Diego, CA 92103
SCRIPPS# 35-16060B DESIGN TEAM # 18003.000

KEY PLAN

EXISTING GRAVITY UTILITIES PLAN

SCALE:
DATE OF ISSUE: 1/27/22

CUP-07

7

Pre Development (10-Year)												
Drainage Area No.	Area (acres)	IMP %	Runoff Coefficient (C) (1)	Time of Concentration, (Tc)				T_c (min) (2)	I10 (in/hr) (3)	V10 (fps)	Q_{10} (cfs)
				US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)				
BASIN 1	11.50	64	0.68	291.4	283.8	475.0	1.6	14.2	2.2	12.6	17.1
BASIN 2	5.72	91	0.96	291.3	290.4	120.0	0.8	5.0	3.4	18.2	18.7
BASIN 3	0.55	91	0.96	290.5	290.0	100.0	0.5	5.0	3.4	5.2	1.8
Total	**17.77**		-	-	-	-	-			-	**37.6**

Pre Development (100-Year)											
Drainage Area No.	Area (acres)	IMP %	Runoff Coefficient (C) (1)	Time of Concentration, (Tc)				T_c (min) (2)	I100 (in/hr) (3)	V100 (fps)	Q_{100} (cfs)
				US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)				
BASIN 1	11.50	64	0.68	291.4	283.8	475.0	1.6	14.2	3.0	13.6	23.3
BASIN 2	5.72	91	0.96	291.3	290.4	120.0	0.8	5.0	4.4	19.2	24.2
BASIN 3	0.55	91	0.96	290.5	290.0	100.0	0.5	5.0	4.4	5.6	2.3
Total	**17.77**		-	-	-	-	-			-	**49.8**

Notes:

(1) Runoff Coeffcient (C) was calculated using Table A-1 Runoff Coefficients for Rational Method

(2) Time of Concentration (Tc) was determined by using Figure A-4 Rational Formula - Overland Time of Flow Nomograph

(3) Intensity (I) of rain fall was obtained from the "Rainfall Intensity-Duration-Frequency Curves for County of San Diego" found in Appendix A of the City of San Diego Drain Design Manual

Appendix C Post Development Hydrologic Work Map & Calculations

NOTES:

1. ALL PUBLIC IMPROVEMENTS ARE TO BE CONSTRUCTED PER CURRENT CITY STANDARDS.

2. THE PROPOSED PROJECT WILL COMPLY WITH ALL THE REQUIREMENTS OF THE CURRENT CITY OF SAN DIEGO STORM WATER STANDARDS MANUAL BEFORE A GRADING OR BUILDING PERMIT IS ISSUED. IT IS THE RESPONSIBILITY OF THE OWNER/DESIGNER/APPLICANT TO ENSURE THAT THE CURRENT STORM WATER PERMANENT BMP DESIGN STANDARDS ARE INCORPORATED INTO THE PROJECT.

3. NO OBSTRUCTION INCLUDING SOLID WALLS IN THE VISIBILITY AREA SHALL EXCEED 3 FEET IN HEIGHT. PER SDMC SECTION 142.0409 (b)(2), PLANT MATERIAL, OTHER THAN TREES, LOCATED WITHIN VISIBILITY AREAS OR THE ADJACENT PUBLIC RIGHT-OF-WAY SHALL NOT EXCEED 36 INCHES IN HEIGHT, MEASURED FROM THE LOWEST GRADE ABUTTING THE PLANT MATERIAL TO THE TOP OF THE PLANT MATERIAL.

GRADING QUANTITIES

GRADED AREA	11.08 [ACRES]	MAX. CUT DEPTH:	49 [FT]
CUT QUANTITIES	155,000 [CYD]	MAX CUT SLOPE RATIO (2:1 MAX):	2:1
FILL QUANTITIES	42,500 [CYD]	MAX. FILL DEPTH:	33 [FT]
IMPORT/EXPORT	112,500 [CYD]	MAX FILL SLOPE RATIO (2:1MAX):	2:1

UTILITY CONSTRUCTION NOTES:

STORM DRAIN

- SD1 PROPOSED STORM DRAIN LINE
- SD2 PROPOSED BUILDING ROOF DRAIN/DECK DRAIN CONNECTION
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- SD5 PROPOSED CURB INLET STRUCTURE
- SD6 PROPOSED BIOFILTRATION PLANTER OVERFLOW
- SD7 CONNECTION TO EXISTING STORM DRAIN LINE
- SD8 PROPOSED STORMWATER CLEANOUT
- SD9 PROPOSED BIOFILTRATION PLANTER PER SAN DIEGO STORMWATER STANDARDS MANUAL.
- SD10 PROPOSED MODULAR WETLAND SYSTEM
- SD11 PROPOSED CISTERN SIZED PER CITY OF SAN DIEGO HYDROMODIFICATION STANDARDS. DIMENSIONS PER PLAN.

LEGEND

- LIMIT OF WORK
- PROPERTY LINE
- FLOW LINE
- GRADE BREAK
- RIDGE LINE
- SAWCUT AND JOIN
- GRADE SLOPE (HORIZONTAL:VERTICAL)
- LIMITS OF GRADING
- GRADING BENCH
- PROPOSED MAJOR CONTOUR
- PROPOSED MINOR CONTOUR
- PROPOSED RETAINING WALL
- EXISTING STORM DRAIN
- PROPOSED STORM DRAIN
- PROPOSED CATCH BASIN
- PROPOSED STORM DRAIN MANHOLE
- DIRECTION OF FLOW
- BUILDING POINT OF CONNECTION
- EXISTING CATCH BASIN & STORM INLET/DRAIN
- NEW PEDESTRIAN CONCRETE PAVING
- NEW VEHICULAR ASPHALT PAVING
- BIOFILTRATION PLANTER AREA
- PROPOSED BUILDING
- EXISTING BUILDING

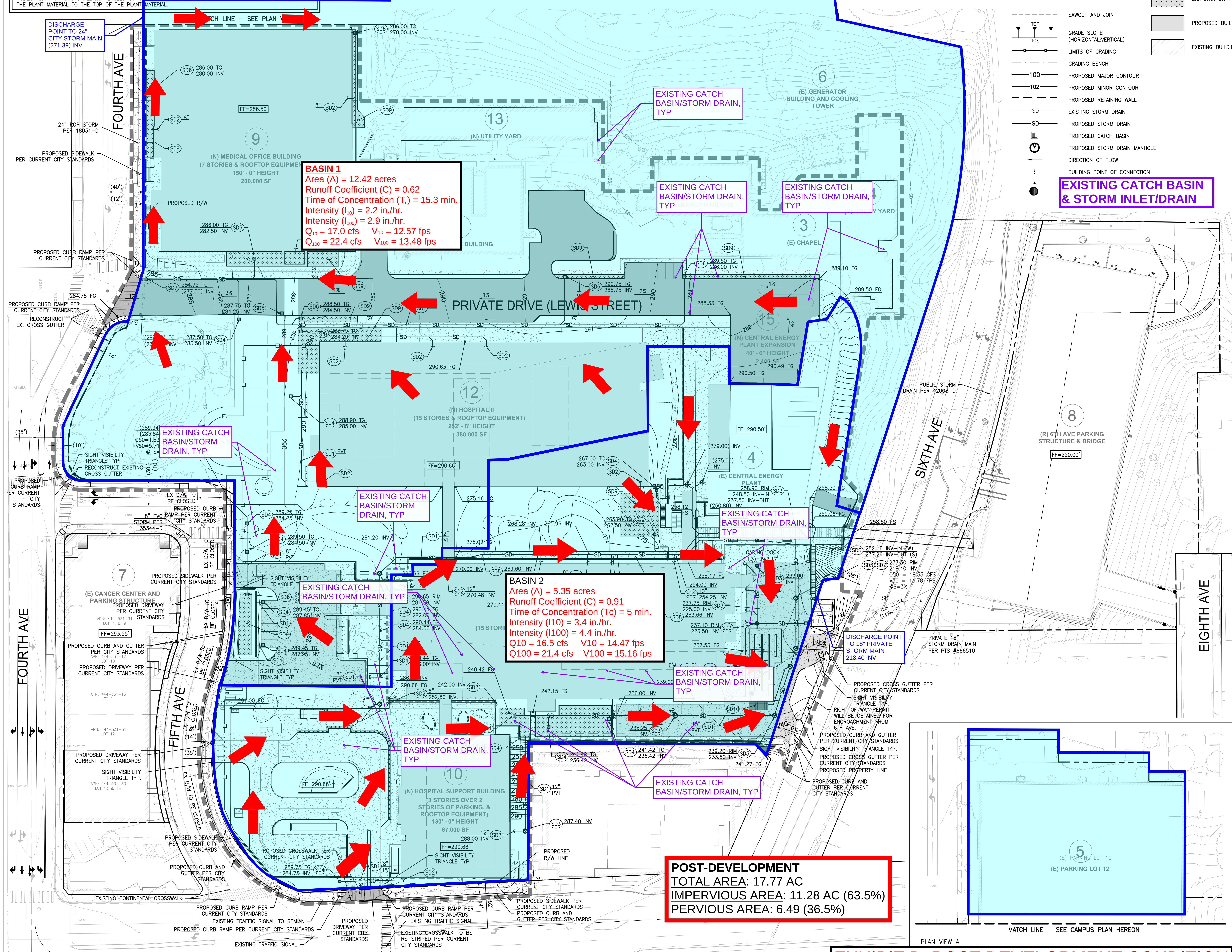

EXHIBIT B: POST DEVELOPMENT CONDITION

CO ARCHITECTS

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Los Angeles, California 90036
323.525.0500 phone, 323.525.0955 fax

PROFESSIONAL STAMP

REGISTERED PROFESSIONAL ENGINEER JEFFREY B. GANZA No. 59894 Exp. 3/31/23 CIVIL STATE OF CALIFORNIA

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REVISIONS

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2	8/14/20	REV 2
3	11/18/20	REV 3
4	3/26/21	REV 4
5	6/11/21	REV 5
6	3/4/22	REV 6
7	5/27/22	REV 7

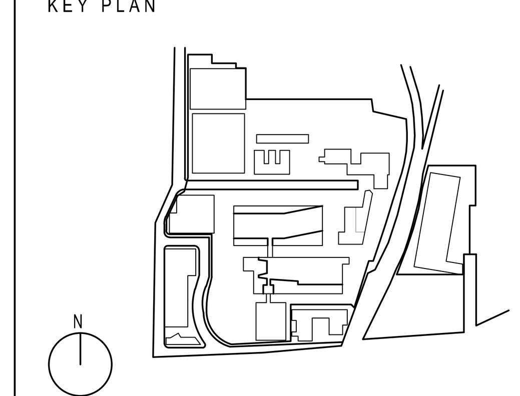
Scripps

CONDITIONAL USE PERMIT MERCY CAMPUS

4077 Fifth Ave, San Diego, CA 92103

SCRIPPS# 35-16060B DESIGN TEAM # 18003.000

KEY PLAN



PRELIMINARY GRADING AND DRAINAGE PLAN

SCALE:
DATE OF ISSUE: 5/27/22

CUP-10

Post Development (10-Year)												
Drainage Area No.	Area (acres)	IMP %	Runoff Coefficient (C) (1)	Time of Concentration, (Tc)				T_c (min) (2)	I10 (in/hr) (3)	V10 (fps)	Q_{10} (cfs)
				US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)				
BASIN 1	12.42	59	0.62	291.4	283.8	450.0	1.7	15.3	2.2	12.6	17.0
BASIN 2	5.35	86	0.91	290.0	265.5	160.0	15.3	5.0	3.4	17.4	16.5
Total	**17.77**		-	-	-	-	-	-	-	-	**33.5**

Post Development (100-Year)											
Drainage Area No.	Area (acres)	IMP %	Runoff Coefficient (C) (1)	Time of Concentration, (Tc)				T_c (min) (2)	I100 (in/hr) (3)	V100 (fps)	Q_{100} (cfs)
				US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)				
BASIN 1	12.42	59	0.62	291.4	283.8	450.0	1.7	15.3	2.9	13.5	22.4
BASIN 2	5.35	86	0.91	290.0	265.5	160.0	15.3	5.0	4.4	18.5	21.4
Total	**17.77**		-	-	-	-	-	-	-	-	**43.8**

Notes:

(1) Runoff Coeffcient (C) was calculated using Table A-1 Runoff Coefficients for Rational Method

(2) Time of Concentration (Tc) was determined by using Figure A-4 Rational Formula - Overland Time of Flow Nomograph

(3) Intensity (I) of rain fall was obtained from the "Rainfall Intensity-Duration-Frequency Curves for County of San Diego" found in Appendix A of the City of San Diego Drain Design Manual

Appendix D Hydraulic Exhibit & Calculations

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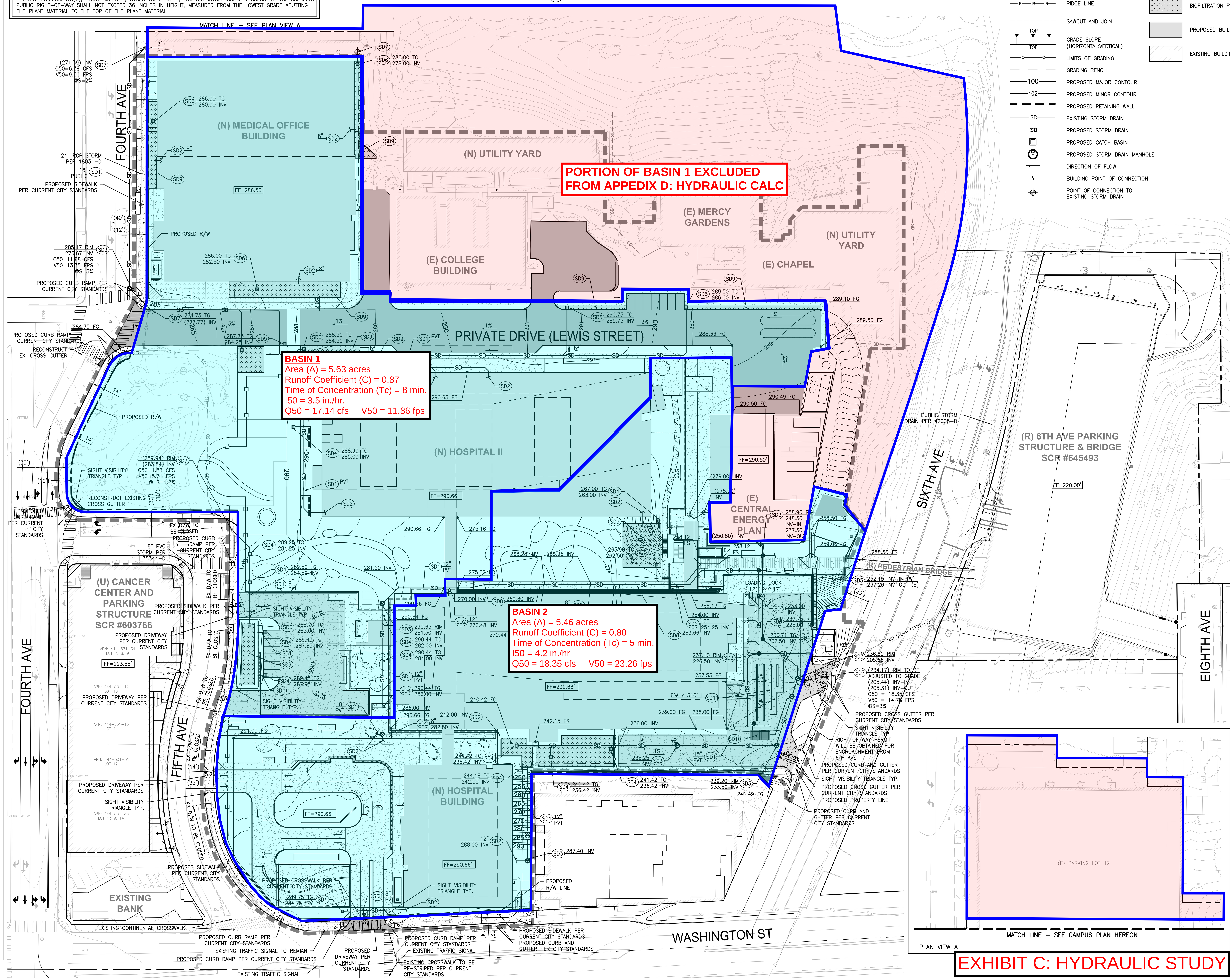

EXHIBIT C: HYDRAULIC STUDY

CO ARCHITECTS

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6	3/4/22	REV 6

Scripps

CONDITIONAL USE PERMIT MERCY CAMPUS

4077 Fifth Ave, San Diego, CA 92103

SCRIPPS# 35-16060B DESIGN TEAM # 18003.000

KEY PLAN

PRELIMINARY GRADING AND DRAINAGE PLAN

SCALE:
DATE OF ISSUE: 1/27/22

SCALE: 1"=40'

CUP-10

Post Development (50-Year)									
Drainage Area No.	Area (acres)	Runoff Coefficient (C) (1)	Time of Concentration, (Tc)				T_c (min) (2)	I50 (in/hr) (3)	Q_{50} (cfs)
			US Elevation (ft)	DS Elevation (ft)	Length	Slope (%)			
BASIN 1	5.63	0.87	291.4	283.8	450.0	1.7	7.4	3.5	17.14
BASIN 2	5.46	0.80	290.0	265.5	250.0	9.8	5.0	4.2	18.35
Total	**11.09**	-	-	-	-	-	-	-	**35.49**

Notes:

(1) Runoff Coeffcient (C) was calculated using Table A-1 Runoff Coefficients for Rational Method

(2) Time of Concentration (Tc) was determined by using Figure A-4 Rational Formula - Overland Time of Flow Nomograph

(3) Intensity (I) of rain fall was obtained from the "Rainfall Intensity-Duration-Frequency Curves for County of San Diego" found in Appendix A of the City of San Diego Drain Design Manual

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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.

April 26, 2019
Kleinfelder Project No. 20183768.001A

Mr. Bruce A. Rainey
Corporate Vice President
Scripps Health
10140 Campus Point Drive, Suite 210
San Diego, California 92121

SUBJECT: Geologic and Geotechnical Engineering Evaluation
Replacement Hospital
Scripps Mercy Hospital San Diego
4077 5th Avenue
San Diego, California

Dear Mr. Rainey:

This report presents the results and recommendations of our geologic and geotechnical engineering evaluation for the design and construction of the new replacement hospital building. The proposed project is to be constructed on the southeastern portion of the existing Scripps Mercy Hospital campus located on 4077 5th Avenue in San Diego, California.

Based on the results of our geotechnical investigation and engineering evaluation, it is our opinion that the project is feasible from a geotechnical standpoint. Our geotechnical investigation and evaluation is based on the provided December 2017 programming documents, subsequent correspondence and our proposed scope of work.

We appreciate this opportunity to be of service and look forward to working with you in the future. If you have any questions about this report, please contact us.

Respectfully submitted,

KLEINFELDER

Salvador Tena
Staff Engineer, PE 89071

Scott H. Rugg CEG 1651
Engineering Geologist

Kevin M. Crennan GE 2511
Senior Engineer

GEOLOGIC AND GEOTECHNICAL ENGINEERING REPORT
REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL SAN DIEGO
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA
KLEINFELDER PROJECT NO. 20183768.001A

APRIL 26, 2019

A Report Prepared for:

Mr. Bruce A. Rainey
Corporate Vice President
Scripps Health
10140 Campus Point Drive, Suite 210
San Diego, California 92121

GEOLOGIC AND GEOTECHNICAL ENGINEERING REPORT
REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL SAN DIEGO
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

Prepared by:

Salvador Tena
Staff Engineer, PE

Scott H. Rugg CEG 1651
Engineering Geologist

Reviewed by:

Kevin M. Crennan GE 2511
Senior Engineer

KLEINFELDER
550 West C Street, Suite 1200
San Diego, California 92101
Phone: 619.831.4600
Fax: 619.232.1039

April 26, 2019
Kleinfelder Project No: 20183768.001A

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TABLES

FIGURES

APPENDICES

EXECUTIVE SUMMARY

The Scripps Mercy Hospital San Diego is located at 4077 5th Avenue in San Diego, California (Figure 1). The proposed replacement hospital building is located in the lower southeastern portion of the hospital campus. The layout of the proposed replacement hospital site is shown in Figure 2.

The proposed structure will be 16 stories above the existing lower parking lot elevation with the lower 3 levels partially retaining the western and northern slopes. The building will have a one-story projection on the 4th story (Level 1) which will connect to the emergency department on the west.

Kleinfelder's field exploration for the replacement hospital building consisted of advancing seven hollow stem auger borings, infiltration testing, geophysical survey and geologic reconnaissance of the site. Boring data from previous nearby studies on the Scripps Mercy campus were also evaluated in this study. Select borings from previous site investigations and the Logs of Borings for this study are included in Appendix A and shown in Figure 2.

The results of our field exploration and review indicate that fill ranging up to approximately 10 feet in depth is present within the proposed lower pad building area and up to 25 feet at the top on the western end of the building. The San Diego Formation and very old paralic deposits are present underlying the fill, and the Pomerado Conglomerate is below the San Diego Formation. These formational units are weakly to strongly cemented and competent. Refusal of advancing the drilling augers was encountered in all of the borings at or near the top of the Pomerado Conglomerate. Analysis of previous boring data indicates that the San Diego Formation is present below fill on the slopes which ascend to the upper campus elevation. No static groundwater was observed in the borings. A more detailed description of each unit is provided in the attached report.

Based on our review of the data collected during the study, it is our opinion that the proposed development is feasible from a geotechnical perspective if the recommendations contained herein are followed. The following key items are conclusions developed from our study:

- Undocumented fill to depths of up to approximately 10 feet is present in some areas of the lower pad area, below the northern slope and up to 25 feet below the western slope surface.
- Based on the results of our geotechnical evaluation, it is our opinion that the undocumented fill is not suitable for support of structural loads in their current condition. Therefore, the removal and recompaction of the undocumented fill within the building area is recommended.
- The lower level of the building can be supported on a mat foundation system bearing entirely in compacted fill. Based on the proposed finish floor elevation of 242 feet mean sea level. Fill will be required to raise portions of the site grade. In addition, removal and recompaction of existing undocumented fill and some formational soils will be required to provide a relatively uniform mat of compacted fill.
- In order to mitigate the potential for differential settlement and address the variable depth of fill up to 25 feet on the western slope, driller pier foundations into formational materials or other measures will be required for Level 1 at elevation 291 feet on the western end of the building.
- Temporary shoring will be required around portions of the foundation excavation to protect existing improvements during construction of subterranean levels.
- The replacement hospital building site is located in a seismically active area and could be subject to relatively strong ground shaking due to earthquakes on active faults in the region. The structures should be designed to tolerate seismic shaking in accordance with the 2016 California Building Code (CBC).
- The site is not located within a State of California Alquist-Priolo fault zone.

This summary of findings should not be relied upon without consulting the attached report for a more detailed description of the geotechnical study performed by Kleinfelder for Scripps Mercy Hospital San Diego. This report is subject to the limitations included in Section 7.0 and the Geotechnical Business Council insert in Appendix G.

1 INTRODUCTION

1.1 PURPOSE

In accordance with your authorization, we have completed a geologic and geotechnical investigation for the replacement hospital building, Scripps Mercy Hospital project located at 4077 5th Avenue, San Diego, California (see Vicinity Map, Figure 1). The purpose of our study was to evaluate the surface and subsurface conditions at the site, evaluate potential geologic hazards, and provide geotechnical recommendations for design and construction. This report presents the results of our background review, subsurface exploration, laboratory testing, geotechnical analyses, conclusions regarding the geotechnical conditions at the project site, and our recommendations.

1.2 PROJECT DESCRIPTION

Our understanding of the project is based on conversations with Scripps Health and Jacobs personnel and review of Sheets 1.01 through 1.16 of progress plans titled “Mercy Hospital Replacement” dated November 19, 2018, prepared by CO Architecture.

The proposed replacement hospital building will be up to 16-stories above the lower parking lot elevation, with subterranean walls on the lower 3-stories on portion of the western and northern sides. The below-grade footprint has approximate plan dimensions of 150 feet by 375 feet. The depth of subterranean levels will be up to 50 feet on the western end and decrease across the northern side due to variable topography. In addition, a loading dock is proposed on the northeastern side of the new hospital building. A portion of the existing site is occupied by the Scripps Behavior Health Unit (Clinic Building #14) which is currently scheduled to be demolished in 2021. The remaining portion of the site is primarily used as a parking lot area and access road and slopes which ascend to upper northern campus.

The lower level of the building will have a finish floor elevation of 242 feet mean sea level be supported on a mat foundation system bearing on the order of 4 feet in thickness, with thickened zones under the corner columns of the moment frames and braced frames. The one-story western portion of the building at Level 1 will have a finish floor elevation of about 291 feet and be supported on drilled pier foundations due to the thickness of undocumented fill in this area. Building foundation loads were not available at the time we prepared this report.

The address and latitude/longitude coordinates for the replacement hospital building are listed below, and the vicinity map is shown on Figure 1. The site and exploration plan is shown on Figure 2.

Address:	Scripps Mercy Hospital San Diego 4077 5th Avenue San Diego, California 92103
Latitude:	32.7509^{O} N
Longitude:	117.1598^{O} W

1.3 GENERAL SITE DESCRIPTION

The Mercy Hospital campus is generally located north of Washington Street, south of Lewis Street, west of 6th Avenue and east of 5th Avenue, as shown on the Vicinity Map, Figure 1. The majority of the developed campus is relatively level with surface elevations on the order of 290 to 295 feet mean sea level (msl). The majority of the proposed replacement hospital site is located on a lower pad area on the southeastern portion of campus with an average elevation of approximately 233 feet MSL. This area contains the existing Scripps Behavior Health Unit (Clinic Building #14) footprint and existing parking lot south of the building. The western-most side of the replacement building site is occupied by a west-ascending slope that is completely covered with shotcrete and rises up to an elevation of approximately 290 feet msl at the pad currently occupied by the Emergency Department (Building 24). The western portion of the southern side of the building will be constructed adjacent to an existing 4-story subterranean parking structure. Portions of the northern-most side of the site is occupied by a north-ascending slope that rises up to an elevation of approximately 290 feet MSL to a pad currently occupied by the main hospital tower and ambulatory addition (Buildings 4 and 9B). An access road to the upper northern campus is located in the proposed loading dock area at the northeastern corner of the building. The site configuration is shown on Figure 2.

1.4 SCOPE OF SERVICES

The scope of services for this study included the following;

- Review of readily available pertinent reports (including previous borings performed at the site), maps and aerial photography;
- Drilling and sampling seven soil borings;
- Geophysical survey of the project site;
- Geologic mapping of the project site;
- Performing geotechnical laboratory testing of the soil samples;
- Providing a site plan with boring locations;
- Discussion of site surface and subsurface conditions encountered, groundwater levels and anticipated excavation characteristics of the materials;
- Discussion of regional geologic setting, geologic features, and geologic hazard potential including the potential for ground rupture due to surface faulting, liquefaction and seismically induced settlement;
- Developing 2016 California Building Code seismic parameters for building design and performing a site specific ground motion hazard analysis;
- Recommendations for temporary shoring;
- Lateral earth pressures for permanent retaining walls;
- Recommendations for foundation design, including foundation type, soil bearing pressures, and anticipated total and differential settlements;
- Performing a preliminary screening of soil corrosivity characteristics;
- Guidelines for earthwork construction including recommendations for site preparation, shoring, fill placement and compaction.
- Recommendations for slab-on-grade floor design and construction;
- Preliminary stormwater infiltration study;
- Preliminary pavement design; and
- Preparation of this report.

2 METHODS OF STUDY

2.1 FIELD EXPLORATION

Our current field exploration consisted of a geologic reconnaissance of the site, performance of a geophysical refraction survey, drilling five borings to evaluate subsurface conditions, drilling two borings for infiltration testing, and collection of soil samples for laboratory testing. The exploration borings were drilled on April 4, 2018 to depths ranging from approximately 20 to 21 feet and the two borings for infiltration testing were drilled to a depth of 5 feet. All five of the deeper borings encountered practical refusal to drilling due to the presence of cemented cobble conglomerate. The drilling contractor ABC LIOVIN of Signal Hill, California used a CME-85 truck mounted drill rig. The rig was equipped with 8-inch diameter hollow stem augers, and a 140-pound hammer dropped from a height of 30 inches to drive the sampler into the soil. In addition, a geophysical refraction microtremor (ReMi) survey was performed by Southwest Geophysics, Inc. on April 2, 2018.

It should be noted that due to the constraints of current site development, most of the proposed building footprint could not be accessed for drilling and we therefore utilized offsite data from previous nearby studies to aid in the interpretation of soil conditions.

A Kleinfelder engineer from our office logged the subsurface soil conditions at each boring location and collected soil samples for laboratory testing. Driven samples were obtained from the borings using Standard Penetration Test and California split spoon samplers lined with three 6-inch brass sleeves. Soil samples obtained from the borings were packaged, sealed in the field to reduce moisture loss and disturbance, and returned to our laboratory for testing. Upon completion, all borings were backfilled with bentonite grout and were patched with concrete. A more detailed description of the Kleinfelder boring exploration program, the logs of the exploratory borings are presented in Appendix A.

Two ReMi profiles, designated as RL-1 and RL-2, were performed to evaluate the shear wave velocity. The survey was performed to develop a compression wave and shear wave velocity profile for the project and estimate average shear wave velocity in the upper 100 feet for seismic design. Fifteen 30-second-long records were recorded using a 24-channel Geometrics Geode seismograph and 4.5 Hz geophones. The location of the geophysical survey is shown on Figures 2 and descriptions of the survey methods and results are presented in Appendix C.

2.2 GEOLOGIC RECONNAISSANCE AND MAPPING

The site was geologically evaluated by our engineering geologist. Part of this work included reviewing historic maps and aerial photography to evaluate the extent of previous earthwork construction at the site. In particular, the slope exposures along 6th Avenue to the east were valuable in observing the Pomerado Conglomerate since the borings encountered refusal at the top of this unit. Data from our geologic review is incorporated into this report.

2.3 LABORATORY TESTING

Geotechnical laboratory testing was performed on representative bulk and driven samples to substantiate field classifications and to provide engineering parameters for geotechnical design. Testing performed consisted of moisture/density measurement, sieve analyses, direct shear, Atterberg limits, expansion index, maximum dry density (Proctor test), R-Value, pH, resistivity, soluble sulfates, and chlorides. Laboratory testing procedures and test results are provided in Appendix B.

2.4 BACKGROUND REVIEW

We have reviewed readily available unpublished geotechnical reports relevant to the subject site. The most pertinent reports to the current project include:

"Geologic and Geotechnical Engineering Evaluation, Scripps Mercy Hospital Expansion, New Emergency Department, 4077 5th Avenue, San Diego California," prepared by Kleinfelder, dated January 20, 2006.

"Supplemental Geotechnical Investigation, Proposed Emergency Department Expansion, Scripps Mercy Hospital, 4077 5th Avenue, San Diego California," prepared by Kleinfelder, dated March 6, 2007.

"Draft Report Geologic and Geotechnical Engineering Evaluation, Scripps Mercy Hospital Expansion, BHU Elevator Tower Replacement, 4077 5th Avenue, San Diego, California," prepared by Kleinfelder, dated March 27, 2007.

"Geotechnical Investigation Emergency Access Road Repair, Scripps Mercy Hospital, 4077 5th Avenue, San Diego, California," prepared by Kleinfelder for Hazard Construction, dated September 21, 2017.

We have also reviewed historic aerial photography and maps of the site as part of our work.

3 GEOLOGY AND SOILS

3.1 REGIONAL GEOLOGIC AND GEOTECTONIC SETTING

The site is located on a coastal terrace within the Peninsular Range geomorphic province (Norris and Webb, 1990). This province stretches for several hundreds of miles south from the Los Angeles area to the tip of Baja California. It is dominated by basement material of Cretaceous-age igneous rocks of the Southern California Batholith and various Jurassic-age metamorphic rocks that are often situated as isolated blocks within the igneous rocks. This igneous/metamorphic complex occupies the regions of central and eastern San Diego County.

The western coastal zone of San Diego County is underlain by a westward thickening wedge of sedimentary units that were deposited on the basement rocks described above. These sedimentary units can be divided into three series of deposits based on their sequence and age of deposition. The oldest sequence consists of claystone, siltstone, sandstone, and conglomerate deposited during late Cretaceous time as an apparent submarine fan (Abbott, 1999). These units crop out on Mt. Soledad in La Jolla, Point Loma, and Carlsbad. The second sequence of sediments was deposited during the Tertiary (Eocene and Pliocene) period within an embayment that stretched from northern San Diego County into Mexico (Kennedy, 1975). The sediments consist of a variety of claystone, siltstone, sandstone, and conglomerate.

The most recent sedimentary deposits consist of early to late Pleistocene near shore marine, estuarine, and delta deposits, also typically identified as terrace deposits. Most of these sediments were deposited on wave cut surfaces (terraces) developed in response to sea level fluctuations during the Pleistocene. The oldest terrace deposits are typically identified as the Lindavista Formation and consist of conglomerate and sandstone with minor clay and silt strata. The youngest terrace deposits (late Pleistocene) are known as the Bay Point Formation. More recent geologic maps (Kennedy and Tan, 2007) have subdivided both the Lindavista and Bay Point Formations into numerous very old paralic deposits (Qvop1 – Qvop13) and old paralic deposits (Qop1 – Qop8) and dropped the previous formal names. The Regional Geologic Map is presented as Figure 4 and shows the local extent of the geologic units described herein.

3.2 REGIONAL FAULTING AND SEISMICITY

The San Andreas fault delineates the boundary between two global tectonic plates consisting of the North American Plate on the east and the Pacific Plate on the west and dominates the seismicity of the Southern California region (Wallace, 1990; Weldon and Sieh, 1985). It stretches from the Gulf of California in Mexico along a northwest alignment through the desert region of Southern California up to Northern California, where it eventually trends offshore north of San Francisco. Within Southern California, the mostly right-lateral strain associated with the plate boundary movement extends well westward for up to 150 miles (241 kilometers) from the main San Andreas fault in the Imperial Valley to well offshore of San Diego.

The major faults east of San Diego (from east to west) include the San Andreas fault, the San Jacinto fault, and the Elsinore fault (see Regional Fault Map, Figure 3). Major faults west of San Diego include the Palos Verdes-Coronado Bank fault, the San Diego Trough fault, and the San Clemente fault. The most dominant zone of faulting within the San Diego region are several faults associated with the Rose Canyon Fault Zone (RCFZ).

Most of the seismic energy and associated fault displacement occurs along the fault structures closest to the plate boundary on the Elsinore, San Jacinto, and San Andreas Faults. Approximately 49 mm/yr (1.9 inches/yr) of overall lateral displacement has been measured geodetically as fault slip across the plate boundary. The Elsinore, San Jacinto, and San Andreas Faults combined account for up to 41 mm/yr (1.6 inches/yr) of the total plate displacement (84%), meaning that the remaining 8 mm/yr (0.3 inches/yr) is accommodated across the faults to the west and east (Bennett, et al, 1996). Recent GPS measurements from the offshore islands to the peninsular ranges indicate about 5 to 7 mm/yr of plate movement is accommodated by the coastal and offshore system of faults, including the Rose Canyon.

Historically, San Diego County has long been considered as a region of negligible seismic hazard. Except for a probable local event in 1862 (Legg and Agnew, 1979), there has been a lack of significant seismic activity within the recorded human history of San Diego County. More recent studies have recognized that the potential for significant seismic events is much greater than earlier believed. This potential has been recognized by the discovery of many active fault traces associated with structures within the RCFZ. Studies within Rose Canyon (east of Mt.

Soledad) have revealed fault strands that have clearly displaced Holocene soil horizons with slip rates from 1 to 2 mm/yr (Lindvall and Rockwell, 1995).

These results indicated that at least the northern onshore portion of the RCFZ is active. Additional studies (Testing Engineers and other, 1985; Patterson and others, 1986; and Kleinfelder, 1998) within downtown San Diego revealed additional fault structures offsetting Holocene soil horizons, suggesting the possibility that the entire mapped onshore alignment of the RCFZ may be active.

More regionally, data has been presented that indicates that the RCFZ may be structurally connected to the Newport-Inglewood Fault Zone (Grant and Rockwell, 2002; Grant and Shearer, 2004) on the north and the San Miguel-Vallecitos fault or the offshore Descanso fault on the south, all of which are active faults. Sahakein, et. al. (2017) processed previously collected seismic reflection and bathymetric data, which indicated relatively narrow (2 kilometer) step-overs fault segments in offshore strands between the two major fault systems. This not only provides additional support of the structural connectively between the two fault systems but also indicates the possibility that they could erupt together with greater magnitude events of up to 7.5M. This larger fault system is thus over 150 miles in length.

3.3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

The results of our investigation indicate that artificial fill and two soil/geologic units underlie the site. As part of our engineering analysis we reviewed historic aerial photography and topographic maps and compared these to the present conditions at the site. Information from this work in conjunction with our current and previous boring data was used to estimate the depth and extent of geologic units below the site. The anticipated geometry is depicted on the attached geologic cross-sections (Figures 5 and 6). We anticipate that the actual fill depth will vary in areas between our borings based on uncertainties of the actual pregrading conditions at the site. Detailed descriptions of these materials are provided in the boring logs (Appendix A). Generalized descriptions are provided in the subsequent sections.

3.3.1 Pavement Materials

All borings in the pavement areas encountered approximately 3 inches of asphalt concrete pavement underlain by about 3 to 4 inches of aggregate base material.

3.3.2 Artificial Fill (af)

Artificial fill material was encountered beneath the aggregate base in all of the boring locations and extended to depths of 4 to 10 feet below the ground surface within our current borings. Based on our evaluation of borings and geologic cross sections from previous reports, the current geologic cross sections on Figures 5 and 6 indicate that the depth of fill likely extends up to approximately 10 feet below the central portion of the proposed building on the lower eastern pad. Fill up to about 25 feet in depth occurs below the west ascending slope on the westernmost side of the project area. The fill material consisted of layers of silty sand and silty sand with gravel and some debris. Due to the variability of the fill material, the presence of debris in the fill and the lack of compaction records from previous site grading operations, the fill material is considered undocumented and unsuitable for supporting structural loads. In addition, the only sampler blow count within the fill was 4 blows per foot which suggest a poorly compacted soil. It is likely that the fill deposits were derived from nearby campus materials during site grading and construction for previous hospital campus development.

3.3.3 Very Old Paralic Deposits (Qvop9)

Very old paralic deposits underlie the fill on the western side of the project area below the existing Emergency Department building. Although not encountered in any of the borings for this study, review of borings from a previous study of the Emergency Department building site indicate that this unit is comprised of a very dense, light reddish brown silty sand with gravel. This bottom of the unit is at approximate elevation 283 feet msl. This material is typically moderately to highly cemented and may be difficult to excavate, particularly during trenching operations for utilities or foundations. Based on the currently proposed building footprint, only the westernmost end of the building is potentially underlain by this units.

3.3.4 San Diego Formation (T_{sd})

The San Diego Formation was encountered below the fill at all boring locations where it was penetrated and below the very old paralic deposits in the area of the existing Emergency Department. This formation consists primarily of yellow to olive brown fine silty sand in a very dense and weakly to moderately cemented condition. It also contains some beds of sandy silt, and hard lean clay with sand. The sampler blow counts in this material obtained during drilling typically ranged from about 22 to more than 50 blows per foot. This unit has a slight dip to the south with an average bottom elevation of 220± feet msl and has a thickness up to about 60 in the area of the Emergency Department to the west. This geologic unit is also present on the slope outcrops to the east of the site and depicted in Figures 8 and 9.

3.3.5 Pomerado Conglomerate (T_p)

The Pomerado Conglomerate occurs directly below the San Diego Formation. It is likely that refusal of advancing augers during drilling was encountered at or near the top of this unit, with penetration of 3 to 7 feet below borings B-1 through B-3. Observations of the slope exposures along 6th Avenue east of the site indicate the unit consists of a brown to yellowish brown, cemented cobble conglomerate. The cobbles are typically 3 to 6 inches in size, but larger cobbles and boulders greater than 12 inches in size are occasionally present on the slope outcrops to the east of site as shown in Figures 8 and 9. The formation exposed on the slope outcrops typically contain between 20 to 50 percent cobbles. This geologic unit is estimated to be up to about 60 feet thick. Geologic mapping reveals that the Pomerado Conglomerate underlies the San Diego Formation at an elevation of approximately 220± feet MSL on the nearby slope. This material was encountered at depths of 13 to 20 feet below the existing ground surface (which corresponds to approximately Elevation +220 to +227 feet MSL). In general, the Pomerado Conglomerate has a gentle southwest dipping structure and is typically higher in elevation on the north and east sides of the site as depicted on the cross-sections.

3.3.6 Groundwater

Groundwater was encountered only within boring B-4 and described as a perched condition. In general, most of the soils encountered are in a moist condition below saturation levels necessary for free groundwater conditions. The majority of the laboratory moisture contents were on the order of 4 to 10 percent. However, three localized samples near the contact of the San Diego Formation and Pomerado Conglomerate had measured moisture contents of 19, 21 and 28 percent. It is possible that perched groundwater or seepage zones may be present at isolated locations. In particular, perched groundwater typically develops at the interface between more permeable fill and less permeable formational materials or between layers of variable permeability. It should be noted that groundwater levels at the site can fluctuate with time due to changes in weather, irrigation, construction, or other influences that were not present at the time the observations were made.

4 SEISMIC AND GEOLOGIC HAZARDS

We have reviewed the site with respect to the potential presence of geologic and/or seismic hazards. These hazards include landslides, expansive soils, liquefaction, seismic compression, fault surface rupture, and flooding. The following sections discuss these hazards and their potential at this site in more detail.

4.1 EXPANSIVE SOILS

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade.

Visual classification of the soils near anticipated subgrade elevations indicates that these soils primarily consist of non- to low-plastic silty sand and clayey sand. Two samples of the near-surface soils were tested for expansion index (ASTM D 4829) and results are presented in the boring logs and Appendix B. The laboratory results of these two samples indicate very low expansive potential based on Section 5.3 of ASTM D4829. Based on the results of our field investigation and previous experience in the site area, it is our opinion that the site soils generally have a very low to low expansion potential. Isolated zones of more expansive soils may also be encountered but are not anticipated. No special requirements for footing and floor slab reinforcement are recommended from a geotechnical perspective.

4.2 SEISMIC GROUND SHAKING

The project site, like all Southern California, is a seismically active area and is likely to experience ground shaking as a result of earthquakes on nearby or more distant faults. The Rose Canyon fault zone and Elsinore fault zones dominate the seismicity of the area. The Rose Canyon fault zone (CDMG, 1998) is located approximately 1.5 miles southwest of the project site.

We understand that the proposed structure will be designed in accordance with the requirements of the latest 2016 edition of the California Building Code (CBC). It should be noted that the seismic provision of the 2016 CBC are based on and refer to (for more requirements) “Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7” (referred to herein as “ASCE 7”).

Based on the results of our borings and geophysical Remi profiles in Appendix C, we classify the site as Site Class C. Site Class C is defined as a very dense soil and soft rock with average shear wave velocities within the upper 30 meters between 1,200 ft/sec. and 2,500 ft/sec., average SPT N>50, or average undrained shear strength Su > 2,000psf. The recommended seismic design parameters are presented in Table 1.

Table 1

Recommended 2016 CBC Seismic Design Parameters

Design Parameter	Symbol	Recommended Value	2016 CBC (ASCE 7) Reference(s)
Site Class	--	C	Section 1613A.5.2
Mapped spectral acceleration for short periods	S_s	1.18g	Section 1613A.5.1
Mapped spectral acceleration for a 1-second period	S_1	0.46g	Section 1613A.5.1
Site Coefficient	F_a	1.0	Table 1613A.5.3(1)
Site Coefficient	F_v	1.3	Table 1613A.5.3(2)
MCE* Peak Ground Acceleration (S_M at T=0)	PGA_M	0.53 g	n/a
MCE* spectral response acceleration for short periods	S_{MS}	1.18 g	Section 1614A.1.1 (Section 21.4)
MCE* spectral response acceleration at 1-second period	S_{M1}	0.61 g	Section 1613A.5.3 (Section 21.4)
Design Peak Ground Acceleration (S_D at T=0)	PGA_D	0.53 g	Section 1802A.2.7
Design spectral response acceleration (5% damped) at short periods	S_{DS}	0.79 g	Section 1613A.5.4 (Section 21.4)
Design spectral response acceleration (5% damped) at 1-second period	S_{D1}	0.41 g	Section 1613A.5.4 (Section 21.4)

*MCE : Maximum Considered Earthquake

Kleinfelder also performed a site specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-10 as specified by the 2016 CBC. The results of the analysis are presented in our January 31, 2019 technical memorandum which is included as Appendix E to this report. The technical memorandum presents the results of our site-specific ground motion hazard analysis, in terms of site-specific response spectra and seismic design parameters, as well as the methodologies and criteria used to perform our ground motion hazard analysis.

4.3 LIQUEFACTION

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or “cyclic mobility”, increased lateral earth pressures on retaining walls, post liquefaction settlement, lateral spreading and “flow failures” in slopes.

Based on the absence of near-surface groundwater, as well as the presence of dense to very dense formational soils, the potential for liquefaction of the subsurface soils at the site is considered low.

4.4 SEISMIC COMPRESSION

Seismic compression results from the accumulation of contractive volumetric strains in unsaturated soil during earthquake shaking. Loose to medium dense granular material with no fines or with low plasticity fines are most susceptible to seismic compression. The site will require excavation to proposed finish grade which we anticipate will be on the very dense San Diego Formation, therefore the hazard posed to the site by seismically induced settlement is considered low.

4.5 FAULT SURFACE RUPTURE

Review of readily available geologic and fault maps does not show any active or potentially active fault features passing through or nearby the site. The closest active fault to the site is the Rose Canyon fault, which is located approximately 1.5 miles to the southwest. The site is not within a State of California Alquist-Priolo Earthquake Fault Zone. In addition, published geologic maps do not show any faults crossing through or nearby the site. Finally, review of predevelopment aerial photographs do not show geomorphic features or lineaments indicative of faulting across the site. Based on this information, the geologic hazard with respect to fault rupture is considered low.

4.6 LANDSLIDES

Landslides are deep-seated ground failures (tens to hundreds of feet deep) in which a large section of a slope slides downhill. Landslides are not to be confused with minor surficial slope failures (slumps), which are usually limited to the topsoil zone and can occur on slopes composed of almost any geologic material. Landslides can cause damage to structures both above and below the slide mass. Undermining of foundations can damage structures above the slide area. Areas below a slide can be damaged by being overridden and crushed by the failed slope material. The site has some perimeter retaining walls that are up to 20 feet in height that are located generally on the north and south perimeter of the site. Steep slopes exists on the north and west perimeter of the site. The northside slope surface is exposed and westside slope surface is covered with shotcrete.

Kleinfelder has reviewed slope stability evaluations performed for previous projects on the Mercy campus which all indicate a low potential for landslides and calculated safety factors in excess of 1.5 for the static case and 1.2 for the pseudostatic case. The northern slope which ascends up to approximately 25 feet to the existing hospital building has an inclination between 1.5 horizontal to 1 vertical (1.5H:1V) and 2H:1V._Visual examination indicates this slope has performed well over the lifetime of the existing building. It is suspected that the outer portion of the slope consists of fill if a sloping excavation was used for construction of the existing BHU building, however, the slope could be all cut into formational soils if temporary shoring was used in construction. The existing western slope will essentially be eliminated by the proposed construction since an approximate 50-foot high subterranean wall will be constructed to the top of slope elevation.

We recommend that fill slopes have a maximum inclination of 2H:1V to achieve a calculated safety factor in excess of 1.5 for the static case and 1.2 for the pseudostatic case.

4.7 SURFICIAL STABILITY AND EROSION

The San Diego and Pomerado Conglomerate formations that underlie the site are exposed on the slopes located along 6th Avenue east of the project site. The formations are predominately granular with weak to moderate cementation. These units are considered moderately susceptible to erosion. In addition, heavy rainfall could trigger shallow surficial slope movement. The erosion should be monitored over time and mitigation measures should be implemented if necessary. We have included surface drainage and erosion control recommendations in Section 5.9 of this report for areas of soil that may be disturbed during construction.

4.8 TSUNAMIS AND SEICHES

A tsunami is a giant sea wave (which can reach over 50 feet in height) usually generated by catastrophic displacement on a submarine fault. Tsunamis can travel at speeds of hundreds of miles per hour over distances of thousands of miles. In the open ocean, tsunamis have large wavelengths and are difficult to detect. As the sea wave approaches shore, the wave decreases in wavelength and increases in amplitude (height). Large tsunamis can travel well beyond the normal wave break of the shoreline and cause damage to near shore structures.

A seiche is an oscillation (wave) of a body of water in an enclosed or semi-enclosed basin that varies in period, depending on the physical dimensions of the basin, from a few minutes to several hours, and in height from several inches to several feet. A seiche is caused chiefly by local changes in atmospheric pressure, aided by winds, tidal currents, and occasionally earthquakes.

The project site is located about 5 ½ miles from the Pacific Ocean and is located at an elevation of approximately 233 feet or more above mean sea level. Additionally, the site is not located adjacent to any large bodies of water that could adversely affect the site in the event of earthquake-induced failures or seiches. Therefore, the hazard with respect to a tsunami or seiche is considered low.

4.9 FLOODING

Flooding occurs as a result of several factors in developed areas. These factors include rainfall rates that exceed an area's ability to absorb or control the runoff; impounded water retained behind a flood control structure (upstream-inundation), failure of a flood control structure (downstream-indundation), Seiches, or tsunami.

The Federal Emergency and Management Administration (FEMA) maintains a collection of Flood Insurance Rate Maps (FIRM), which cover the entire United States. These maps identify those areas which may be subjected to 100-year and 500-year cycle floods. A set of these maps for the County of San Diego are available for viewing on the FEMA website (https://msc.fema.gov/portal). Based on our review of FEMA map panel 06073C1618G, the site is not within any designated flood zones and therefore the potential for flooding of the proposed development is considered low.

5 DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

Based on the results of our background review, subsurface exploration, laboratory test results, and engineering analyses, it is our opinion that the construction of the proposed replacement hospital building is feasible from a geotechnical standpoint provided the recommendations of this report are incorporated into the design and construction of the project.

The following recommendations were prepared based on our understanding of the project as depicted on the Sheets 1.01 through 1.16 of progress plans titled "Scripps Mercy Hospital San Diego," dated November 19, 2018, and prepared by CO Architecture.

Geotechnical considerations include the following:

- The site is located in the seismically active Southern California area. The structures should be designed in accordance with the 2016 California Building Code requirements for seismic design.
- There are no known active faults crossing the site. Based on this information it is our opinion that the hazard with respect to fault rupture is low.
- Static groundwater was not encountered within the depths explored. However, perched water or seeps might be encountered in excavations. One exploration location encountered seepage or a localized perched groundwater condition and elevated moisture contents were measured in several samples
- Existing fill material was encountered to depths of up to 10 feet below the existing ground surface from the lower parking lot and up to 25 feet below the upper northern pad. The existing fill soils are considered undocumented and unsuitable for support of building foundation loads in their current condition.
- We anticipate that the bottom of mat foundation elevation for the proposed building will be underlain by both undocumented fill and the San Diego Formation. Remedial grading will be required to remove and recompact all of the undocumented fill and to overexcavate some of the formational areas so that the entire lower mat foundation is supported on fill. Drilled pier foundations are recommended to support the upper one-story portion at the western end of the building.

- The on-site formational soils are generally suitable for re-use as engineering materials following removal of oversized and deleterious materials and proper moisture conditioning.

5.1 SITE GRADING

The following recommendations were prepared based on our understanding of the project as previously described in this report. Kleinfelder should be provided with updated plans by the design team if design is modified.

5.1.1 General

All site preparation and earthwork operations should be performed in accordance with applicable codes including the 2016 California Building Code, Section 1803A. All reference in this report to maximum dry density is established in accordance with American Society for Testing and Materials (ASTM) ASTM D 1557. We recommend that site earthwork and construction be performed in accordance with the following recommendations and guidelines presented in the Suggested Guidelines for Earthwork Construction included in Appendix F. In case of conflict, the following recommendations supersede those outlined in Appendix F.

5.1.2 Site Preparation

The existing Scripps Behavior Health Unit (Clinic Building #14), paved parking lots and portions of the northeast access road will be demolished and removed prior to construction of the proposed Hospital Building. Existing slope improvements such as shotcrete on the western slope will also be removed. Man-made structures, including buried pipes, utilities, etc., should be completely removed within the building pad. Excavations for removal of any man-made items should be dish-shaped and backfilled with properly compacted engineered fill per Section 5.1.4. All surficial vegetation and deleterious material should be stripped and completely removed from the proposed site area.

5.1.3 Subgrade Preparation

After demolition, all of the existing undocumented fill should be removed and recompacted with the building area. In addition, areas with little to no fill at foundation subgrade should be overexcavated to remove cut/fill transitions and provide a minimum thickness of compacted fill below the mat foundation of 5 feet. The exposed surface should be scarified to a depth of 12 inches, moisture conditioned and compacted. The need for scarification may be evaluated and waived by the geotechnical engineer at the time of grading if cobble, cemented material or other conditions are deemed appropriate.

5.1.4 Engineered Fill

The existing undocumented fill material is considered suitable for re-use as engineered fill, however the potential presence and amount of debris within undocumented fill is difficult to assess from the limited small diameter borings. The onsite San Diego Formation materials may be used as engineered fill, as they are not anticipated to contain oversized rock, organic materials, and deleterious debris. If encountered, oversize material in excess of 6 inches in diameter should not be used in engineered fill and material larger than 3 inches should not be used within the upper 6 inches for foundation subgrade. The onsite soil placed as engineered fill should be moisture conditioned between 0 and 3 percent above optimum moisture content, and compacted to a minimum of 95 percent relative compaction in structural areas and 90 percent in non-structural areas, based on ASTM D 1557.

Import materials used as engineered fill should consist of clean, granular material that has less than 30 percent passing the #200 sieve, a minimum R-value of 20, and expansion index of 20 or less as evaluated by ASTM 4829. Imported engineered fill should be moisture conditioned between 0 to 3 percent above optimum moisture content and compacted to a minimum of 95 percent relative compaction in structural areas and 90 percent in non-structural areas, based on ASTM D 1557.

Although the optimum lift thickness for fill soils will be dependent on the size and type of compaction equipment utilized, fill should generally be placed in uniform lifts not exceeding approximately 8 inches in loose thickness. Oversized material, rocks, or hard clay lumps greater than 6 inches in dimension should not be used in compacted fills and greater than 3 inches in dimension should not be used in the upper 3 feet in structural areas. In pavement and exterior flatwork areas the upper 12 inches of subgrade soils should be moisture conditioned 0 to 3 percent above optimum content, and compacted to at least 95 percent relative compaction, just prior to placement of aggregate base.

5.1.5 Pipe Bedding and Trench Backfill

Pipe bedding should consist of sand or similar granular material having a sand equivalent value of 30 or more. The sand should be placed in a zone that extends a minimum of 6 inches below and 12 inches above the pipe for the full trench width. The bedding material should be compacted to a minimum of 90 percent of the maximum dry density. Trench backfill above pipe bedding may consist of approved, onsite or import soils placed in lifts no greater than 8 inches loose thickness and compacted to 90 percent of the maximum dry density.

5.1.6 Temporary Excavation Slopes

Shallow, temporary utility trench excavations are anticipated for installation of the required utility lines. If very steep or vertical-sided excavations in excess of 5 feet deep are necessary, we recommend the sidewalls be shored in accordance with OSHA standards to provide temporary trench stability during construction. The contractor should be responsible for the structural design and safety of the temporary shoring system and we recommend that this design be submitted to Kleinfelder for review and approval.

For preliminary planning of OSHA sloping and shoring requirements, we recommend that fill materials be considered as Type C soils and that native formational materials be considered as Type B soils. The actual OSHA soil type should be determined by the contractor's "competent person" based on conditions exposed in the field.

Heavy construction loads, such as those resulting from stockpiles and heavy machinery, should be kept a sufficient distance away from the top of the excavation or shoring to prevent unanticipated surcharge loading. All surface water should be diverted away from excavations.

5.1.7 Excavation Considerations

Excavation in the existing fill and San Diego Formation can generally be accomplished with conventional, heavy earthmoving equipment in good operating condition. Although not anticipated, if excavations extend into the Pomerado Conglomerate, the excavation effort is anticipated to be difficult due to cemented layers of cobbles and gravel. Potential excavations for utility trenches, soldier beams for shoring, drilled piers or tiedown anchors within the Pomerado Conglomerate may be particularly difficult due to cobble size and content.

5.1.8 Dewatering

Static groundwater was not encountered within the exploratory borings performed by Kleinfelder. During our field exploration Boring B-4 location, we encountered seepage or a perched groundwater condition. Elevated moisture contents were also measured within a few soil samples near the contact of the San Diego Formation and Pomerado Conglomerate. The water is likely associated with perched water seeping the ground from the landscape areas located nearby at permeable lenses within the formation soils. The regional groundwater table is probably in excess of 100 feet below the hospital campus. Significant groundwater is therefore not anticipated to be encountered within the planned excavation depths and it is anticipated that dewatering will not be required. However, some minor groundwater may be encountered in the excavations due to perched groundwater that may be located near the contact of differing geologic units or soil types. Perched groundwater, if encountered, may require collection, control and disposal of minor amounts of water.

5.2 TEMPORARY SHORING

While the details of site excavation and temporary excavation support are not known at this time, we anticipate that the proposed excavation will require temporary shoring around most, if not all, of the perimeter of the site during construction to protect existing improvements. Temporary shoring is anticipated to be required around portions of the excavation perimeter to protect existing improvements such as buildings, parking areas and roads, utilities, power poles, etc. Based on the proximity and elevation of excavations relative to existing buildings such as the Emergency Department on the west and hospital on the northwest, underpinning may also be necessary. The shoring height may be up to approximately 55 feet in this area to accommodate the removal and recompaction of formational soils and remove a cut/fill transition under the building.

Shoring is anticipated to consist of closely-spaced soldier piles and wooden lagging as it is typically the most feasible and economical method for soils in the San Diego area. However, drilling holes for soldier beams within the Pomerado Conglomerate below approximate elevation 220 feet msl would require a larger diameter and additional effort than other materials onsite. Based on our experience with nearby projects, the caving potential of the onsite soils is moderate for undocumented fill and low for formational materials. Recommendations for this system are provided in the following sections. Section 5.2.4 provides a summary of geotechnical requirements for earth retaining shoring that are addressed in Section 1812A.4.1 of the 2016 CBC.

To accommodate installation of the soldier beams, wide-flange beam sections may be installed into pre-drilled holes surrounded by concrete below excavation depth and cement slurry within the lagging depths. In the unanticipated event that caving of the drilled holes occurs, drilling slurry or casing may be required. In addition, caving of drilled holes for tieback anchors is possible in undocumented fill.

5.2.1 Tieback Anchors

Tiebacks derive their load capacity through frictional resistance along the grouted “bonded zone” which is located beyond the active wedge. For design of tieback anchors, we recommend the active wedge may be assumed at an angle of 30 degrees from vertical, passing through a point located at least 5 feet behind the bottom of the excavation. We recommend the portion of the anchor within the “unbonded zone” within the active wedge either have a sleeve so that it is not bonded to the grout or be backfilled using sand/cement slurry. The shoring contractor should determine the suitable drilling method for tieback installation based on the subsurface conditions at the site and on their experience with similar materials. Tiebacks should be installed at angles between 15 and 30 degrees from horizontal.

Since the load-carrying capacity of the tieback anchors will depend on various site-specific equipment and method-related factors, design tieback capacities should be confirmed by performance testing. We recommend performance testing and proof testing of anchors be performed in accordance with the latest edition of the Post-Tensioning Institute's (PTI) Recommendations for Prestressed Rock and Soil Anchors. Performance test for the anchors shall be at a minimum of two (2) times the design loads and shall not exceed 80 percent of the specified minimum tensile strength of the anchor rod. A creep test is required for all pre-stressed anchors that are performance tested. All production anchors shall be tested at 150 percent of design loads and shall not be greater than 70 percent of the specified minimum tensile strength of the anchor rod

Based on experience from nearby projects, the unit friction between the grout and the soil may be assumed to be on the order of 4,000 psf for design of tiebacks if post-grouting is performed. However, it is important that the unbonded length not be grouted if post-grouting is performed. If post-grouting is not performed, we recommend a unit friction of 1,000 psf be used. If tieback anchors are installed at an angle below the horizontal, tieback resistance should be taken as the horizontal component of the total anchor capacity. Additionally, the shoring designer should be aware that the vertical component of the total anchor capacity may act as a downward load on the shoring system.

5.2.2 Timber Lagging

Timber lagging may be used between the soldier piles to support the exposed soils. Since the lagging is generally left in place, treated lumber should be used. Lagging should be designed for the lateral pressures recommended in Section 5.2.3 of this report. The soil bridging between the stiffer soldier beam elements and the intermediate wooden lagging results in a reduced lateral earth pressure in the lagging area. The CBC allows soil arching effects to be considered in the design of lagging. We recommend 0.6 of the design earth pressure, or a uniform pressure of 400 psf for level ground without surcharge. It is our understanding that conventional design of earth pressures for soldier beam and wooden lagging walls incorporate this pressure into standard design tables for wooden lagging but do not include surcharge pressures. Therefore, additional surcharge pressures should be added if these tables are utilized.

If possible, structural walls should be cast directly against the shoring, eliminating the need for backfilling a narrow space. Voids between the soil and lagging should be grouted to mitigate the potential for the voids to propagate to the surface and to protect the existing improvements.

Voids identified during lagging operations may also be immediately filled by compacting soil behind the lagging.

5.2.3 Lateral Pressures

Cantilever shoring can typically be used for retained shoring heights up to about 12 to 15 feet with level backfill and no surcharge loading, however this height could be increased by using larger beams. Tie-back anchors would be needed for higher walls or sloping backfill. Cantilever shoring supporting undocumented fill should be designed to resist an equivalent fluid weight of 35 pcf for level ground conditions in fill and 30 pcf for level ground conditions in formational material. Thirty percent of any areal surcharge adjacent to the shoring (including soil stockpiles) may be assumed to act as a uniform horizontal pressure against the shoring for cantilever conditions.

For higher walls with level ground, tied-back shoring systems should be designed to resist a uniform horizontal soil pressure of 30H (in psf) at the west side of the excavation and 25H (in psf) at the north, south, and east sides of the excavation, where H is the wall height in feet. Increased loads due to ascending slopes should also be added to this. Thirty percent of any permanent or temporary surcharge loads adjacent to the shoring (including existing structures, temporary soil stockpiles, material staging, construction trailers, etc.) may be assumed to act as a uniform horizontal pressure against the shoring. Horizontal pressure resulting from foundations adjacent to the shoring system can be estimated using the method presented on Figure 10 (based on US Navy, 1986). Strip footings can be represented as line loads and spread footings can be represented as point loads. Special cases such as combinations of sloping and shoring or other surcharge loads (not specified above) may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical engineer on a case-by-case basis. The above pressures do not include hydrostatic pressures since groundwater was not encountered within the depths explored at the site. A graphical summary of the recommended lateral earth pressures for temporary shoring design is presented in Figure 11.

All soldier piles should extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. An equivalent fluid unit weight of 400 pcf may be used in formational soils and 300 pcf may be used in fill for allowable passive pressure against soldier piles that extend below the level of excavation. To account for three-dimensional effects, the passive pressure may be assumed to act on an area 2 times the width of the embedded portion of the pile, provided adjacent piles are spaced at least 3 pile diameters center-to-center. Additionally, we recommend a factor of safety of 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 4,000 psf in formational soils and 3,000 in fill soils.

Lateral movement of a shored excavation will depend on the type and relative stiffness of the system used, the workmanship of the contractor, and other factors beyond the scope of this study. The shoring engineer should design the system to limit lateral movements and settlements so that effects on adjacent structures, existing utilities to remain in place, and other existing site improvements are minimized.

In addition to monitoring of existing structures, horizontal and vertical movements of the shoring system should also be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor's responsibility. At a minimum, we recommend that the tops of the soldier beams be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the foundation is completed. The soldier beams should be surveyed at spacings no greater than 50 feet on-center.

5.2.4 Summary of CBC Shoring Requirements

In addition to the above recommendations, the following 13 recommendations are presented to correspond to the 13 issues contained in Section 1812A.4.1 of the 2016 CBC for geotechnical requirements of earth retaining shoring.

1. The drill hole diameter shall be sized to provide a minimum grout cover of 0.5 inches, and the area of pre-stressing strands within the hole should not exceed 15 percent of the total area of the hole for multiple-element tendons. In general, the holes should have a minimum diameter of 6 inches. The minimum center-to-center spacing should be at least four times the nominal anchor diameter or 4 feet, whichever is greater. Reduction factors for group effects may be provided if tiebacks need a closer spacing.

2. Tieback anchors derive their load capacity through frictional resistance along the grout “bonded zone”, which is located beyond the “active wedge”. The length of the unbounded length will decrease with depth, the active wedge may be assumed at an angle of 30 degrees from the vertical, passing through a point located at least 5 feet behind the bottom of the excavation. The tieback anchors should have a maximum un-bonded length of 20 feet and a minimum bonded length of 20 feet.

3. The anchor length should be determined by the structural engineer based on the provided soil strength/grout bond, the structural load and the design anchor spacing.

4. We recommend an allowable bond stress of 4,000 psf at the ground /grout interface. A factor of safety of 1.67 may be used to calculate the ultimate bond stress for temporary anchors. We understand that these anchors are temporary. For permanent anchors (if any), a minimum factor of safety of 2.0 shall be applied to ground/grout interface as required by PTI-2004 Section 6.6.

5. We recommend a minimum grout pressure and post-grout pressure of 300 psi for tieback installation for the anchor.

6. We understand that the shoring is temporary during construction of the subterranean levels. Therefore, corrosion protection of the temporary anchors is not required. Class II corrosion protection should be applied if the shoring is utilized for more than 24 months. Class I corrosion protection would be required for permanent anchors.

7. Performance test for the anchors shall be at a minimum of two (2) times the design loads and shall not exceed 80 percent of the specified minimum tensile strength of the anchor rod. A creep test is required for all pre-stressed anchors that are performance tested. All production anchors shall be tested at 150 percent of design loads and shall not be greater than 70 percent of the specified minimum tensile strength of the anchor rod.

8. Recommendations for lateral earth pressure and surcharge pressure were presented in previous sections of this geotechnical report. Design for seismic increment of earth pressure loading is not required since we understand the shoring is temporary.

9. We recommend maximum lateral deformations on the order of 1-inch at the top of the soldier pile, ½-inch at the tie-back anchor locations and ½-inch at the drilled pier concrete shafts at the lowest grade level.

10. We recommend an allowable vertical soil bearing pressure of 3,000 psf, friction resistance of 300 psf and lateral passive soil resistance of 400 psf in formational soils and 300 psf in fill for the drilled pier concrete shafts. The bottom of drilled holes should be cleaned and observed by the geotechnical engineer if end bearing is used in design. A safety factor of 3 was used on bearing pressure, 2 for frictional resistance, and 1.5 for passive resistance. Additionally, we recommend a factor of safety of 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 4,000 psf in formational soils and 3,000 psf for fill.

11. To account for three-dimensional effects, the passive pressure may be assumed to act on an area 2 times the width of the embedded portion of the pile, provided adjacent piles are spaced at least 3 pile diameters, center-to-center.

12. The shoring contractor should be responsible for using a drilling method to establish a stable hole with the specified dimensions and tolerances. Rotary, auger or percussion drilling methods are acceptable. Temporary casing is not anticipated but may be utilized if caving is encountered. Centralizers should be used on the tendons.

13. We recommend that geotechnical observation and monitoring of shoring installation be performed by Kleinfelder on a continuous basis. Installation should be performed in accordance with the recommendations in this report and Section 1812A.4.1 of the 2016 CBC

5.3 FOUNDATION RECOMMENDATIONS

We recommend that foundations be supported entirely on either formational material or compacted fill to mitigate the potential for differential settlement. We understand that a mat foundation will be utilized to support the lower building footprint and recommend that drilled piers extending into formational soils be utilized for the upper one-story western portion. Based on the currently proposed bottom of mat foundation elevation of 235 feet and understanding of subsurface conditions, we anticipate that the mat foundations for the lowest level of the proposed building will traverse a cut fill transition. Therefore, we recommend that all of the existing undocumented fill be removed and recompacted and that the cut or shallow formational areas be overexcavated to a minimum depth of 5 feet below the bottom of mat foundation. All structural fill should be placed at a minimum relative compaction of 95 percent of the maximum dry density determined by ASTM D 1557. Due to the presence of up to 25 feet of fill below the upper western pad by the emergency department, we recommend that drilled pier foundations be embedded into formational soils in this area. The existing fill soils at the site are not considered suitable for support of building foundation loads in their current condition and are too deep to remove and recompact in this area. Recommendations for shallow spread or continuous footings are also presented below as they may be utilized in the loading dock area and possibly other locations.

5.3.1 Mat Foundations

Based on the currently proposed finish floor elevation, we understand the building will likely be supported on a mat foundation bearing entirely on compacted fill. The mat foundation bearing on the compacted fill with a minimum relative compaction of 95 percent may be designed using an allowable bearing pressure of 10,000 psf. We recommend an uncorrected vertical modulus of subgrade reaction for a one-foot square plate, k_1, of 275 pci be used for design of the mat foundation. The k_1 value is based on a settlement of 1 inch for a one-foot square plate and should be modified to account for the mat foundation width, B. The modified vertical modulus of subgrade reaction may be calculated using the following equation with actual mat dimensions:

$$k_s = k_1\left(\frac{B+1}{2B}\right)^2$$

Where:

k_1 is the subgrade modulus for a one-foot square foundation (= 275 pci).

k_s is the subgrade modulus in pci for a foundation with width B (B is the least dimension of the mat in feet). This value approaches 0.25 k_1 for large dimensions.

The allowable design bearing value can be increased by one third for short-term loading due to seismic and wind forces. The mat foundation should have a minimum embedment of 18 inches and should be designed by a licensed structural engineer in the state of California for the specific loading conditions.

5.3.2 Drilled Pier Foundations

Foundations underlain by fill at the top of the western slope should be supported by drilled pier foundations to mitigate the potential for differential settlement since the undocumented fill is too deep to remove and recompact. Axial capacity for 2-, 3- and 4-foot diameter CIDH piles was evaluated using the computer program SHAFT version 5.0 by Ensoft, Inc., which uses methodologies based on the procedures outlined in the FHWA drilled shaft design manual (Reese and O'Neill, 2010) to estimate pile capacities. Our evaluations neglected the skin frictional capacity in the undocumented fill soils due to the potential for long term settlement. Although there is variable depth of fill across the site, we performed calculations for the estimated deepest fill depth of 260 feet msl.

A plot of recommended CIDH pile embedment depths versus allowable downward axial capacities is presented in Figure 12 for pile diameters of 2, 3 and 4 feet. These capacities are based on a safety factor of 3.0 on end bearing and 2.5 on skin friction. The actual depth to formational materials should be verified during foundation drilling and the corresponding shaft length either verified or revised at that time. We recommend a minimum embedment of 9 feet into formational materials for axial capacity.

Anticipated total settlements are not expected to exceed 1 inch and differential settlements over a 40-foot span are not expected to exceed 75 percent of the total settlement. The CIDH piles should contain reinforcing steel as determined by the project structural engineer.

The lateral load capacity of CIDH piles will depend upon several factors including the pile diameter and embedment depth, the depth of existing fill at the pile location, the pile spacing within a group, and the structural stiffness of the pile. Kleinfelder can evaluate lateral loading on piles if this information is required in design.

Performance of CIDH piles is strongly dependent on construction methods and procedures. The foundations should be constructed only by contractors highly experienced in this type of construction, and under strict construction monitoring and quality control by the geotechnical engineer. This is especially important for single CIDH pile supports due to the lack of redundancy.

Based on the anticipated site locations, the fill and San Diego Formation may be excavated with conventional heavy-duty drilling equipment. Groundwater was not encountered within the test borings performed at the site; however, some water seeps could be encountered during CIDH construction. In particular, seepage can occur at the interface of the more permeable fill and the less permeable underlying formational materials. If such zones are encountered during construction, the geotechnical engineer should assess the conditions and develop appropriate recommendations on a case-by-case basis. Any subsurface or surface water that enters the CIDH excavations should be removed prior to the placement of concrete. The CIDH piles should not deviate by more than 1 percent from vertical. The excavation bottoms should be cleaned of debris, soil cuttings or other deleterious material prior to the placement of reinforcing steel or concrete.

We recommend that a representative of Kleinfelder observe excavation of the CIDH piles to log conditions encountered, confirm the subsurface conditions and to check that required embedment into the formational soils is achieved. The drilling foundation contractor should have a sufficient quantity available of reinforcing cages, so that CIDH shafts are backfilled with concrete the same day of drilling, or no more than 24 hours after drilling. We should also observe the CIDH excavations prior to the placement of steel reinforcement or concrete to check for plumbness and that the bottom has been properly cleaned.

We did not observe caving or sloughing from the sides of the holes drilled for our foundation investigation. Consequently, we do not expect that casing or drilling fluids will be needed for this project. However, if unexpected soft or loose soil zones are encountered within the fill, or there are zones of relatively cohesionless or clean sands/gravel in the fill or formation, temporary casing may be needed to support hole sides where these zones are encountered.

A representative of our firm should observe concrete placement, installation of reinforcement, and obtain appropriate samples of concrete for strength testing. In addition, we should perform appropriate field tests, such as slump and air entrainment of the concrete, to check conformance with project specifications.

5.3.3 Spread Foundations

For foundations entirely in compacted fill soils, an allowable foundation pressure of 3,000 pounds per square foot (psf) can be used for retaining walls, screen walls or equipment pads. For foundations entirely in formational soils, an allowable foundation pressure of 5,000 may be used. The allowable design bearing value can be increased by one third for transient loading due to seismic and wind forces. Localized foundation excavations which encounter fill should be deepened and filled fill a minimum 3-sack cement slurry or unreinforced concrete up to design foundation elevation.

Anticipated total settlements will be evaluated when foundation loads are provided but are not expected to exceed 1 inch. Differential settlements over a 40-foot span are not expected to exceed 75 percent of the total settlement. Shallow foundations should contain reinforcing steel as determined by the project structural engineer.

Resistance to horizontal loadings can be developed by passive earth pressure on the sides of footings and frictional resistance developed along the footings bottoms. Passive resistance to lateral earth pressures may be calculated using an allowable equivalent fluid unit weight of 450 psf within formational soils or 350 psf within compacted fill. An allowable frictional coefficient of 0.30 may be applied to vertical dead loads supported either formational or fill soils. Both of these values were calculated with a safety factor of 1.5. The passive pressure and frictional resistance can be combined to resist lateral loads, provided that the larger value is reduced by 50 percent.

Footings may experience a reduction in bearing capacity or an increased potential to settle when located in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse, and/or a loss of serviceability. To reduce this risk, utility excavations should not extend below a 2H:1V plane projected downward from 12 inches above the bottom of the outside edge of the footing. Also, no parallel utility excavations should be made within a lateral distance of 18 inches outside the footing.

Prior to placing reinforcing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. All footing excavations should be observed by the project geotechnical engineer immediately prior to placement of reinforcing steel and concrete to check that the recommendations contained herein are implemented during construction.

5.3.4 Tiedown Anchors for Uplift Resistance

Recommendations for tiedown anchors are provided in case they are needed for resist resistance. For design of tiedown anchors, we recommend using an average allowable soil to concrete friction value of 1 ksf, and an unbonded length of 10 feet. We have assumed a safety factor of 2.0. From a geotechnical perspective, anchor design is dependent on the allowable friction, diameter, length and spacing. For example, a 16-inch diameter pile and an ultimate capacity of 120 kips, would result in an approximate bonded length of 30 feet and a total depth of 40 feet. Prior to installation of production anchors, we recommend that a load test be performed to verify the assumed friction value. Additional recommendations, installation specifications and load test specifications can be provided when the selected anchor type and structural loads are provided.

The performance of tiedown anchors is highly dependent on installation procedures and requires continuous observation. Following installation and foundation construction, the anchors should be proof tested and locked off.

5.4 INTERIOR CONCRETE SLABS-ON-GRADE

Interior concrete slabs-on-grade may be used with structures not supported on mat foundations. Engineered fill supporting concrete slabs should be scarified to a depth of 6 inches, moisture conditioned to within 3 percent above optimum and compacted to at least 95 percent relative compaction per ASTM D1557. A modulus of subgrade reaction, k, of 175 pounds per cubic inch (pci) may be used to design floors, pavements, and walkways on the compacted subgrades. For slabs subjected to pedestrian-type loadings, we recommend a minimum floor slab thickness of 4 inches.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, and/or curling of the slabs. High water-cement ratio and/or improper curing will increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

The floor slab should be underlain by at least 4 inches of clean coarse sand or fine gravel to provide a capillary moisture break and uniform support to the slab. In cases where the floor may have a vapor/moisture sensitive covering (e.g. tile, linoleum, carpet, wood), may be in a humidity controlled environment, or may likely have one or both of these conditions in the future, we recommend a polyolefin vapor barrier membrane be utilized between the prepared subgrade and the bottom of the floor slab.

Subsurface moisture and vapor naturally migrate upward through the soil. Where the soil is covered by a building or pavement, this subsurface moisture will collect and transmit through the concrete slab-on-grade. Traditional Visqueen vapor barriers may be considered marginally effective and have been shown to eventually disintegrate with time. To reduce the impact of this subsurface moisture and the potential impact of future introduced moisture (such as landscape irrigation or precipitation) we recommend utilizing a polyolefin vapor barrier membrane between the subgrade and slab-on-grade. This vapor barrier membrane should consist of a polyolefin sheeting at least 15 mil in thickness, have a water vapor permeance less than 0.01 perms (ASTM F1249), a puncture resistance of at least 2,200 grams (ASTM D1709), and a tensile strength of at least 45 lbf/in (ASTM D882).

The polyolefin vapor barrier membrane described above should be highly resistant to tearing, cracking, flaking, or puncturing during construction and should not disintegrate with time. A granular subbase below the membrane or a sand or gravel layer on top of the membrane is not required. In accordance with recommendations in ACI guidelines and many flooring companies, placement of the concrete slab may be directly on the vapor barrier. This eliminates the potential for water to be trapped in the blotter layer that could later be transmitted through the slab and adversely affect the flooring system. However, a reduced joint spacing, slab reinforcement, a low shrinkage mix design, and/or other measures to reduce the potential for slab curl should be implemented by the concrete slab designer.

We recommend that the vapor barrier be installed in accordance with ASTM E1643, “Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs”. Some salient features of ASTM E1643 are discussed below. All joints and seams should have a minimum 6-inch overlap and be taped. The area of tape adhesion should be free from dust, dirt, and moisture. All penetrations must be sealed using a combination of membrane, tape, and mastic. The tape and mastic used should conform to the vapor barrier manufacturer's recommendations. Care should be taken at the lateral terminations so that vapors do not go around the membrane. This may be accomplished by placing the membrane on top of the footing and against the vertical wall so that the membrane will be sandwiched between the footing, vertical wall, and poured concrete floor slab. If damaged, the membrane should be repaired prior to placing concrete.

It is emphasized that we are not floor moisture-proofing experts. We make no guarantee nor provide any assurance that use of the capillary break will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level. The builder and designers should consider all available measures for slab moisture protection. Exterior grading and/or adjacent landscaping have an impact on the potential moisture beneath floor slabs. Exterior grading and/or adjacent landscaping should be designed to address the potential for increased moisture below moisture sensitive slabs and should at least reference the recommendations contained in Section 5.9 (Site Drainage) of this report.

5.5 PERMANENT BASEMENT AND SITE RETAINING WALLS

Permanent restrained retaining walls for the subterranean levels will likely be placed both against the temporary shoring in areas where used and be backfilled in areas where temporary excavation slopes are used. After permanent bracing such as floor slabs has been installed, it is important that the tieback anchors used for the shoring be detentioned and documented by the geotechnical engineer. Cantilever retaining walls are not shown on the current plans but may be used in some areas of the site such as loading dock area, therefore, recommendations are provided in the following sections if site walls are included in project design.

The basement walls should be dampproofed and permanent drainage provisions should be made against the permanent wall. Vertically placed composite geosynthetic drainage strips are typically used rather than open graded gravel. The wall drains should be connected to closed conduits at the base of the walls and brought to a storm drain, sump or other suitable discharge location.

5.5.1 Wall Foundations

Cantilevered masonry and poured-in-place concrete walls with shallow reinforced concrete footings are considered suitable for site retaining walls. Cantilevered concrete retaining walls may be supported on shallow continuous footings founded entirely on either engineered fill or undisturbed native formational soil. Where existing undocumented fill soils are present, the fill should be removed and recompacted so that retaining walls over 3 feet in height are supported by engineered fill. Shallow foundations supported on engineered fill or formation should be designed for an allowable bearing pressure of 3,000 psf. Estimated total settlements for retaining walls constructed in accordance with the recommendations contained herein are anticipated to be less than ¾ inch. Differential settlements are expected to be less than ½ inch within 40 feet.

Retaining wall foundations should be embedded at least two feet below the lowest adjacent grade or to the depth necessary to provide adequate factors of safety against sliding and overturning as determined by the retaining wall designer, whichever is greater. Reinforcement should be provided as required by the Regional Standard Drawings (if used) or as directed by the wall designer for load carrying purposes.

All footings should be extended in depth as necessary so that no existing or proposed utility trenches will extend below a plane having a downward slope of 2H:1V from 12 inches above the bottom edge of the footing. In addition, no parallel trenches should be within 18 inches from the closest edge of the footing. New footings should not be excavated below the bottom of adjacently located existing foundations. For all retaining walls, we recommend a minimum horizontal distance from the outside face of the footing to daylight of 6 feet.

The basement walls will be supported on the mat foundation for the building. Recommendations for mat foundation design were previously provided in Section 5.3.1 of this report.

5.5.2 Static Lateral Earth Pressures

Lateral pressures acting against masonry and poured-in-place concrete retaining walls can be calculated using soil equivalent fluid weight (efw). The efw value used for design depends on allowable wall movement. Walls that are free to rotate at least 0.5 percent of the wall height can be designed for the active efw. Retaining walls that are restrained at the top (such as basement walls), or are sensitive to movement and tilting should be designed for the at-rest efw.

Values given in the Table 2 below are in terms of equivalent fluid weight and assume a triangular distribution. These values assume that onsite or imported, sandy soils (SP, SM, SC) will be used as backfill and that the backfill is well drained and above the static water table. If walls with undrained backfill are to be used Kleinfelder should be consulted for additional evaluation and recommendations.

Table 2
Equivalent Fluid Weights (efw) for Calculating Lateral Earth Pressures

Wall Type	Conditions	Level Backfill (psf)
Restrained - Basement Wall	At-Rest	55
Free to Rotate - Site Wall	Active	35

Fifty and thirty percent of any uniform areal surcharge placed at the top of the wall may be assumed to act as a uniform horizontal pressure over the entire wall for the at-rest and active cases, respectively. As a minimum, we recommend that a traffic surcharge equivalent to 2 feet of soil backfill be assumed as a surcharge for the at-rest condition. For this condition a pressure of 120 psf may be assumed to act as a uniform horizontal pressure over the entire height of the wall, H. Horizontal pressures resulting from foundation or other surcharge loads adjacent to retaining walls can be estimated using the method presented in Figure 10.

For passive resistance on retaining wall foundations, we recommend using an allowable equivalent fluid weight of or 450 psf for undisturbed formational material or 350 pcf for footings poured neat against properly compacted engineered fill. The upper 12 inches of material in areas not protected by adjacent concrete slabs or footings should not be included in design for passive resistance to lateral loads. The allowable coefficient of friction between the bottom of the footing and formational soil or engineered fill can be assumed as 0.30. Both of these values were calculated with safety factor of 1.5. The passive pressure and frictional resistance can be combined to resist lateral loads, provided that the larger value is reduced by 50 percent.

5.5.3 Seismic Design of Retaining Walls

Retaining walls should be designed to resist dynamic earth pressures from earthquake loading. For both cantilever and restrained conditions, walls can be designed using an incremental seismic force of $12H^2$ for the Design Earthquake PGA (in pounds per linear foot of wall length, with H as the wall height in feet), which are additive to the static active earth pressure described above. The incremental seismic force acts at 0.5H above the base of the wall.

Allowable bearing pressure values described in previous sections of this report can be increased by one-third when calculating resistance caused by loads of short duration, such as earthquake loads. Restraining passive pressure and friction values should not be increased by this amount, but a lower factor of safety than is normally applied to static loads could be used. This factor of safety for dynamic load conditions should not be less than 1.2.

5.5.4 Wall Drainage

The above-recommended values do not include lateral pressures due to hydrostatic water pressures generated by infiltrating surface water that may accumulate behind the walls. Therefore, wall backfill materials should be free draining and provisions should be made to collect and remove excess water that may accumulate behind earth retaining structures.

Wall drainage may be provided by free-draining gravel surrounded by non-woven synthetic filter fabric or by prefabricated, synthetic drain panels. In either case, drainage should be collected by collector pipes at the base and directed to a sump, storm drain, weep hole(s), or other suitable location for disposal. The drainage discharge should not be permitted to discharge over soil in a manner that would cause erosion. If utilized, we recommend that drainage gravel consist of durable stone having 100 percent passing the 1-inch sieve and zero percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a permeability of at least 0.02 centimeters per second, a minimum flow rate of 50 gallons per minute per square foot of fabric, and a minimum puncture strength of 50 pounds. The geotextile manufacturer's recommendations should be followed for installation of a drainage fabric system.

5.6 EXTERIOR CONCRETE FLATWORK

All flatwork and exterior concrete should be supported on at least 12 inches of compacted, low to very low expansive engineered fill. The concrete slabs for walkways and sidewalks should have a nominal thickness of 4 inches. The exposed subgrade should be scarified to a depth of 6 inches, uniformly moisture conditioned to between optimum and 3 percent above optimum moisture content, and compacted to at least 95 percent relative compaction as per ASTM D1557.

5.7 PAVEMENT SECTIONS

We are providing preliminary recommendations for potential asphalt parking areas and associated drives. Due to the variability of site soils and for purposes of analysis and design of pavements, we have assumed an R-value for typical subgrade materials in the area. One sample of similar material was collected from the project site on the southwest side of campus and resulted in an R-value of 66. For the purposes of this study, we assumed an R-value of 20 for design of pavement sections. Final pavement sections can be adjusted based on testing of actual soils encountered during construction.

5.7.1 Flexible Pavements

Flexible pavement sections have been evaluated in general accordance with the Caltrans method for flexible pavement design. Traffic indices of 5.0 and 6.0 were used to facilitate the design of parking areas, driveways, and fire access. Recommended flexible pavement sections for these conditions are given in Table 3.

Table 3

Recommended Flexible Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5	3	7
6	4	8.5
8	5	13.5

The flexible pavement should conform to, and be placed in accordance with, current Caltrans Standard Specifications. The aggregate base should comply with Section 26 of the specifications. The aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D1557). The aggregate base can be placed directly on the subgrade provided it has been compacted to 95 percent of the ASTM D1557 maximum dry density at moisture contents of 0 to 3 percent over optimum to a minimum depth of 12 inches. We recommend that all pavement areas conform to the following criteria:

1. All trench backfill, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade.
2. An adequate drainage system should be provided to prevent surface water from saturating the subgrade soil.
3. A periodic maintenance program should be incorporated to include sealing cracks and other measures.

All concrete curbs separating pavement and landscaped areas should extend below the bottom of adjacent aggregate base materials.

5.7.1 Rigid Pavement

Rigid pavements are typically used in truck traffic, parking entrances, or trash enclosure areas (typical traffic index of 6). The recommended minimum rigid pavement section is given in Table 4.

Table 4

Recommended Rigid Pavement Sections

Traffic Index	Concrete Pavement (inches)	Class 2 Aggregate Base (inches)
6	6	10

The recommended minimum rigid pavement section is 7 inches of Portland cement concrete (PCC) over 12 inches of Class 2 Aggregate Base. Concrete for use in rigid pavements should have a flexural strength of at least 600 psi when tested in accordance with ASTM C78.

The concrete pavement should be constructed in an approximate 15-foot square grid system. If a square system is impractical, rectangular panels can be used with the longitudinal distance a maximum of 20 feet.

Longitudinal or transverse control joints should be constructed by hand forming or placing a pre-molded filler such as “zip strips.” Longitudinal or transverse construction joints should be keyed. Expansion joints should be used to isolate fixed objects abutting or within the pavement area. The expansion joint should extend the full depth of the pavement. Joints should run continuously and extend through integral curbs and thickened edges. We recommend that joint layout be adjusted to coincide with the corners of objects and structures.

The recommended pavement sections for both flexible and rigid pavements are based on the following conditions:

1. Utility trench backfill should be properly placed and adequately compacted to provide a stable subgrade. Trench backfill below the 12 inches of pavement soil subgrade should be compacted to a minimum of 90 percent relative compaction (ASTM D 1557).

2. An adequate drainage system should be provided to prevent surface water from saturating the subgrade soil. Pavements should be sloped at least 1/2 percent to provide positive drainage, and not be allowed to pond.

3. A periodic maintenance program should be incorporated to include sealing cracks and other measures.

4. Aggregate base materials and the upper 12 inches of subgrade below aggregate base should be compacted to a minimum of 95 percent of ASTM D 1557 maximum dry density.

5. The finished subgrade should be brought to a firm and unyielding condition at the time aggregate base is laid and compacted.

6. Asphalt concrete pavement and aggregate base materials should conform to Section 02510, Parts 2 and 3 of the Standard Specifications for Construction of Public Works (Greenbook), current edition. Portland cement concrete pavement should conform to Subsections 201-1 and 302-6 of the Greenbook and City of San Diego Pavement Design Standards Schedule "J" SDG-113.

7. Concrete curbs separating pavement from landscaped areas extend at least six inches into the subgrade to reduce movement of moisture into the aggregate base layer. This reduces the risk of pavement failures to subsurface water originating from landscaped areas.

8. Concrete should be cured with a suitable curing compound or be kept continuously moist for a period of at least seven days in general accordance with Greenbook or ACI guidelines.

9. Traffic should be kept off newly placed concrete for at least seven days or until its flexural strength exceeds 600 pounds per square inch.

5.7.2 Crushed Miscellaneous Base

Crushed Miscellaneous Base may be used as a direct substitute for Caltrans Class II base, provided that it conforms to the Standard Specifications for Public Works Construction (200 2.4). Crushed miscellaneous base shall consist of broken and crushed asphalt concrete or Portland cement concrete and may contain crushed aggregate base or other rock. The material shall be free of any detrimental quantity of deleterious material. Material retained on the 4.75mm (No. 4) sieve shall contain no more than 15 percent gravel particles. Minimum required R-value for crushed miscellaneous base is 80.

5.8 PRELIMINARY CORROSIVE SOIL SCREENING

A preliminary corrosive soil screening for the onsite materials was completed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory testing on two representative soil samples obtained during our current subsurface evaluation and review of previous nearby results. Laboratory testing was performed to evaluate soluble chloride, soluble sulfate content, resistivity and pH of soil. Results of the tests are provided in Appendix B.

Caltrans defines a "corrosive site" as one where one or more representative soil and/or water samples contain concentrations of soluble chloride of 0.05 percent (by weight) or greater, soluble sulfate concentrations of 0.2 percent or greater or the pH is 5.5 or less. Based on the laboratory test results, this site is considered "non-corrosive" by the Caltrans definition. The Portland Cement Association (PCA, 1988) defines concrete exposure to sulfate attack as negligible for soil with a water soluble sulfate content of 0.00 to 0.10 percent (by weight), moderate for a sulfate content of 0.10 to 0.20 percent, severe for a sulfate content of 0.2 to 2.00 percent, and very severe for a sulfate content over 2.00 percent. Test results indicate concrete exposure to sulfate attack as moderate for this site.

Upon saturation, the minimum resistivity results of two tests were 1,000 and 1,800 ohm-cm. A previous result of fill soil at BHU-1 indicated a minimum resistivity of 250 ohm-cm. Resistivity values under 500 are considered very corrosive and those between 1,000 and 2,000 ohm-cm are considered moderately corrosive.

We recommend that the corrosion test results be reviewed and evaluated by the project designers considering the proposed improvements and project lifespan requirements. A corrosion engineer can be contacted for detailed evaluation of corrosion potential and corrosion resistant design.

5.9 SITE DRAINAGE AND EROSION CONTROL

Final elevations at the site should be planned so that positive drainage is established around structures. Positive drainage is defined as a slope of 2 percent or more for a distance of 5 feet or more away from structure foundations. Roof gutters and downspouts should be installed on structures. Downspouts should discharge to controlled drainage systems and drainage gradients should be maintained to carry all surface water off the site. Ponding should not occur on the site.

Planters should be built so that water exiting from them will not seep into the foundation areas or beneath slabs and pavement. In any event, the maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to properly sustain the landscaping plants. Should excessive irrigation, waterline breaks, or unusually high rainfall occur, saturated zones and perched groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the foundation or landscaped areas. Potential sources of water, such as water pipes, drains, garden ponds, and the like, should be frequently examined for signs of leakage or damage. Any such leakage or damage should be repaired promptly.

5.10 SIGN AND LIGHT POLE SUPPORT

Proposed sign structures and light standard foundations as columns embedded in earth or embedded in concrete footings in the earth to resist both axial and lateral loads, can be designed in general accordance with Section 1807 of the 2016 California Building Code (CBC). We have conservatively assumed that foundations will be embedded in fill materials with the foundation properties as Class 4 Material as defined by the CBC, Table 1806A.2. We recommend that a lateral soil-bearing pressure of 150 lbs/ft2 per foot of depth below natural grade be used. An allowable soil-bearing pressure of 2,000 lbs/ft2 may be used to support vertical loads. The allowable lateral soil-bearing pressure may be increased by a factor of 2 for short-term lateral loads, as allowed in Section 1806A.3.4 of the 2016 CBC, provided the structures will not be adversely affected by a ½ inch of motion at the ground surface.

5.11 STORMWATER INFILTRATION STUDY

We have evaluated the site in conformance with the City of San Diego 2018 BMP Design Manual. For the purpose of this report, infiltration is defined as the flow of water through the ground surface and percolation is defined as the downward flow of water through the subsurface soil layers. Infiltration may be controlled primarily by factors such as the type and porosity of the surface filtering media, maintenance of these media, surface slope, surface vegetation, and intensity, duration, and type of precipitation. Percolation may be controlled primarily by the soil types and properties such as grain size and density, soil layering, porosity, hydraulic head, and the proximity to the groundwater. Surface drainage and maintenance will largely determine the site's infiltration rate and the amount of water that will infiltrate for any given storm. The percolation rate will depend locally on the soil layering and will be controlled by the finer grained soil layers.

Borehole percolation testing was the selected method for field infiltration testing at the site. Two percolation tests were performed at two different locations of the site. The percolation tests were performed in general accordance with those set forth in California Test 750, "Method for Determining the Percolation Rate of Soils Using a 6-Inch-Diameter-Test Hole". The tests were performed in drilled holes advanced to a depth of 5 feet below existing site grades. The measured percolation rates have been converted to an adjusted short-term infiltration rate based on borehole geometry using the Porchet Method (Ritzema, 1994) and are presented in Table 5. These values are converted to long term design infiltration rates later in this report by using correction factors based on Worksheet D.5-1 of the BMP Design Manual.

Table 5

Summary of Adjusted Infiltration Rates

Boring	Tested Depth from Ground Surface (feet)	Adjusted Short Term Infiltration Rate (inch/hour)	Soil Description
PERC-1	5	0.2	Silty Sand with Gravel
PERC-2	5	2.5	Silty Sand

Note that relatively clean water was used to perform the tests above. However, surface runoff water from the site would likely contain silt, clay, oil and/or other materials that would eventually decrease the percolation rates. The provided field percolation rates in Table 5 do not include reduction factors for long term performance. These values are converted to long term design infiltration rates later in this report by using correction factors based on Worksheet D.5-1 of the BMP Design Manual.

Based on visual soil classifications and laboratory testing of the two soil samples collected during our field exploration at the percolation test locations, subsurface materials mostly consist of silty sand within the depths of the test. Testing performed consisted of sieve analyses from samples obtained at the bottom of the borings. The results are presented on the boring logs and lab results in Appendix A and Appendix B, respectively.

5.11.1 Mitigation Measures

The following bullets present typical considerations (geotechnical and other) for implementation of infiltration systems, along with site specific conditions.

- Presence of fill soils below building footprint. The site is underlain by up to about 10 feet of fill however the proposed building footprint depth will be lower than 10 feet from existing grade.
- Presence of shallow formational material. The site has San Diego Formation approximately 4 to 10 feet in depth from the ground surface of lower parking lot. Water from overlying BMPs would likely perch on the less permeable formation materials and move laterally to the more permeable material in utility trenches or wall backfill.
- Building sites located adjacent to or within landslide hazard areas or hillside grading areas. These sites are not located near landslide hazard areas.
- Sites with initial seasonal high groundwater elevation within 10 feet of the invert of a proposed basin. The sites are not within 10 feet of high groundwater table.
- Site soils with a moderate or high potential for liquefaction. The sites have a low potential for liquefaction.
- Site soils with a moderate or high expansion potential. The majority of observed soils within the infiltration test areas appear to have low expansion potential.
- Sloping sites. The proposed BMP basin sites are generally in flat and/or near gently sloping areas.
- Sites with soil and/or groundwater contamination. According to the California State Water Resources Control Board Geo Tracker Database, the closest site cleanup is located over 0.15 miles to the south at 330 Washington St, San Diego. Contamination has not been identified in the near the project site.

5.11.2 Data Evaluation

The results of the field testing program provide a design infiltration rate based on correction factors contained within Tables D.5-1 and D.5-2 of the 2018 BMP Design Manual, as summarized in the following table.

Table 6

Suitability Assessment Related Considerations for Infiltration Facility Safety Factors*

Consideration	High Concern (3 points)	Medium Concern (2 points)	Low Concern (1 point)
Assessment methods (see explanation below)	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 infiltration area with localized infiltration measurement methods (e.g., infiltrometer) Moderate spatial resolution	Direct measurement with localized (double-ring infiltrometer & borehole) infiltration testing methods at relatively high resolution or Use of extensive test pit infiltration measurement methods (Extensive refers to large excavation, filling with water and monitoring drawdown – ideally 30 to 100 square feet)
Texture Class	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 borings/test pits indicate moderately homogeneous soils	Soil borings/test pits indicate relatively homogeneous soils
Depth to groundwater/ impervious layer	<5 ft below facility bottom	5-15 ft below facility bottom	>15 below facility bottom

Table 7

Design Related Considerations for Infiltration Facility Safety Factors*

Consideration	High Concern (3 Points)	Medium Concern (2 Points)	Low Concern (1 Point)
Level of pretreatment/ expected influent sediment loads	Limited pretreatment using gross solids removal devices only, such as hydrodynamic separators, racks and screens AND tributary area includes landscaped areas, steep slopes, high traffic areas, or any other areas expected to produce high sediment, trash, or debris loads.	Good pretreatment with BMPs that mitigate coarse sediments such as vegetated swales AND influent sediment loads from the tributary area are expected to be moderate (e.g., low traffic, mild slopes, stabilized pervious areas, etc.).	Excellent pretreatment with BMPs that mitigate fine sediments such as bioretention or media filtration OR sedimentation or facility only treats runoff from relatively clean surfaces, such as rooftops/non-sanded road surfaces.
Redundancy / resiliency	No “backup” system is provided; the system design does not allow infiltration rates to be restored relatively easily with maintenance.	The system has a backup pathway for treated water to discharge if clogging occurs or infiltration rates can be restored via maintenance.	The system has a backup pathway for treated water to discharge if clogging occurs and infiltration rates can be relatively easily restored via maintenance.
Compaction during construction	Construction of facility on a compacted site or increased probability of unintended/ indirect compaction.	Medium probability of unintended/ indirect compaction.	Equipment traffic is effectively restricted from infiltration areas during construction and there is low probability of unintended/ indirect compaction.

*As presented in Table D.5-2 in Appendix D on page D-17 of BMP Design Manual

5.11.3 Design Infiltration Rates

Based on our evaluation of the percolation test data discussed in a preceding section of this report, the soils encountered exhibit infiltration rates for *short-term, non-factored infiltration rates between **0.2** and **2.5** inch/hour*. The long-term design infiltration rate was calculated by using the following correction factors based on Worksheet D.5-1 of the BMP Design Manual. The completed worksheets are presented in Appendix D. Design infiltration rates have been estimated for PERC-1 through PERC-2 and the values are presented in Table 8. However, feasibility of infiltration for preliminary purposes a factor of safety of 2.0 must be used with the non-factored infiltration rates. For preliminary purposes the design infiltration rates are **0.1** and **1.25** inch/hour.

Table 8

Design Infiltration Rates*

Boring	Safety Factor	Long Term Design Infiltration Rate (Inch/hour)
PERC-1	2.0	0.11
PERC-2	2.5	1.00

* Worksheet D.5-1 of PERC-2 is attached in Appendix D. Note for PERC-1 it was considered "partial infiltration" and based on Section D.5.4 BMP Manual a Factor of Safety of 2.0 must be used for partial infiltration. Note for preliminary feasibility of infiltration a factor of 2.0 must be used.

5.11.4 Recommendations and Conclusions

Based on the design infiltration rates and the completed Geotechnical and Groundwater Investigation Requirement Worksheet C.4-1 contained in the BMP Design Manual, we classify the site as a feasibility screening category of "Partial Infiltration". The completed C.4-1 worksheets for each BMP location proposed at the site are included in Appendix D of this report.

Based on the field percolation testing, geotechnical observations, laboratory data, and completion of the BMP Manual Worksheets, it is our opinion that the project site is categorized as not suitable for full infiltration. The site is underlain by silty and clayey sand fill soils over very dense and variably depth of San Diego Formation that overlay the Pomerado Conglomerate Formation. In our opinion, the underlying formation material has low void ratio and cementation therefore low permeability characteristics. We recommend that basins be located as far away from proposed building foundations and utility trenches as feasible in order to minimize the potential effects of lateral migration of water.

6 ADDITIONAL STUDIES

The recommendations within this report were based on progress drawings and should be updated if final design differs from that assumed herein. The review of plans and specifications, and the observation and testing by Kleinfelder of earthwork related construction activities, are an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client will be assuming our responsibility for any potential claims that may arise during or after construction. The required tests, observations, and consultation by Kleinfelder during construction include, but is not limited to:

- A review of plans and specifications;
- Observation of site clearing;
- Construction observation and density testing of fill material placement, trench backfill, subgrade preparation, and aggregate base for pavements; and
- Observation of foundation excavations and foundation construction.

7 LIMITATIONS

This report has been prepared for the exclusive use of Scripps Health and their consultants for specific application to the subject project. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No warranty, express or implied, is made.

The scope of services was limited to the field exploration program described in this report. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions presented herein are based on field explorations, laboratory testing, engineering analyses and professional judgement.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues addressed in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, laboratory tests, and our understanding of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, or locations of the improvements, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid until the

changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, ground improvement, preparation of foundations, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on site and off site), or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

Kleinfelder will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials. Additional important information about this report is presented in the attached Geotechnical Business Council insert in Appendix G.

8 REFERENCES

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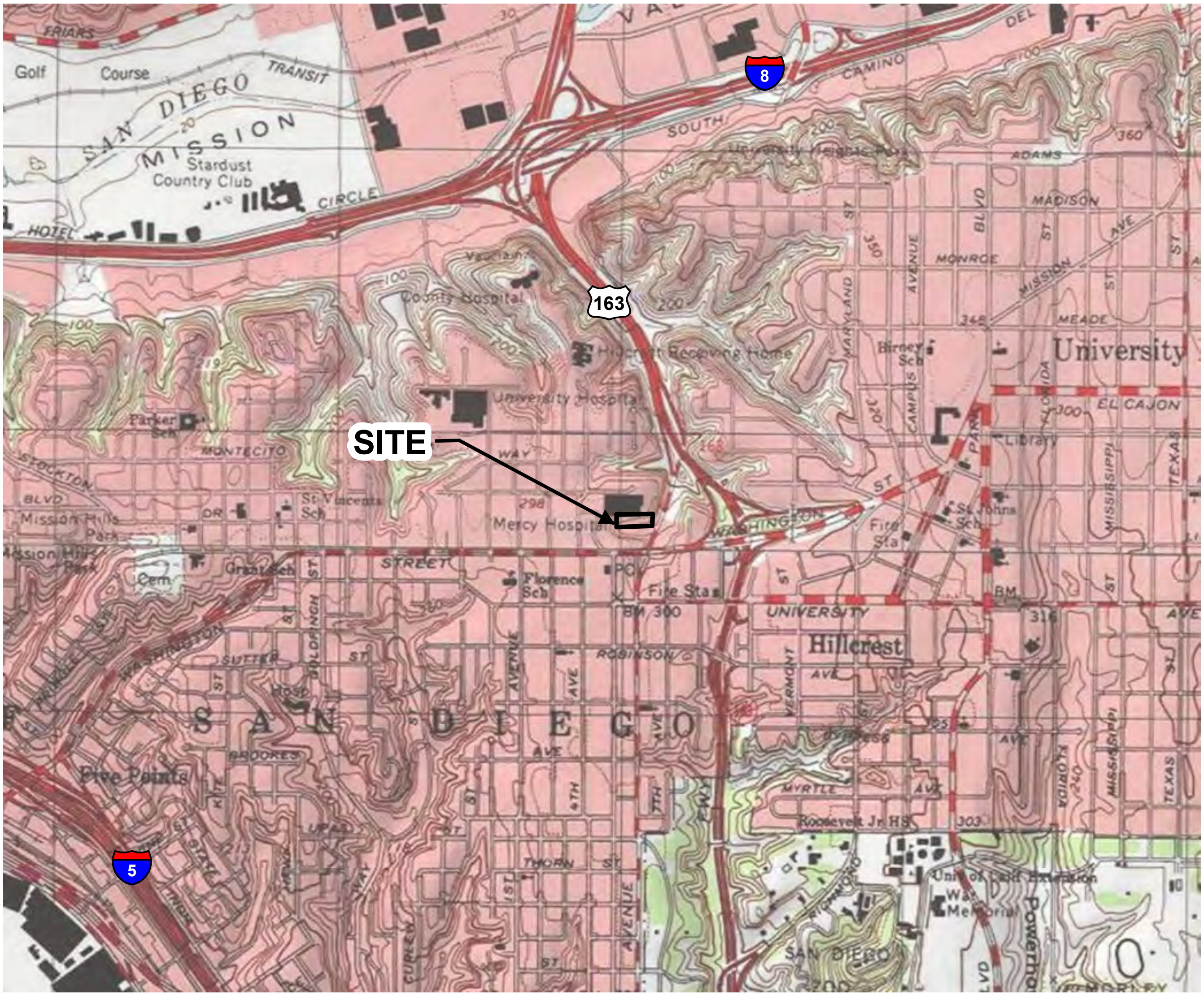
Kleinfelder, “Supplemental Geotechnical Investigation, Proposed Emergency Department Expansion, Scripps Mercy Hospital, 4077 5th Avenue, San Diego California,” dated March 6, 2007.

Kleinfelder, “Draft Report Geologic and Geotechnical Engineering Evaluation, Scripps Mercy Hospital Expansion, BHU Elevator Tower Replacement, 4077 5th Avenue, San Diego, California,” dated March 27, 2007.

FIGURES



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PROJECT NO. 20183768	VICINITY MAP	FIGURE
DRAWN: 4/13/2018		
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CHECKED BY: ST	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	
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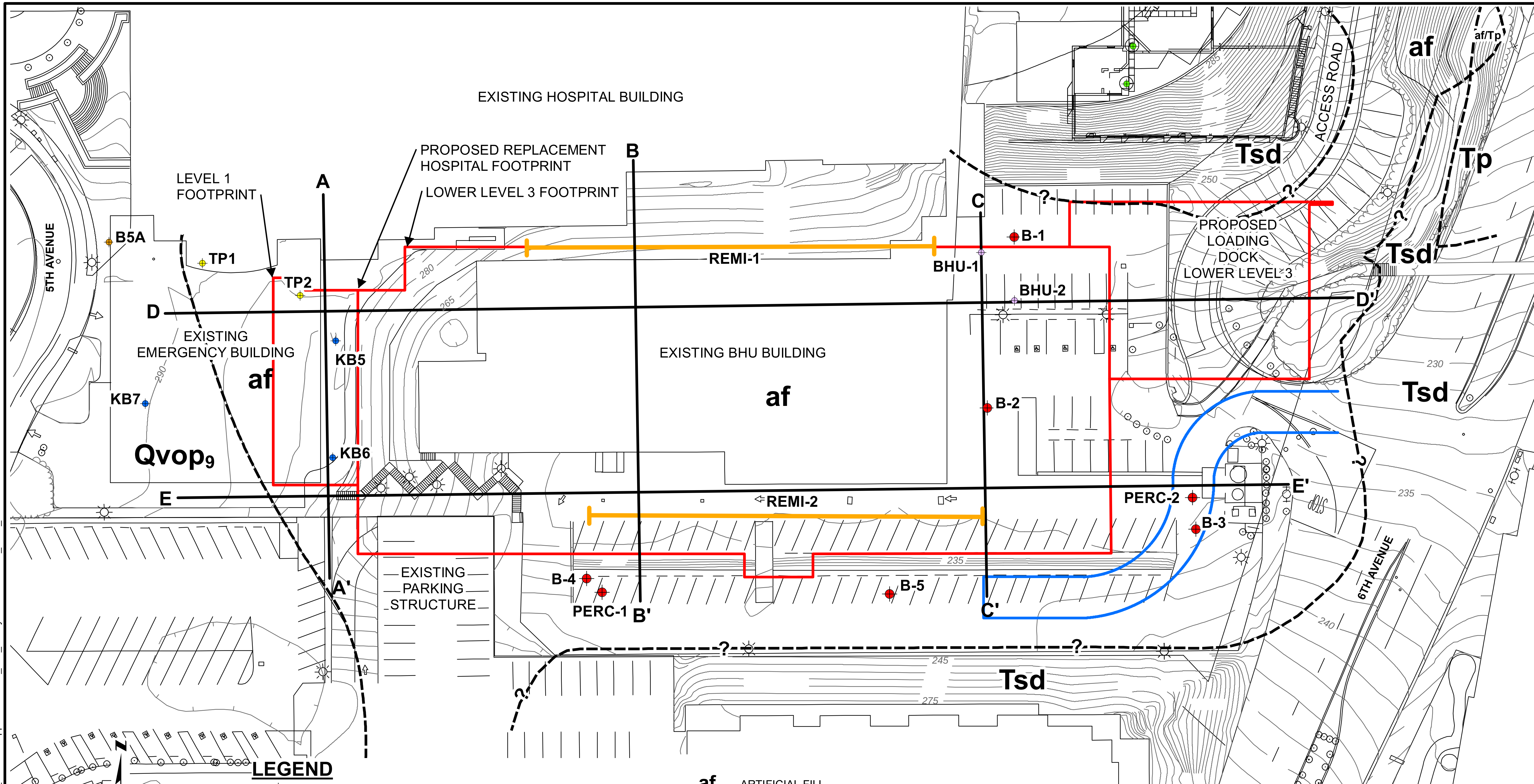

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LEGEND

- APPROXIMATE BORING LOCATION, KLEINFELDER 2018
- APPROXIMATE BORING LOCATION, KLEINFELDER 2006
- APPROXIMATE BORING LOCATION, KLEINFELDER 2005
- APPROXIMATE BORING LOCATION, BENTON 1994
- APPROXIMATE BORING LOCATION, WOODWARD CLYDE 1986
- APPROXIMATE BORING LOCATION, BENTON 1977
- APPROXIMATE LOCATION OF CROSS SECTION, KLEINFELDER 2018
- APPROXIMATE LOCATION OF ReMi LINE
- PROPOSED REPLACEMENT HOSPITAL FOOTPRINT
- PROPOSED FIRE LANE

af	ARTIFICIAL FILL
$Qvop_9$	VERY OLD PARALIC DEPOSITS
Tsd	SAN DIEGO FORMATION
Tp	POMERADO CONGLOMERATE
- - -	GEOLOGIC CONTACT

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PROJECT NO. 20183768	EXPLORATION MAP		FIGURE
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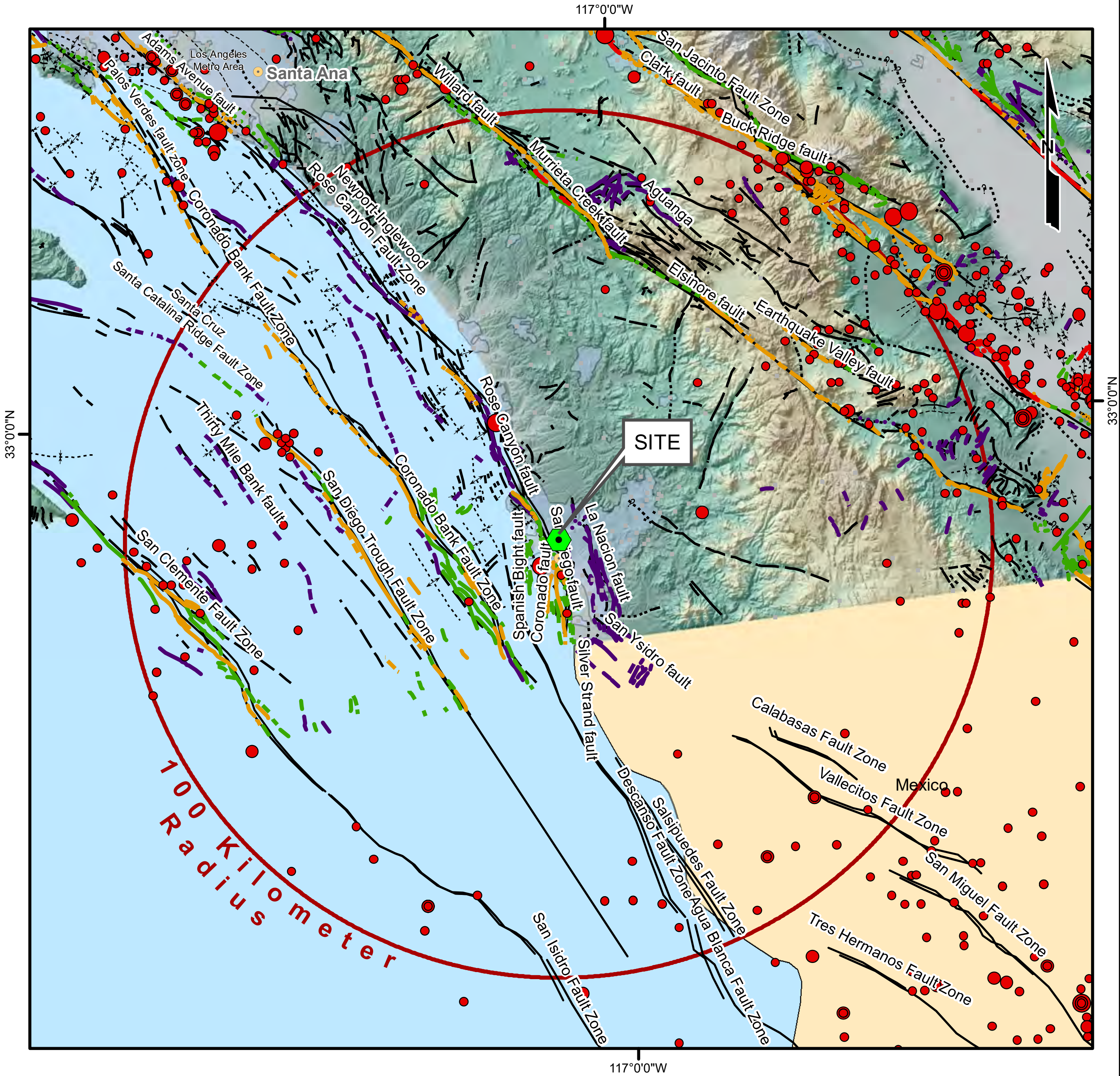

Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2009)

Historic displacement (< 200 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Holocene displacement (< 11,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Quaternary displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

ANSS Earthquakes

Magnitude
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7.0 - 7.9
- 8.0 - 8.9

35 17.5 0 35
Kilometers

\\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20183768_001_Mercy\mxd\20183768_EQ_Merc.mxd

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REGIONAL FAULT MAP
AND EARTHQUAKE EPICENTERS
(1800 - JUNE 2016)

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
3

LEGEND

- af - ARTIFICIAL FILL
- Qya - YOUNG ALLUVIAL FAN DEPOSITS
- Qvop - VERY OLD PARALIC DEPOSITS, UNITS 6, 8, 9, AND 10
- Tsd - SAN DIEGO FORMATION
- Tmv - MISSION VALLEY FORMATION
- Tp - POMERADO CONGLOMERATE

SOURCE:
GEOLOGIC MAP OF THE SAN DIEGO 30'X60' QUADRANGLE, CALIFORNIA, KENNEDY AND TAN 2008.

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PROJECT NO.	20183768
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DRAWN BY:	JP
CHECKED BY:	ST
FILE NAME:	20183768_Geo_Merc.mxd

REGIONAL GEOLOGIC MAP

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
4

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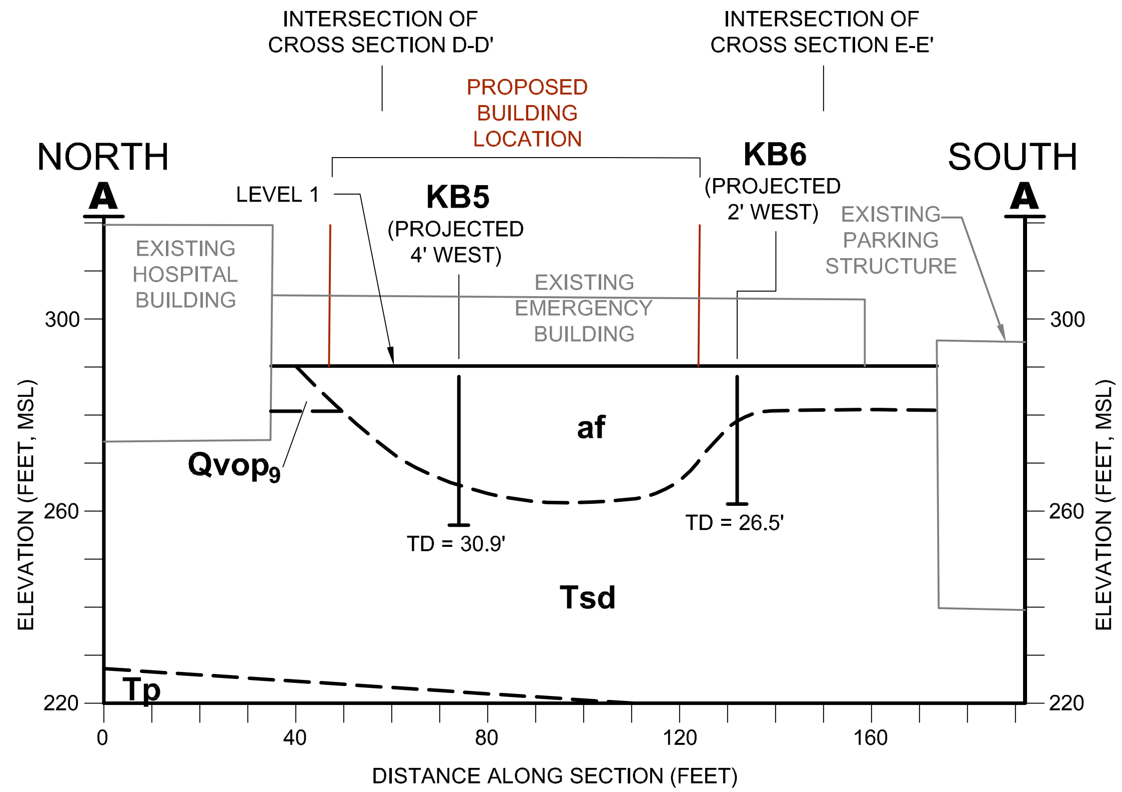

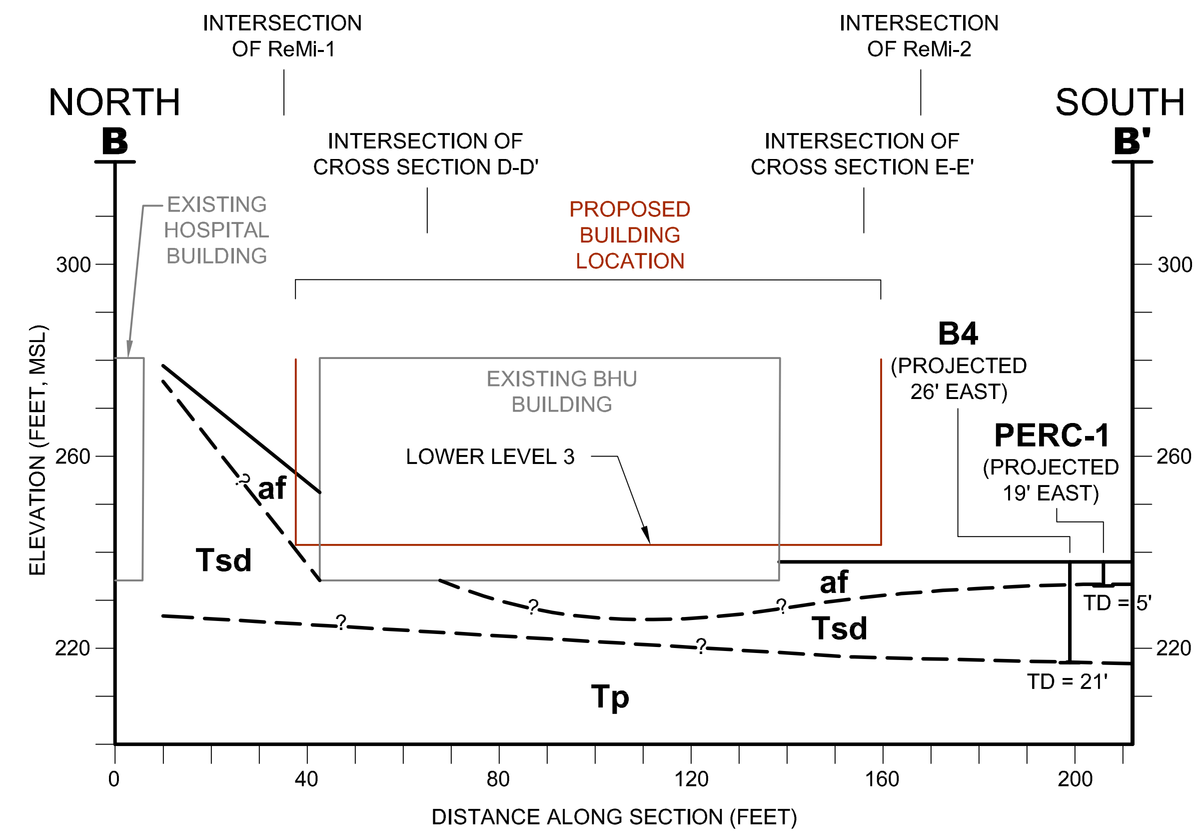

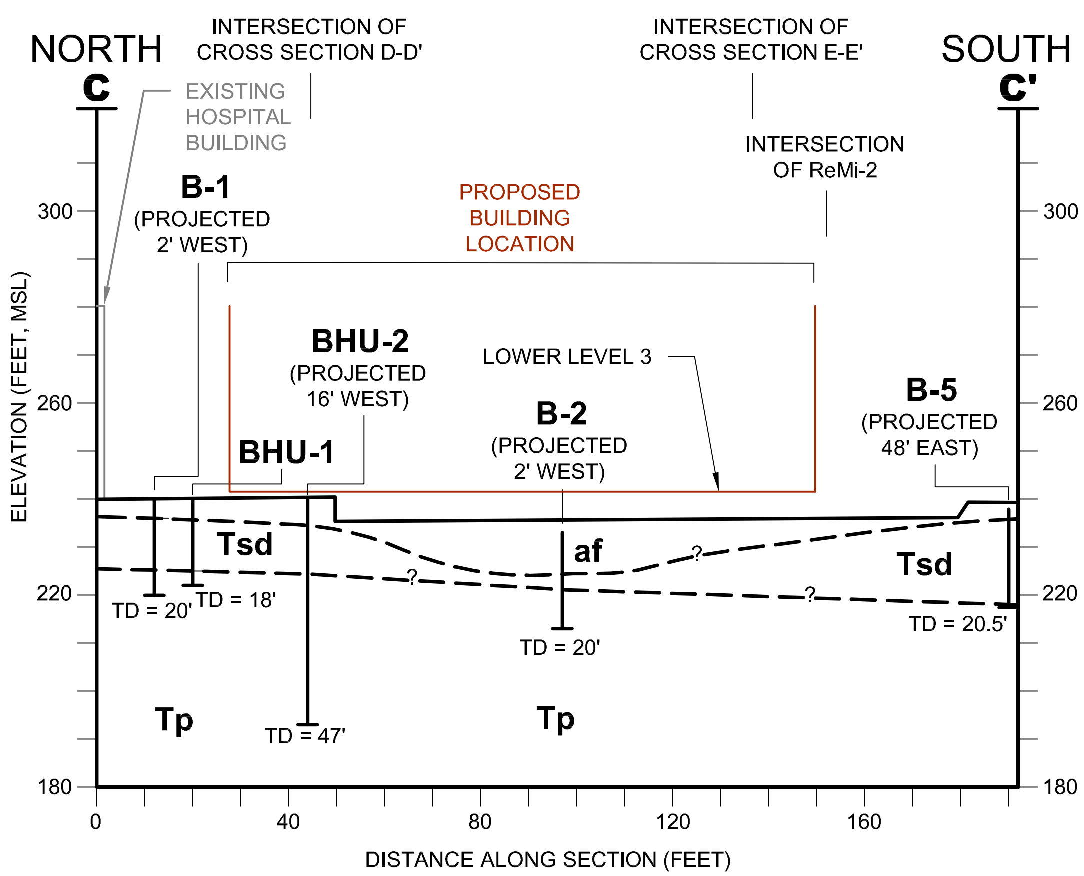

LEGEND

af	ARTIFICIAL FILL
$Qvop_9$	VERY OLD PARALIC DEPOSITS
Tsd	SANTIAGO FORMATION
Tp	POMERADO CONGLOMERATE
— — —	GEOLOGIC CONTACT

B-2
(PROJECTED 5' EAST)
TD = 26.5'
APPROXIMATE LOCATION OF BORING, SHOWING ID, PROJECTION AND TOTAL DEPTH

40 20 0 40
APPROXIMATE HORIZONTAL AND VERTICAL SCALE (feet)

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PROJECT NO.	20183768
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DRAWN BY:	JP
CHECKED BY:	KC
FILE NAME:	20183768_SectMer.dwg

GEOLOGIC CROSS SECTIONS A-A', B-B', AND C-C'

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
5

CAD FILE: \\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20183768_001_Mercy\mxd\ LAYOUT: A-A'-B-B' PLOTTED: 22 Apr 2019, 4:36pm, JPatay
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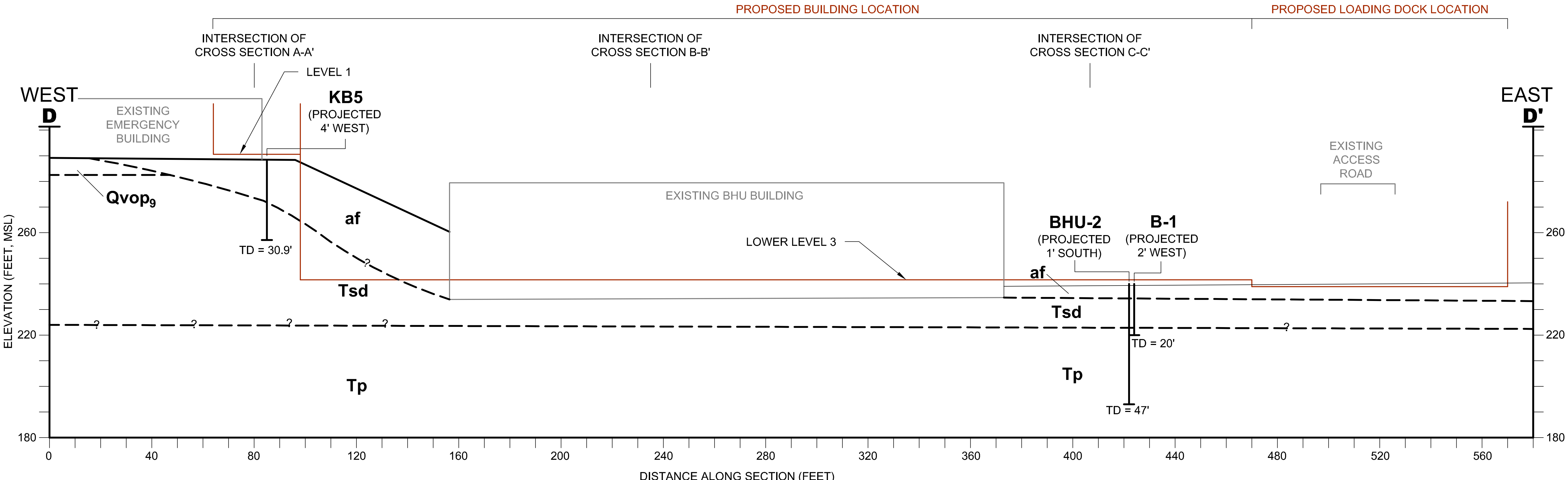
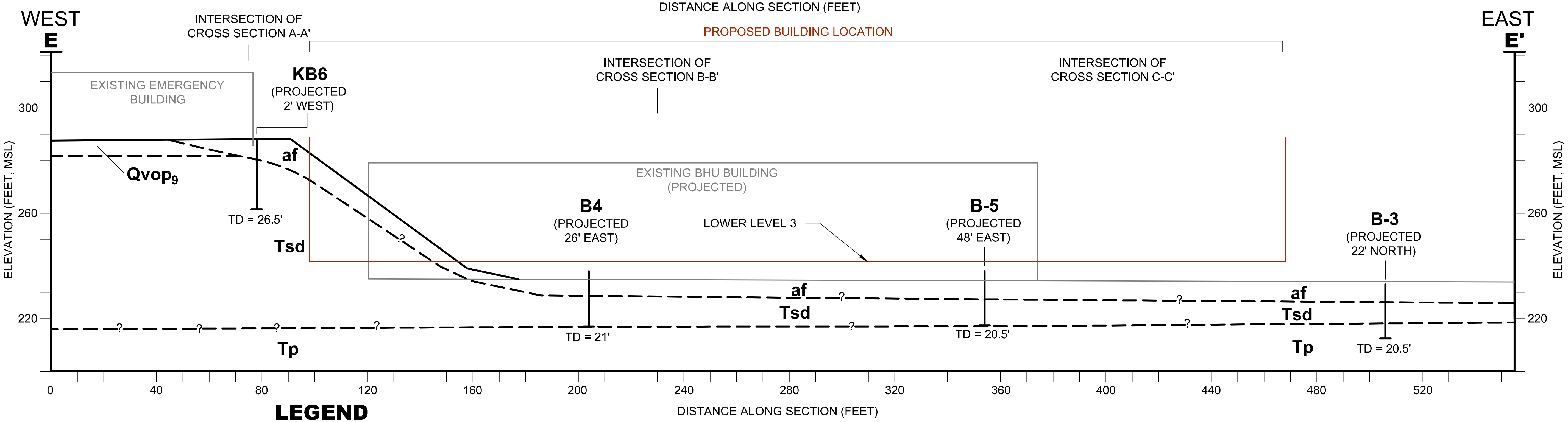
ATTACHED XREFS:





LEGEND

af	ARTIFICIAL FILL
Tsd	SANTIAGO FORMATION
Tp	POMERADO CONGLOMERATE

B-2 (PROJECTED 5' EAST) TD = 26.5' — APPROXIMATE LOCATION OF BORING, SHOWING ID, PROJECTION AND TOTAL DEPTH

— — — GEOLOGIC CONTACT

40 20 0 40

APPROXIMATE HORIZONTAL AND VERTICAL SCALE (feet)

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PROJECT NO.	20183768
DRAWN:	4/22/2019
DRAWN BY:	JP
CHECKED BY:	KC
FILE NAME:	20183768_SectMer.dwg

GEOLOGIC CROSS SECTIONS D-D' AND E-E'

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE **6**

CAD FILE: \\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20183768_001_Mercy\mxd\ LAYOUT: D-D E-E 22 Apr 2019, 4:34pm, JPatay

ATTACHED IMAGES:
ATTACHED XREFS:

Google Earth 2018

DESIGNED BY: S.Tena	DATE 4/12/18	Photograph 1	SCALE NOT TO SCALE
DRAWN BY: S.Tena	4/12/18		
CHECKED BY: K. Crennan	4/12/18	Replacement Hospital Scripps Mercy Hospital 4077 5th Avenue San Diego, California	FIGURE NO. 7

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N

Surface on a Cut Slope near 6th Avenue and Lewis Street

Google Earth 2018

DESIGNED BY: S.Tena	DATE 4/12/18	Photograph 2	SCALE NOT TO SCALE
DRAWN BY: S.Tena	4/12/18		
CHECKED BY: K. Crennan	4/12/18	Replacement Hospital Scripps Mercy Hospital 4077 5th Avenue San Diego, California	FIGURE NO. 8

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N

Surface on a Cut Slope near 6th Avenue and Lewis Street

Google Earth 2018

KLEINFELDER Bright People. Right Solutions.	N	DESIGNED BY: S.Tena	DATE 4/12/18	Photograph 3	SCALE NOT TO SCALE
		DRAWN BY: S.Tena	4/12/18		
		CHECKED BY: K. Crennan	4/12/18	Replacement Hospital Scripps Mercy Hospital 4077 5th Avenue San Diego, California	FIGURE NO. 9

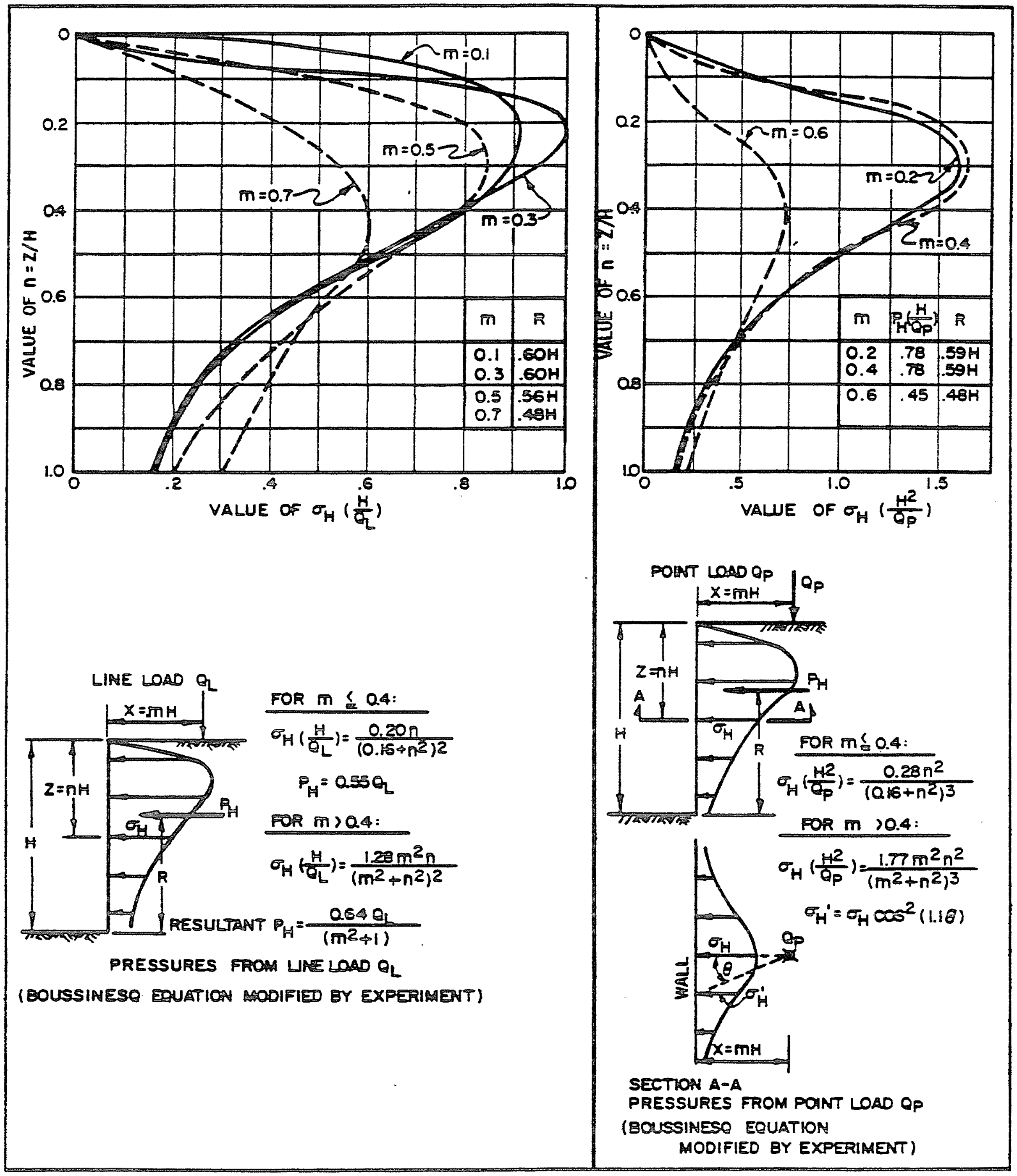
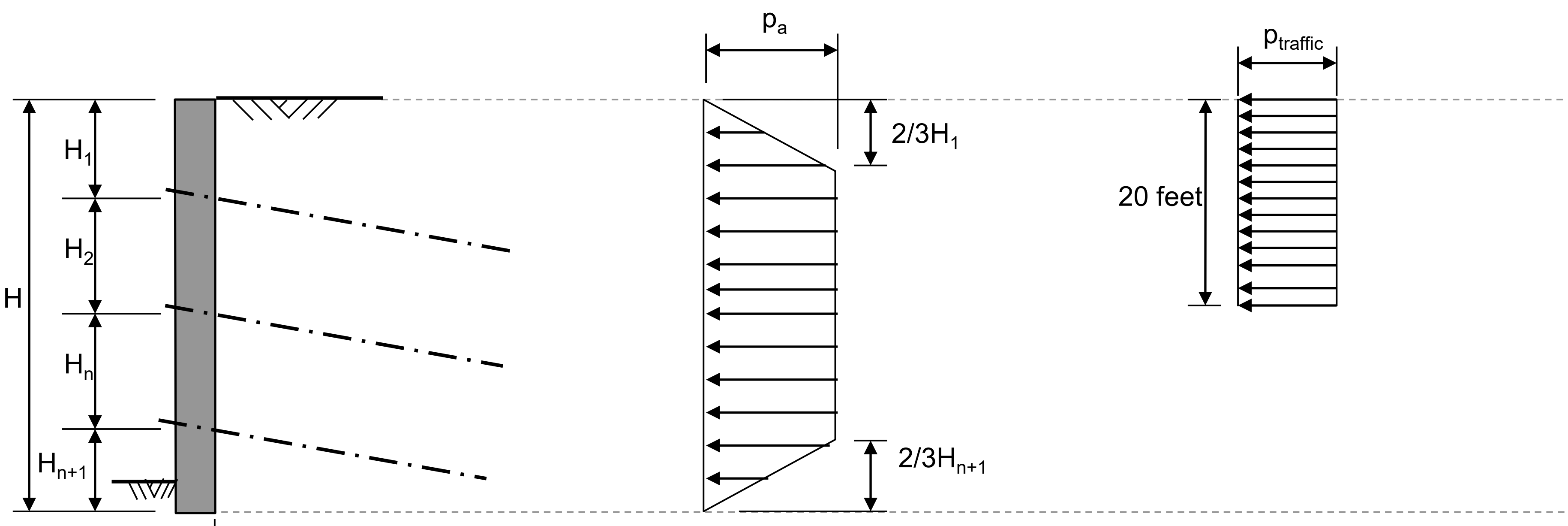


FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

KLEINFELDER *Bright People. Right Solutions.*	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	**FIGURE** **10**
PROJECT NO. **20183768.001A**	**Horizontal Pressures on Rigid Wall from Surface Load** (US Navy, 1986)	



RETAINING WALL SKETCH
(Geometry and pressure diagrams not to scale)

ACTIVE EARTH PRES.
p_a=25*H in psf, H in feet (Northside)
p_a=25*H in psf, H in feet (Southside)
p_a=30*H in psf, H in feet (Westside)
p_a=25*H in psf, H in feet (Eastside)

TRAFFIC OR UNIFORM SURCHARGE EARTH PRES.
$p_{traffic}$=120 psf

Notes:

1) These earth pressures should be used in conjunction with the recommendations in the text of the Project Geotechnical Report.

2) Design GW elevation assumed below bottom of excavation during temporary shoring.

3) For passive resistance on retaining walls, we recommend using an equivalent fluid weight of 350 pcf for engineered fill and 450 pcf for San Diego Formation and Pomerado Formation.

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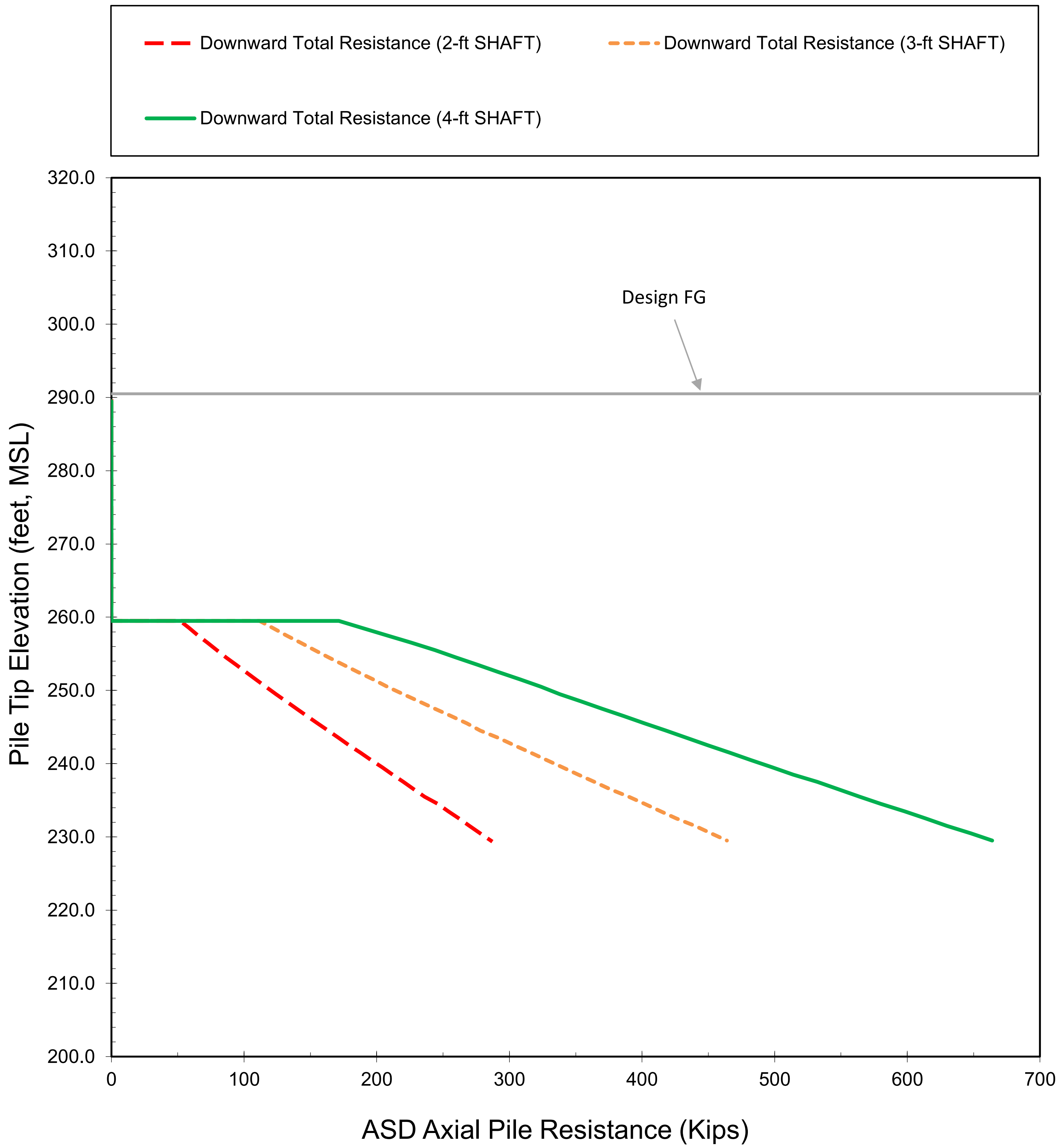
PROJECT NO.	20183768.001A
DATE:	May 2018
BY:	ST
CHECKED:	KC
File Name:	TiebackPressures.ppt

LATERAL EARTH PRESSSURES FOR TIEBACK SHORING

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
11



NOTES:

1. The resistance curves represent ASD axial resistance for single CIDH shafts.
2. The resistance curves include skin friction and end bearing.
3. The original grade elevation is +290.5 feet MSL; the finish grade elevation is +290.5 feet MSL.
4. Assume no groundwater.
5. The pile head elevation is +289.5 feet MSL.
6. An ASD resistance factor of 2.5 is applied to the side resistance and 3.0 is applied to the end bearing.

KLEINFELDER Bright People. Right Solutions.	Project Number:	20183768.001A	FACTORED AXIAL PILE RESISTANCE CIDH (ASD DESIGN)	FIGURE NO.
	Date:	4/19/2019		
	Entry By:	ST	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	12
	Checked By:	KC		
	Date:	4/24/2019		

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APPENDIX A

Boring Logs

APPENDIX A
BORING LOGS

The field investigation for the replacement hospital building was conducted April 4, 2018. Seven borings B-1 through B-5 and PERC-1 through PERC-2 were performed for the investigation of the proposed Hospital Building. We have also used information from previous borings near the project site in our analysis and have therefore included logs of these borings as part of this report in Appendix A.1.

The Unified Soil Classification System (USCS) was used for our soil descriptions, the chart and a Boring Log Legend are presented as Figures A-1 and A-2, respectively. The Boring Logs for the proposed replacement Hospital Building are presented as Figure A-3 through A-9. The Logs of Borings describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the boring number, drilling date, and the initials of the logger and name of drilling subcontractor. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and intact samples of representative earth materials were obtained from the borings.

Seven borings were drilled at the site on April 4, 2018. The borings were advanced to termination or refusal depths ranging from 5 feet to 21 feet, using a CME-85 drill rig. The truck mounted drill rig was operated by ABC LIOVIN and was equipped with 8-inch O.D. hollow stem augers.

The approximate locations of the borings are shown on the Exploration Map, Figure 2.

In-place soil samples were obtained at the test boring locations using a California penetration sampler or a Standard penetration sampler (SPT) driven a total of 18-inches (or until practical refusal) into the undisturbed soil at the specified sample depth. The sampler was driven using a 140-pound auto-hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count (or N-value) and is recorded on the Boring Logs. Please note that these blow counts have not been adjusted for the effects of overburden pressure, input driving energy, rod length, sampler correction, or boring diameter correction. In addition, to the California and SPT samples, we also obtained bulk samples from the drill cuttings.

An engineer from our office supervised the field operations and logged the borings according to the Unified Soil Classification System (USCS). The boundaries shown between soil types on the logs and cross sections are approximate as the transition between different soil layers may be gradual. Therefore, variations in the subsurface profile should be anticipated throughout the site. The samples retrieved from the borings were sealed, labeled, and transported to our laboratory for further evaluation and testing. Upon completion of the drilling operations, the boreholes were backfilled as required by the County of San Diego Department of Environmental Health (DEH).

PLOTTED: 06/06/2018 04:51 PM BY: STena
gINT FILE: Klf_gint_master_2017 PROJECT NUMBER: 20183768.001A OFFICE FILTER: SAN DIEGO
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [LEGEND 1 (GRAPHICS KEY) USCS_SOU_CAL]

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- HQ CORE SAMPLE (2.500 in. (63.5 mm.) core diameter)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING
- SONIC CONTINUOUS SAMPLER

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS
WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/ or 1>Cc >3	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu ≥4 and 1≤ Cc≤3	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/ or 1>Cc >3	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/ or 1>Cc >3	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu ≥6 and 1≤ Cc≤3	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/ or 1>Cc >3	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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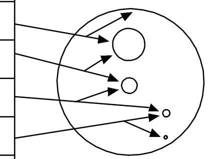
PROJECT NO.:	20183768	GRAPHICS KEY	FIGURE
DRAWN BY:	ST		
CHECKED BY:	SHR	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	A-1
DATE:	4/6/2018		
REVISED:	6/6/2018		

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

Term of Use	AMOUNT	
	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

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PROJECT NO.: 20183768	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: ST		
CHECKED BY: SHR	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	A-2
DATE: 4/6/2018		
REVISED: 6/6/2018		

Date Begin - End:	4/04/2018	**Drilling Company:**	ABC LIOVIN		**BORING LOG B-1**
Logged By:	S.Tena	**Drill Crew:**	Raul & Juan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-85	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 240.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		ASPHALT: (3 inches)												
		BASE COURSE: (4 inches)												
		ARTIFICIAL FILL (af) **Silty SAND (SM)**: fine-grained, non-plastic to low plasticity, light gray (5Y 7/2), moist, subrounded gravel (1")	S1			48"		10.8						**Expansion Index=** 1
235	5	**SAN DIEGO FORMATION (Tsd)** **Poorly Graded SAND with Silt (SP-SM)**: medium-grained, non-plastic, yellow (10YR 7/6), moist, very dense	S2		BC=15 27 47	18"	SP-SM	4.7	108.1	100	5.9			Hard drilling at 7 feet.
230	10	**Silty SAND (SM)**: fine-grained, light olive gray (5Y 6/2), moist, very dense, weakly cemented, iron oxide staining	S3		BC=11 25 37	18"		8.4						
225	15	**Sandy SILT (ML)**: light olive gray (5Y 6/2), moist	S4		BC=2 4 18	18"								**pH=** 9.1 **Resistivity=** 1800 ohm-cm **Sulfates=** 84 ppm **Chlorides=** 53 ppm **DIRECT SHEAR TEST** Cohesion= 300 psf Friction Angle= 37° Hard drilling and shattering at 18 feet.
		Lean CLAY with Sand (CL): low plasticity, pale brown (2.5Y 7/4), moist	S5					27.6	94.7	100	70			
		POMERADO CONGLOMERATE (Tp) **Clayey SAND with Gravel (SC)**												
220	20					NR								
215	25													

The boring was terminated because of practical auger refusal (↑) at approximately 20 ft. below ground surface. The boring was backfilled with bentonite on April 04, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by ALTA SURVEY.

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PROJECT NO.: 20183768	BORING LOG B-1	FIGURE
DRAWN BY: ST		A-3
CHECKED BY: SHR	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	
DATE: 4/6/2018		
REVISED: 6/6/2018		PAGE: 1 of 1

Date Begin - End:	4/04/2018	**Drilling Company:**	ABC LIOVIN		**BORING LOG B-2**
Logged By:	S.Tena	**Drill Crew:**	Raul & Juan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-85	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

Approximate Ground Surface Elevation (ft.): 233.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			FIELD EXPLORATION						LABORATORY RESULTS						
	0		**ASPHALT**: (3 inches)												
			BASE COURSE: (3 inches)												
			ARTIFICIAL FILL (af) **Clayey SAND (SC)**: medium-grained, non-plastic, grayish brown (10YR 5/2) to brown (10YR 5/3), moist, subrounded gravel (1"), brick fragments	S1			48"		7.3						**REMOLDED DIRECT SHEAR TEST** Cohesion: 600 psf Friction Angle: 29° **pH=** 8.0 **Resistivity=** 1000 ohm-cm **Sulfates=** 1320 ppm **Chlorides=** 11 ppm **ASTM D1557 Method A=** Max. Dry Unit Wt.: 127.6 pcf Opt. Water Content: 9.3%
230															
	5		brown (10YR 5/3), very loose	S2		BC=4 2 2	18"		6.7						
225															
	10		**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: brown (10YR 4/3) to light olive brown (2.5Y 5/3), moist, very dense	S3		BC=9 22 50/2"	8"		7.6		95	38	22	3	
220			**POMERADO CONGLOMERATE (Tp)** **Clayey SAND with Gravel (SC)** -subrounded gravel on the soil cuttings												
	15			S4		BC=50/2"	NR								Hard drilling at 18 feet. Shattering during drilling at 20 feet. Attempted a second location approximately 7 feet East from original location, encountered refusal at 17 feet.
215			-angular gravel (1") on the soil cuttings.												
	20														
210															
	25														
205															

The boring was terminated because of practical auger refusal (↑) at approximately 20 ft. below ground surface. The boring was backfilled with bentonite on April 04, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by ALTA SURVEY.

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PROJECT NO.: 20183768	BORING LOG B-2		FIGURE
DRAWN BY: ST			
CHECKED BY: SHR	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA		A-4
DATE: 4/6/2018			
REVISED: 6/6/2018			PAGE: 1 of 1

BORING LOG B-3

Date Begin - End:	4/04/2018	**Drilling Company:**	ABC LIOVIN
Logged By:	S.Tena	**Drill Crew:**	Raul & Juan
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-85
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

Approximate Ground Surface Elevation (ft.): 233.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		ASPHALT: (3 inches)											
		BASE COURSE: (4 inches)											
		ARTIFICIAL FILL (af) **Silty SAND with Gravel (SM)**: medium-grained, non-plastic, yellowish brown (10YR 5/6), moist, subrounded gravel (3")	S1		48"		3.9		54	16			
230													
	5	**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: fine-grained, non-plastic, light gray (2.5Y 7/2) to olive brown (2.5Y 4/3), moist, dense, iron oxide staining	S2	BC=20 22 14	18"		7.4	100.8					
225													
	10	pale brown (2.5Y 7/4), very dense, weakly cemented	S3	BC=7 16 27	18"		18.8						
220													
	15	**POMERADO CONGLOMERATE (Tp)** **Clayey SAND with Gravel (SC)**: fine to medium-grained, yellowish brown (10YR 5/8), subrounded gravel (2"), cobble fragments high plasticity	S4	BC=12 50/5"	11"		10.2	98.3			58	36	
215													Hard drilling at 20 feet. Attempted a second location approximately 5 feet North from original location, encountered refusal at 16 feet.
	20	white (2.5Y 8/1) with light olive brown (2.5Y 5/6)	S5	BC=50/6"	6"		6.9						
210													
	25												
205													

The boring was terminated because of practical auger refusal (↑) at approximately 20.5 ft. below ground surface. The boring was backfilled with bentonite on April 04, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by ALTA SURVEY.

KLEINFELDER
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PROJECT NO.: 20183768	BORING LOG B-3	FIGURE
DRAWN BY: ST		
CHECKED BY: SHR	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	A-5
DATE: 4/6/2018		
REVISED: 6/6/2018		PAGE: 1 of 1

PLOTTED: 06/06/2018 04:48 PM BY: STena
gINT FILE: Klf_gint_master_2017 PROJECT NUMBER: 20183768.001A OFFICE FILTER: SAN DIEGO
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [_KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End:	4/04/2018	**Drilling Company:**	ABC LIOVIN		**BORING LOG B-4**
Logged By:	S.Tena	**Drill Crew:**	Raul & Juan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-85	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 238.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0	(3 inches)												
		(3 inches)	S1			42"		10.3		82	34			**R-Value=** 66
235		**ARIFICIAL FILL (af)** **Silty SAND with Gravel (SM)**: fine-grained, non-plastic, light olive gray (5Y 6/2), moist light yellowish brown (2.5Y 6/4)												
	5	**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: fine-grained, non-plastic, light gray (5Y 7/2), moist, very dense, weakly cemented, iron oxide staining	S2		BC=17 36 50/5"	17"		6.9						
230														
	10		S3		BC=7 30 40	18"		9.3	94.8					
225														
	15	light olive gray (5Y 6/2), dense, increase in moisture	S4		BC=10 16 13	18"		21.5						**pH=** 9.1 **Resistivity=** 1800 ohm-cm **Sulfates=** 84 ppm **Chlorides=** 53 ppm
220														
	20	**POMERADO CONGLOMERATE (Tp)** **Silty SAND (SM)**: fine to medium-grained, light gray (5Y 7/2) with brownish yellow (10YR 6/8), subrounded gravel (2"), some Clay content	S5		BC=7 50/5"	6"		10.9	122.9					Hard drilling at 20 feet.
215														
	25													
210														

The boring was terminated because of practical auger refusal (↑) at approximately 21 ft. below ground surface. The boring was backfilled with bentonite on April 04, 2018.

GROUNDWATER LEVEL INFORMATION:
Perched groundwater was observed at approximately 21 ft. below ground surface at the end of drilling.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by ALTA SURVEY.

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20183768	BORING LOG B-4	FIGURE
DRAWN BY: ST		
CHECKED BY: SHR	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	A-6
DATE: 4/6/2018		
REVISED: 6/6/2018		PAGE: 1 of 1

Date Begin - End:	4/04/2018	**Drilling Company:**	ABC LIOVIN		**BORING LOG B-5**
Logged By:	S.Tena	**Drill Crew:**	Raul & Juan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-85	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 238.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		ASPHALT: (3 inches)												
		BASE COURSE: (4 inches)												
235		**ARTIFICIAL FILL (af)** **Silty SAND (SM)**: fine-grained, non-plastic to low plasticity, light olive brown (2.5Y 5/3), moist, wood debris	S1			48"		8.4						**Expansion Index=** 0
	5	**SAN DIEGO FORMATION (Tsd)** **Poorly Graded SAND with Silt (SP-SM)**: medium-grained, non-plastic, brownish yellow (10YR 6/6), moist, very dense	S2		BC=17 26 48	18"	SP-SM	4.7	110.2	93	6.4			
230	10		S3		BC=9 18 27	18"		7.7						
225		**Silty SAND (SM)**: fine-grained, non-plastic, light gray (5Y 7/2), moist, very dense												
	15	**Sandy SILT (ML)**: light olive gray (5Y 6/2) to light yellowish brown (2.5Y 6/4), iron oxide staining	S4		BC=6 18 29	18"				100	63			**DIRECT SHEAR TEST** Cohesion: 550 psf Friction Angle: 34°
220	20	-subrounded gravel (1.5") in the soil cuttings	S5		BC=50/4"	NR								Hard drilling at 20 feet.
215	25													
210														

The boring was terminated because of practical auger refusal (↑) at approximately 20.5 ft. below ground surface. The boring was backfilled with bentonite on April 04, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by ALTA SURVEY.

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PROJECT NO.: 20183768	BORING LOG B-5	FIGURE
DRAWN BY: ST		A-7
CHECKED BY: SHR	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	
DATE: 4/6/2018		
REVISED: 6/6/2018		PAGE: 1 of 1

Date Begin - End:	4/04/2018	Drilling Company:	ABC LIOVIN		**BORING LOG PERC-1**
Logged By:	S.Tena	**Drill Crew:**	Raul & Juan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-85	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

Approximate Elevation (feet)	Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description (Approximate Ground Surface Elevation (ft.): 238.00; Surface Condition: Asphalt)	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			ASPHALT: (3 inches)												
			BASE COURSE: (3 inches)												
235			**Silty SAND with Gravel (SM)**: medium-grained, non-plastic to low plasticity, grayish brown (2.5Y 5/2), moist, subrounded gravel (1"), little Clay content	S1			24"				83	29			
	5														

The boring was terminated at approximately 5 ft. below ground surface. The boring was backfilled with gravel and concrete on April 05, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by ALTA SURVEY.

Elevation scale: 230, 225, 220, 215, 210; Depth scale: 10, 15, 20, 25

BORING LOG PERC-2

Date Begin - End:	4/04/2018	**Drilling Company:**	ABC LIOVIN
Logged By:	S.Tena	**Drill Crew:**	Raul & Juan
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-85
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 233.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			ASPHALT: (3 inches)												
			BASE COURSE: (4 inches)												
			Silty SAND (SM): fine-grained, non-plastic, grayish brown (2.5Y 5/2), moist												
230				S1			24"				91	48			
	5														
225															
	10														
220															
	15														
215															
	20														
210															
	25														
205															

The boring was terminated at approximately 5 ft. below ground surface. The boring was backfilled with gravel and concrete on April 05, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by ALTA SURVEY.

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PROJECT NO.:	20183768
DRAWN BY:	ST
CHECKED BY:	SHR
DATE:	4/6/2018
REVISED:	6/6/2018

BORING LOG PERC-2

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

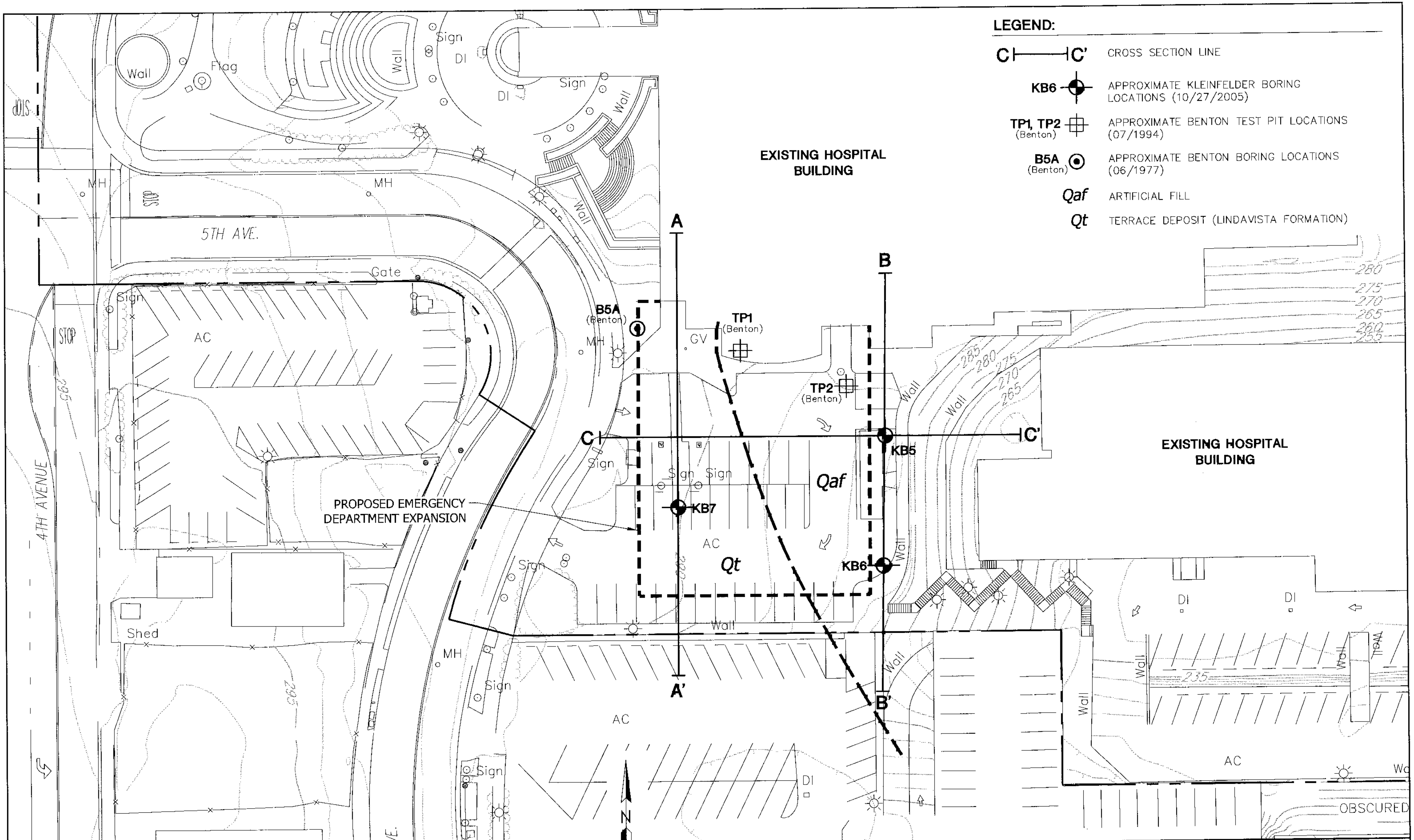
FIGURE
A-9

APPENDIX A.1

Previous Boring Logs

GEOLOGIC AND GEOTECHNICAL ENGINEERING EVALUATION
SCRIPPS MERCY HOSPITAL EXPANSION
NEW EMERGENCY DEPARTMENT
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

January 20, 2006



KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: SR	FN: 57557SITE-1
PROJECT NO. 57557	DATE: 11/2005

BORING LOCATION AND SITE GEOLOGY MAP

SCRIPPS MERCY HOSPITAL EMERGENCY DEPARTMENT SAN DIEGO, CALIFORNIA

FIGURE **2**

APPENDIX A

Boring Logs

APPENDIX A
BORING LOGS

A Unified Soil Classification System (USCS) chart and a Boring Log Legend are presented as Figures A1 and A2, respectively. The Logs of Borings performed by Kleinfelder are presented as Figures A3 through A5. The Logs of Borings describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The log also shows the boring number, drilling date, and the names of the logger and drilling subcontractor. The borings were logged by an engineering geologist using the USCS classification system. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and intact samples of representative earth materials were obtained from the borings.

The exploratory borings were advanced using a CME 75 truck-mounted drill rig, equipped with 8-inch-diameter hollow-stem augers. Each boring was backfilled with hydrated bentonite and patched at the surface with asphalt.

In-place soil samples were obtained at the test boring locations using a California sampler and a Standard Penetration Sampler (SPT) driven a total of 18-inches (or until practical refusal), into the undisturbed soil at the bottom of the boring. The soil sampled by the California sampler (3-inch O.D., 2.4-inches I.D.) was retained in 6-inch long brass tubes for laboratory testing. An additional 2-inches of soil from each drive remained in the cutting shoe and was usually discarded after visually classifying the soil. The samplers were driven using a 140-pound hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count and is recorded on the Logs of Borings. Please note that these blow counts have not been adjusted for the effects of overburden pressure, input driving energy, rod length, sampler correction, or boring diameter correction.

Boring and Test Pits performed by BEI (1977, 1994) are also included in this Appendix.

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: SR | FN: 57557VIC
PROJECT NO. 57557 | DATE: 11/2005

SOIL CLASSIFICATION CHART

SCRIPPS MERCY HOSITAL
EMERGENCY DEPARTMENT
SAN DIEGO, CALIFORNIA

FIGURE
A1

LOG SYMBOLS:

BULK/BAG SAMPLE

MODIFIED CALIFORNIA SAMPLER
(2-1/2 inch outside diameter)

CALIFORNIA SAMPLER
(3 inch outside diameter)

STANDARD PENETRATION
SPLIT SPOON SAMPLER
(2 inch outside diameter)

NO SAMPLE RECOVERY

SHELBY TUBE

▼ WATER LEVEL
(level after completion)

∇ WATER LEVEL
(level where first encountered)

ABBREVIATIONS:

SA – (38%) SIEVE ANALYSIS (PERCENT PASSING #200 SIEVE)
WA – (38%) – ONE POINT GRAIN SIZE ANALYSIS (PERCENT PASSING #200 SIEVE)
PI – PLASTICITY INDEX
LL – LIQUID LIMIT
DS – DIRECT SHEAR TEST
'R' – R-VALUE TEST
CORR – CORROSIVITY TEST
EI – UBC EXPANSION INDEX
LC – LABORATORY COMPACTION TEST
M&D – MOISTURE & DENSITY
PP – POCKET PENETROMETER

GENERAL NOTES:

1. *Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.*
2. *No warranty is provided as to the continuity of soil conditions between individual sample locations.*
3. *Logs represent general soil conditions observed at the point of exploration on the date indicated.*
4. *In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.*

CONSISTENCY CRITERIA BASED ON FIELD TESTS

RELATIVE DENSITY	SPT* (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	0 – 15
Loose	4 – 10	15 – 35
Medium Dense	10 – 30	35 – 65
Dense	30 – 50	65 – 85
Very Dense	>50	85 – 100

		TORVANE	POCKET** PENETROMETER
CONSISTENCY	SPT (# blows/ft)	UNDRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH
Very Soft	<2	<0.13	<0.25
Soft	2 – 4	0.13 – 0.25	0.25 – 0.5
Medium Stiff	4 – 8	0.25 – 0.5	0.5 – 1.0
Stiff	8 – 15	0.5 – 1.0	1.0 – 2.0
Very Stiff	15 – 30	1.0 – 2.0	2.0 – 4.0
Hard	>30	>2.0	>4

* NUMBER OF BLOWS OF 140 POUNDS HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM-1386 STANDARD PENETRATION TEST)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ. FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: SR | FN: 57557VIC
PROJECT NO. 57557 | DATE: 11/2005

KEY TO LOGS

**SCRIPPS MERCY HOSITAL
EMERGENCY DEPARTMENT
SAN DIEGO, CALIFORNIA**

FIGURE **A2**

DATE DRILLED:	10/27/2005	WATER DEPTH:	None
DRILLING COMPANY:	Baja Exploration	DATE OBSERVED:	10/27/2005
DRILLING METHOD:	CME 75, 140 lb hammer, 30" drop	GROUND ELEVATION:	±288 feet MSL
		LOGGED BY:	SHR
HOLE DIAMETER:	8" Hollow Stem Auger (HSA)	REVIEWED BY:	BRB

ELEVATION (ft.)	DEPTH (feet)	SAMPLES: BULK	SAMPLES: DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	NOTES, LAB, PID READINGS (ppm)
	0						4 inches of asphalt concrete pavement over 3 inches gravel base			
							FILL:			
285			■	24	1		SILTY SAND (SM), light greenish gray, moist to wet, medium dense, small rootlets observed	105.6	15.6	DS, M&D
	5			27	2		SILTY SAND with trace GRAVEL (<5%) (SM), dark greenish gray, moist, medium dense			
280							SANDY SILT (ML), medium greenish gray, moist, dense, fine grained roots, trace gravel, reddish brown mottling			
	10		■	24	3			105.2	12.7	M&D
275										
	15			26	4					SA (68%)
270										
	20		■	69	5		No recovery due to cobble Broken cobble in tip			
265							**SAN DIEGO FORMATION:**			
							SILTY SAND (SM), light greenish gray, moist to wet, dense, fine grained, mostly massive with minor observed bedding, weakly to moderately cemented			
	25			50/4"	6					
260										
	30		■	50/6"	7			89.2	10.5	DS, M&D
							Total depth of boring 30.9 feet No groundwater observed Boring backfilled with bentonite grout			
255										

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

PROJECT NO. 57557

SCRIPPS MERCY HOSPITAL
EMERGENCY DEPARTMENT
SAN DIEGO, CALIFORNIA

LOG OF BORING KB5

FIGURE
A3

DATE DRILLED:	10/27/2005	WATER DEPTH:	None
DRILLING COMPANY:	Baja Exploration	DATE OBSERVED:	10/27/2005
DRILLING METHOD:	CME 75, 140 lb hammer, 30" drop	GROUND ELEVATION:	±288 feet MSL
		LOGGED BY:	SHR
HOLE DIAMETER:	8" Hollow Stem Auger (HSA)	REVIEWED BY:	BRB

ELEVATION (ft.)	DEPTH (feet)	SAMPLES: BULK	SAMPLES: DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	NOTES, LAB, PID READINGS (ppm)
	0						4 inches of asphalt concrete			
		X			1		**FILL:**			EI=0
285			■	50/5"	2		SILTY SAND (SM), greenish gray, moist to wet, medium dense, fine grained, driving on cobble, poor sample recovery			'R'=65
	5		■	27	3					
280							**SAN DIEGO FORMATION:**			
	10		■	50/4"	4		SILTY SAND (SM), yellow brown with reddish brown mottling, moist, dense, very fine grained, weakly to moderately cemented	87.8	10.9	
275										
	15		■	45	5		Becomes light greenish gray			
270										
	20		■	50/6"	6		becomes very dense, fine grained			
265										
	25		■	50/5"	7					
260							Bottom of boring 26.5 feet No groundwater observed Boring backfilled with bentonite grout			
	30									
255										

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

PROJECT NO. 57557

SCRIPPS MERCY HOSPITAL
EMERGENCY DEPARTMENT
SAN DIEGO, CALIFORNIA

LOG OF BORING KB6

FIGURE A4

DATE DRILLED:	10/27/2005	WATER DEPTH:	None
DRILLING COMPANY:	Baja Exploration	DATE OBSERVED:	10/27/2005
DRILLING METHOD:	CME 75, 140 lb hammer, 30" drop	GROUND ELEVATION:	±290 feet MSL
		LOGGED BY:	SHR
HOLE DIAMETER:	8" Hollow Stem Auger (HSA)	REVIEWED BY:	BRB

ELEVATION (ft.)	DEPTH (feet)	SAMPLES: BULK	SAMPLES: DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	NOTES, LAB, PID READINGS (ppm)
							4 inches of asphalt concrete pavement over 4 inches gravel base			
							LINDAVISTA FORMATION:			
				50/4"	1		SILTY SAND with trace GRAVEL (SM), light reddish brown, dry, very dense, coarse grained, very angular gravel 0 to 10%			M&D, SA (12%)
285	5			50/4"	2		1.5" total sample recovery, probable slough and not representative of in place material	112.8	7.3	
							Gravel zone 6 to 8 feet			
							SAN DIEGO FORMATION:			
280	10			85	3		SILTY SAND (SM), light greenish gray, dry to moist, very dense, fine grained, slightly micaceous, massive, weakly to moderately cemented	97.2	10.5	M&D
275	15			35	4		Medium greenish gray silt bed at 15 to 16 feet Light greenish gray, dry to moist, dense to very dense, slightly micaceous, massive			
270	20			50/5"	5		Becomes moist, very dense, slightly micaceous, massive, vertical fracture in sample			
							Bottom of boring 20.9 feet No groundwater observed Boring backfilled with bentonite			
265	25									
260	30									

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

PROJECT NO. 57557

SCRIPPS MERCY HOSPITAL
EMERGENCY DEPARTMENT
SAN DIEGO, CALIFORNIA

LOG OF BORING KB7

FIGURE **A5**

PREVIOUS TEST PITS AND BORINGS BY
BENTON ENGINEERING INC.
(1977 and 1994)

SUMMARY SHEET

PIT NO. 1

DEPTH/FEET	SAMPLE NUMBER	DESCRIPTION	SOIL CLASSIFICATION	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	
0		2" Asphaltic Concrete	PAVEMENT					Fil
1	(1) 1	Brown, Moist, Compact	SILTY FINE TO MEDIUM SAND	8.8	11.0	119.8	3.90	
2	[2] (2)	Red Brown, Moist, Medium Compact	FINE TO MEDIUM SANDY CLAY	8.0	3.3	103.1	0.67	
3	(NR)			30.0	--	--	--	
		Brown, Slightly Moist, Medium Compact, 15-20% Gravel to 1½" with Loose Lenses	SILTY FINE TO MEDIUM SAND					
		Red Brown, Slightly Moist, Very Firm, Lightly Cemented	SILTY FINE TO MEDIUM SAND					

Stop at 3'-4"

Pit No. 2

DEPTH/FEET	SAMPLE NUMBER	DESCRIPTION	SOIL CLASSIFICATION	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	
0		5" Asphaltic Concrete	PAVEMENT					
1	(1) 1	Brown, Moist, Compact, Decomposed Granite Base	SILTY FINE TO MEDIUM SAND	7.0	11.6	111.3	0.24	
2	(2)	Light Gray Brown, Moist, Compact, 15-20% Gravel to 1", 1 - 4½" Cobble	GRAVELLY SILTY FINE TO MEDIUM SAND	8.0	15.4	106.9	0.74	Fil
3–4	(3)	Asphalt and Concrete Chunks to 6"	GRAVELLY SILTY FINE TO MEDIUM SAND	9.0	13.2	107.5	1.19	
	(NR)			16.0				
		Light Gray Brown, Moist, Compact, Few Gravel to 1"	SILTY FINE SAND					

Stop on Concrete Chunk at 4'-2"
Need to Enlarge Pit to Remove
Concrete and Deepen.

□ Indicates Loose Bulk Sample

○ Indicates Undisturbed Drive Sample

(NR) Indicates Sample Not Recovered

PROJECT NO.		DRAWING NO.
94-6-17F	BENTON ENGINEERING, INC.	2

SUMMARY SHEET

BORING NO. 5A

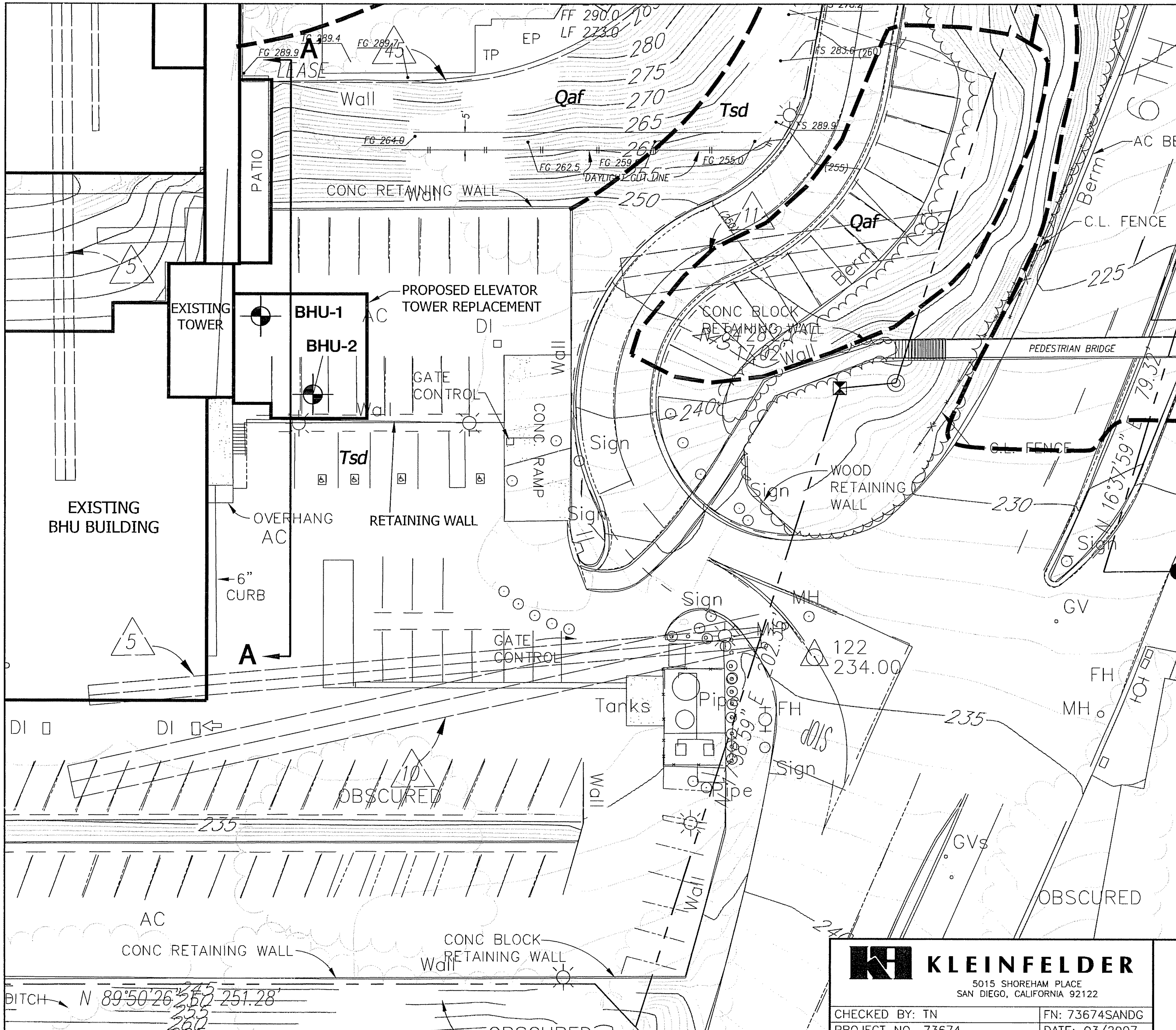
ELEVATION 290.5'

Depth in Feet	Sample Number	Description	Soil Type	Drive Energy Ft. Kips/Ft.	Field Moisture % Dry Wt.	Dry Density Lbs/Cu. Ft.	Shear Resistance Kips/Sq. Ft.	
0		3 1/2 Inches Asphaltic Concrete	PAVING					FILL
1–3	1A, [1], 1	Yellow Brown and Red Brown, Moist, Compact, 20 to 30 Percent Gravel to 1 1/2 Inches	GRAVELLY SLIGHTLY CLAYEY FINE TO MEDIUM SAND	1.0	18.5	108.4	1.14	
3–5.5	[2]	Brown, Moist, Firm, With Clay Binder, Occasional Gravel to 1 Inch Saturated Soft	SILTY FINE TO MEDIUM SAND					
5.5–8		Gray Red Brown, Very Moist, Firm, Occasional Gravel to 1 Inch	CLAY					
8–11		Light Yellow Brown, Moist, Firm to Medium Firm, With Clay Binder, 40 to 50 Percent Gravel to 3 Inches, Cobble to 6 Inches at 7 Foot Depth, 12 Inch Cobble at 7.5 Foot Depth	GRAVELLY SILTY FINE TO COARSE SAND					
11–12	[3], 2	Light Yellow Brown and Light Gray, Moist, Very Firm	SILTY FINE SAND	14.6	11.2	91.9	1.79	
14	3			14.6	12.6	91.8	2.03	
17	4			19.5	14.9	97.6	0.99	
20	5			17.9	15.9	97.7	1.28	

Continued on Drawing No. 14

DRAFT REPORT
GEOLOGIC AND GEOTECHNICAL ENGINEERING EVALUATION
SCRIPPS MERCY HOSPITAL EXPANSION
BHU ELEVATOR TOWER REPLACEMENT
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

March 27, 2007



LEGEND:

BHU-1	APPROXIMATE BORING LOCATION
A——A'	CROSS SECTION LINE
— — —	APPROXIMATE GEOLOGIC CONTACT
Qaf	ARTIFICIAL FILL
Tsd	SAN DIEGO FORMATION

N

0 30 60

APPROXIMATE GRAPHIC SCALE (FEET)

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

CHECKED BY: TN	FN: 73674SANDG
PROJECT NO. 73674	DATE: 03/2007

SITE PLAN AND GEOLOGIC MAP

SCRIPPS MERCY HOSPITAL EXPANSION
BHU ELEVATOR TOWER REPLACEMENT
SAN DIEGO, CALIFORNIA

FIGURE
2

APPENDIX A

Boring Logs

APPENDIX A
BORING LOGS

Our field exploration consisted of drilling two exploratory borings (designated BHU-1 and BHU-2) during August 2006 within the footprint of the proposed Elevator Tower Replacement. The borings were advanced to termination depths of 18 and 47 feet using a CME-75, truck-mounted drill rig, equipped with 8-inch-diameter hollow-stem augers. Air-rotary percussion drilling methods were utilized to advance the deeper portion of Boring BHU-2 due to the presence of cobbles. Each boring was backfilled with hydrated bentonite and patched at the surface with asphalt.

In-place soil samples were obtained at each test boring location using a modified California penetration sampler or a Standard penetration sampler (SPT) driven a total of 18-inches (or until practical refusal) into the undisturbed soil at the bottom of the boring. The samplers were driven using a 140-pound hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count and is recorded on the Logs of Borings. Please note that these blow counts have not been adjusted for the effects of overburden pressure, input driving energy, rod length, sampler correction, or boring diameter correction. The soil sampled by the California sampler (3-inch O.D., 2.4-inches I.D.) was retained in 6-inch long brass tubes. An additional 2-inches of soil from each drive remained in the cutting shoe and was usually discarded after visually classifying the soil. Bulk and intact samples of representative earth materials obtained from the borings were sealed in the field and returned to our laboratory for testing.

A Unified Soil Classification System (USCS) chart and a Boring Log Legend are presented as Figures A1 and A2, respectively. The Logs of Boring performed by Kleinfelder are presented as Figure A3 and A4. The Logs of Boring describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The log also shows the boring number, drilling date, and the names of the logger and drilling subcontractor.

The boring was logged by an staff engineer using the USCS classification system. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual.

The rig was equipped with 8-inch diameter hollow stem augers and a 140-pound down-hole hammer dropped from a height of 30 inches to drive the sampler into the soil. Due to the presence of cobbles, air-rotary percussion drilling methods were also employed in Boring BHU-2. The locations of the borings are shown on the Boring Location Plan, Figure 2.

An engineer from our office logged the soil conditions exposed in the borings and collected soil samples for laboratory testing.

Driven samples were obtained from the borings using Standard Penetration Test and Modified California samplers. The sampler was generally driven 18 inches into undisturbed soil using the hammer. The number of blows necessary to drive the sampler was recorded at each sampling location. Soil samples obtained from the borings were packaged, sealed in the field to reduce moisture loss and disturbance, and returned to our laboratory for testing. Upon completion, the boring was backfilled with bentonite grout and the surface was patched with asphalt concrete. A more detailed description of the field exploration program, and logs of the exploratory borings, is presented in Appendix A.

DATE DRILLED:	8/2/2006	WATER DEPTH:	None
DRILLING COMPANY:	Test America	DATE OBSERVED:	08/02/2006
DRILLING METHOD:	CME-75, 140 lb hammer, 30" drop	GROUND ELEVATION:	±240 feet MSL
		LOGGED BY:	JL
HOLE DIAMETER:	8" Hollow Stem Augers	REVIEWED BY:	SHR/TRN

ELEVATION (feet)	DEPTH (feet)	SAMPLES: BULK	SAMPLES: DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	COMMENTS/ ADDITIONAL TESTS
							2 inches of Asphalt Concrete Pavement			
							3 inches of Aggregate Base			
		X			1		**FILL (Qaf):** GRAVELLY SILT (ML), light brown, moist medium dense, subangular and subrounded gravel			'Corr'
235	5						---Concrete debris			
			■	75	2		POORLY GRADED SAND (SP), orange brown, moist, medium dense, fine to coarse grained, trace gravel	109.4	4	
							SAN DIEGO FORMATION (Tsd):			
230	10						SANDY SILT (ML), greenish gray, moist, dense			
			■	60	3		POORLY GRADED SAND (SP), gray to orange brown, moist, very dense, fine grained			
225	15		■	100/3"	4		GRAVELLY SILT (ML), orange brown with gray, moist, very dense, subrounded gravel			
							---Drilling action indicates cobble layer at 15 to 18 feet.			
							POMERADO CONGLOMERATE (Tp):			
							SILTY SAND (SM), yellowish brown, moist, very dense, with gravel and cobble, drill action indicates cobble layer at 15 to 18 feet			
220	20						Bottom of boring at 18 feet due to auger refusal on cobbles. Groundwater was not observed in the borehole during drilling.. Borehole was backfilled with bentonite grout and the surface patched with quickcrete.			
215	25									
210	30									

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

PROJECT NO. 73674

SCRIPPS MERCY HOSPITAL EXPANSION
BHU ELEVATOR TOWER REPLACEMENT
SAN DIEGO, CALIFORNIA

LOG OF BORING BHU-1

FIGURE **A3**

DATE DRILLED:	8/3/2006	WATER DEPTH:	None
DRILLING COMPANY:	Test America	DATE OBSERVED:	08/03/2006
DRILLING METHOD:	CME-75, 140 lb hammer, 30" drop; Rotary drilling followed by air percussion drilling method.	GROUND ELEVATION:	±240 feet MSL
		LOGGED BY:	JL
HOLE DIAMETER:	8" Hollow Stem Augers	REVIEWED BY:	SHR/TRN

ELEVATION (feet)	DEPTH (feet)	SAMPLES: BULK	SAMPLES: DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	COMMENTS/ ADDITIONAL TESTS
							2 inches of Asphalt Concrete Pavement			
		X			1		3 inches of Aggregate Base			
							FILL (Qaf):			
							GRAVELLY SILT (ML), brown, moist, loose to medium dense, subrounded and subangular gravel			
235	5						Concrete debris in fill			
							SAN DIEGO FORMATION (Tsd):			
			■	50/6"	2		SANDY SILT (ML), light olive to gray, damp to moist, dense to very dense, massive	92.0	9	DS
230	10									
			■	69	3					LL=30 PI=5
225	15									
							POMERADO CONGLOMERATE (Tp):			
							SILTY SAND (SM), yellowish brown, damp to moist, very dense, with abundant gravel and cobble			
		X			4		--- Switched to air percussion drilling			
220	20									
215	25									
210	30	X			5					
		X								

KLEINFELDER
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SAN DIEGO, CALIFORNIA 92122

PROJECT NO. 73674

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BHU ELEVATOR TOWER REPLACEMENT
SAN DIEGO, CALIFORNIA

LOG OF BORING BHU-2

FIGURE **A4**

ELEVATION (feet)	DEPTH (feet)	SAMPLES BULK	SAMPLES DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION (Continued From Previous Page)	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	COMMENTS/ ADDITIONAL TESTS
		X			6					
200	40	X			7		SILTY SAND (SM), yellowish brown, damp to moist, very dense, with gravel and cobble			
195	45	X			8					
							Bottom of boring at 47 feet No groundwater observed Boring backfilled with grout			
190	50									
185	55									
180	60									
175	65									
170	70									
165	75									

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

PROJECT NO. 73674

SCRIPPS MERCY HOSPITAL EXPANSION
BHU ELEVATOR TOWER REPLACEMENT
SAN DIEGO, CALIFORNIA

LOG OF BORING BHU-2

FIGURE

A5

APPENDIX B

Laboratory Test Results

APPENDIX B
LABORATORY TEST RESULTS

GENERAL

Laboratory tests were performed on selected, representative samples as an aid in classifying the soils and to evaluate physical properties of the soils, which may affect foundation design and construction procedures. A description of our laboratory testing program is presented below:

CLASSIFICATION

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory excavations in Appendix A.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Natural moisture content and dry density tests were performed on the intact samples collected. Moisture content was evaluated in general accordance with ASTM Test Method D 2216; dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. The values are shown on the boring logs in Appendix A.

SIEVE ANALYSIS

Sieve analyses were performed on nine samples from the site to evaluate the gradation characteristics of the soil and to aid in its classification. The tests were performed in general accordance with ASTM Test Method D 422. The test results are presented on Figures B-1 through B-9.

ATTEBERG LIMITS TEST

Liquid and plastic limits tests were conducted on two selected samples. The test was performed in general accordance with ASTM D 4318. In addition, the values are shown on the boring logs in Appendix A and Figure B-10.

DIRECT SHEAR TEST

Three direct shear tests were performed on representative soil samples to evaluate the shear strength of the site soils. One of the samples was remolded to 90% of maximum dry density determined by ASTM D 1557. The soil samples were tested in a saturated state and subjected to three different normal pressures in general accordance with ASTM Test Method D3080. The test results are presented on Figures B-11 through B-13.

COMPACTION CURVE

One test was performed on a representative soil sample to evaluate the laboratory compaction characteristics of the soil using the modified effort for compaction. The test procedure was in general accordance with the ASTM D 1557. The test result is presented on Figure B-14.

R-Value

R-value test was performed on a selected soil sample to evaluate resistance value of the near surface soils. The test was performed in general accordance with ASTM Test Method D2844. The result is presented in Table B-1.

Table B-1

R-Value Test Result

Sample	Depth (ft)	R-Value
B-4/S1	0.5-4	66

EXPANSION INDEX TEST

Two expansion index (EI) tests were performed on representative samples of the near-surface materials obtained during our investigation. The test was performed in accordance with ASTM D4829. The corrected expansion index values for the samples are presented in Table B-2. The test result indicates a very low expansion potential when compared to Table B-3 to qualitatively evaluate the expansion potential of the site soils.

Table B-2

Expansion Index Test Results

Boring	Depth (ft)	Soil Type	EI
B-1	1-5	Silty Sand	1
B-5	1-5	Silty Sand	0

Table B-3

ASTM D 4829 Expansion Index and Potential

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

CORROSION TEST

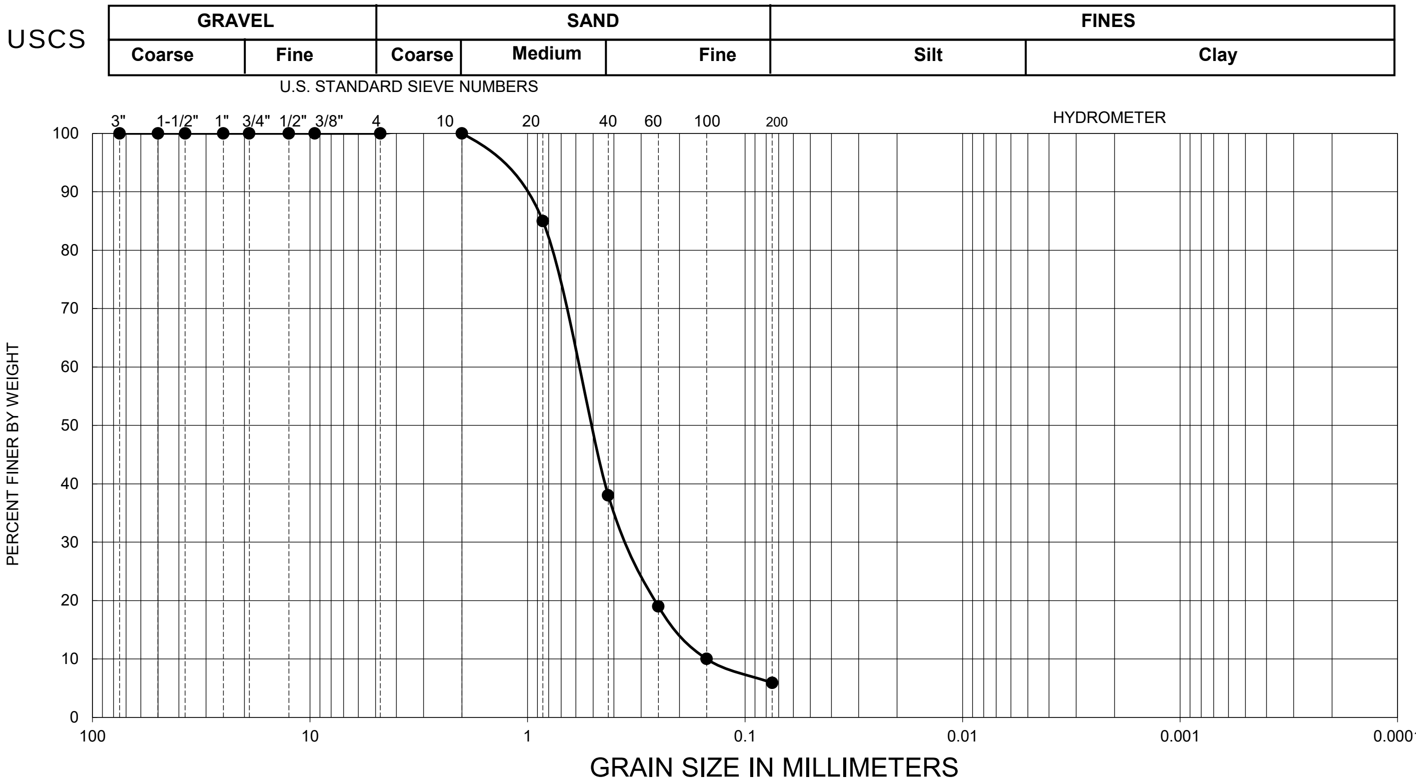
The sulfate and chloride contents and pH of two selected samples were evaluated in general accordance with California Test 417 and California Test 422. Our boring logs and these test results should be reviewed by a qualified corrosion engineer to evaluate the general soil stratigraphy corrosion potential with respect to construction materials and determine whether further testing is warranted. The test results are presented on Table B-4 and Figures B-15 through B-16.

Table B-4

Corrosion Test Results

Sample	Depth (ft)	pH	Minimum Resistivity (ohm-cm)	Sulfate (%)	Chloride (%)
B-2/S1	1-5	8.0	1000	0.132	0.001
B-1 & B-4	15-16.5	9.1	1800	0.008	0.005

Date Tested: 5/1/2018



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B1	S2	6-6.5	5.9	SP-SM

Sample Description	Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	85
	No. 40	0.425 mm	38
	No. 60	0.25 mm	19
	No 100	0.15 mm	10
	No 200	.075 mm	5.9

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
Bright People. Right Solutions.

Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

GRADATION TEST RESULTS

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

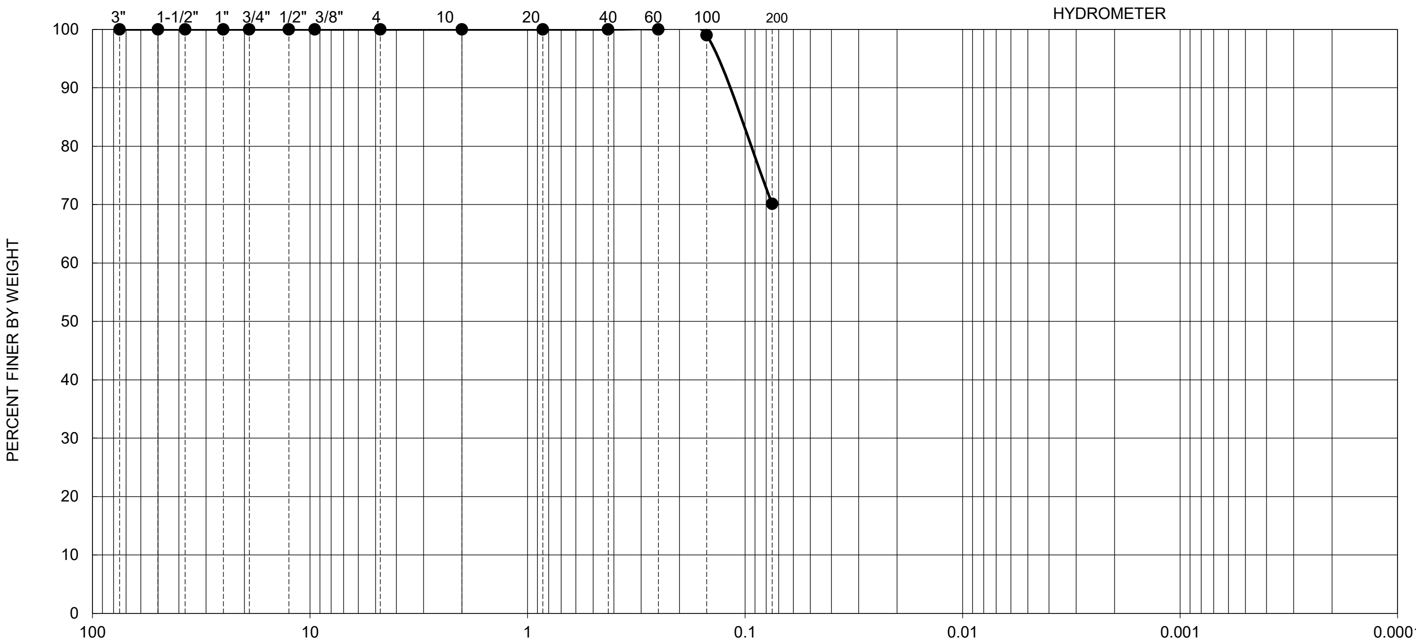
B-1

Date Tested: **5/1/2018**

USCS

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

U.S. STANDARD SIEVE NUMBERS



GRAIN SIZE IN MILLIMETERS

Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B1	S5	16-16.5	70.1	CL

Sample Description	Lean clay with sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	100
	No. 60	0.25 mm	100
	No 100	0.15 mm	99
	No 200	.075 mm	70.1

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Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

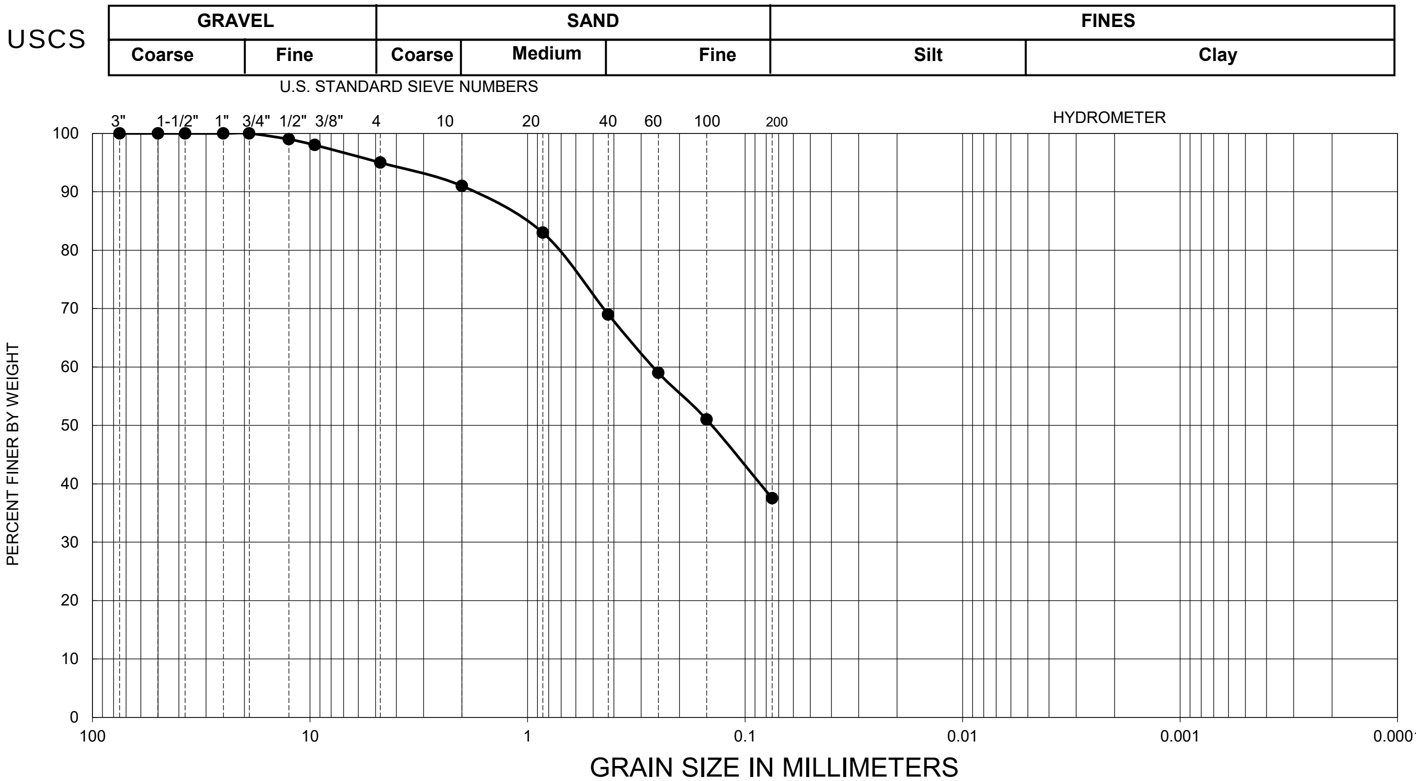
GRADATION TEST RESULTS

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-2

Date Tested: **5/1/2018**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B2	S3	11-11.5	37.5	SM

Sample Description	Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	99
	3/8"	9.5 mm	98
	No. 4	4.75 mm	95
	No. 10	2.0 mm	91
	No. 20	0.85 mm	83
	No. 40	0.425 mm	69
	No. 60	0.25 mm	59
	No 100	0.15 mm	51
	No 200	.075 mm	37.5

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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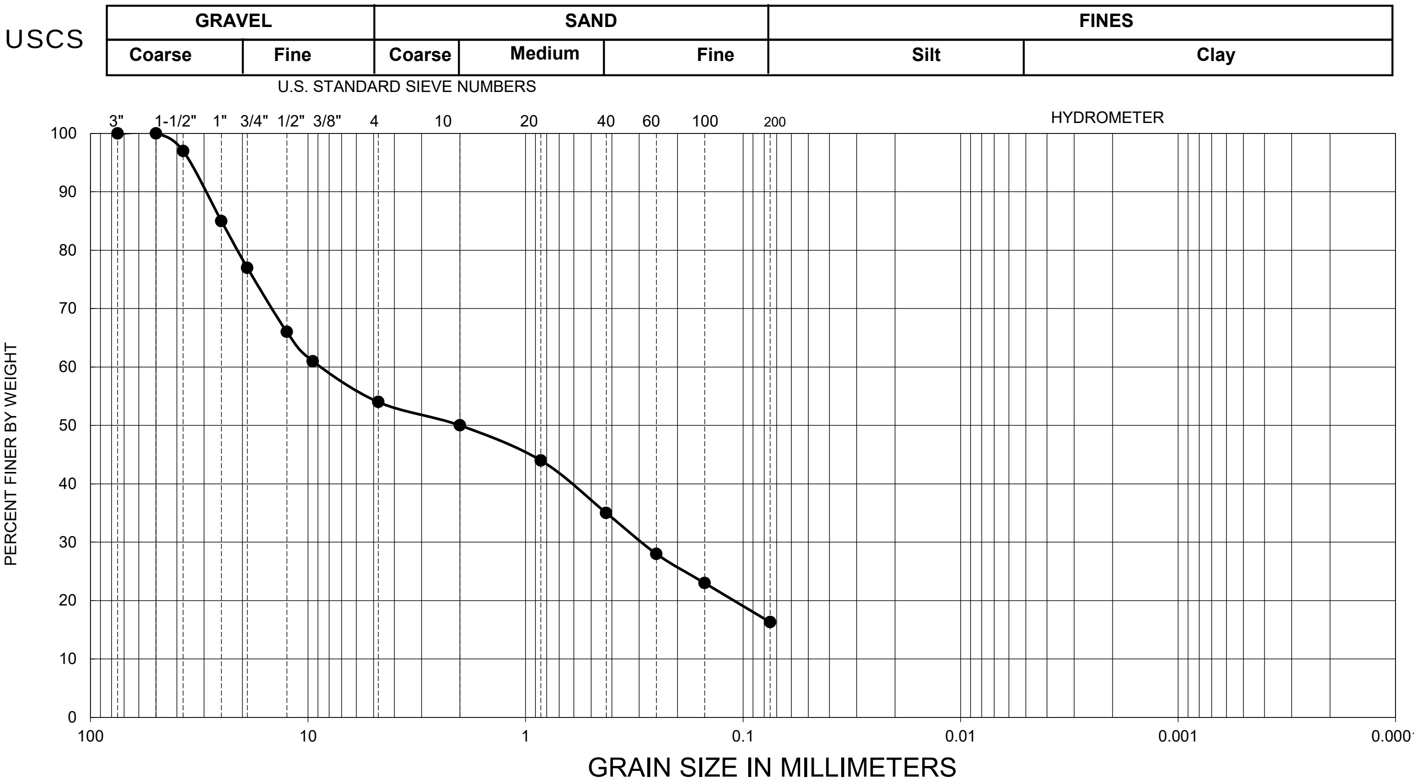
Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

GRADATION TEST RESULTS

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
B-3

Date Tested: 4/27/2018



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B3	S1	1-5	16.3	SM

Sample Description	Silty sand with gravel

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	97
	1"	25 mm	85
	3/4"	19 mm	77
	1/2"	12.5 mm	66
	3/8"	9.5 mm	61
	No. 4	4.75 mm	54
	No. 10	2.0 mm	50
	No. 20	0.85 mm	44
	No. 40	0.425 mm	35
	No. 60	0.25 mm	28
	No 100	0.15 mm	23
	No 200	.075 mm	16.3

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Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

GRADATION TEST RESULTS

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SCRIPPS MERCY HOSPITAL
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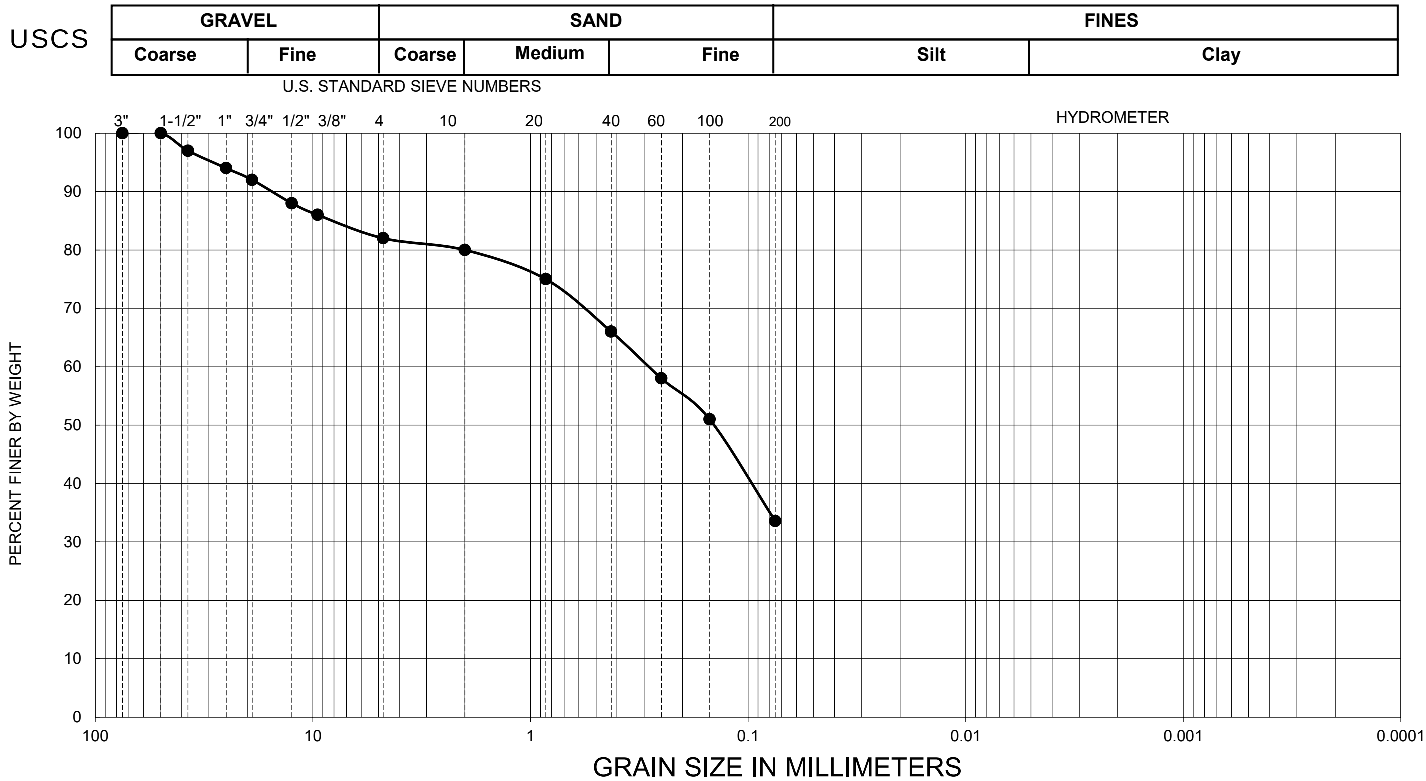
SAN DIEGO, CALIFORNIA

FIGURE

B-4

Date Tested: **4/27/2018**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B4	S1	0.5-4	33.6	SM

Sample Description	Silty sand with gravel

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	97
	1"	25 mm	94
	3/4"	19 mm	92
	1/2"	12.5 mm	88
	3/8"	9.5 mm	86
	No. 4	4.75 mm	82
	No. 10	2.0 mm	80
	No. 20	0.85 mm	75
	No. 40	0.425 mm	66
	No. 60	0.25 mm	58
	No 100	0.15 mm	51
	No 200	.075 mm	33.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
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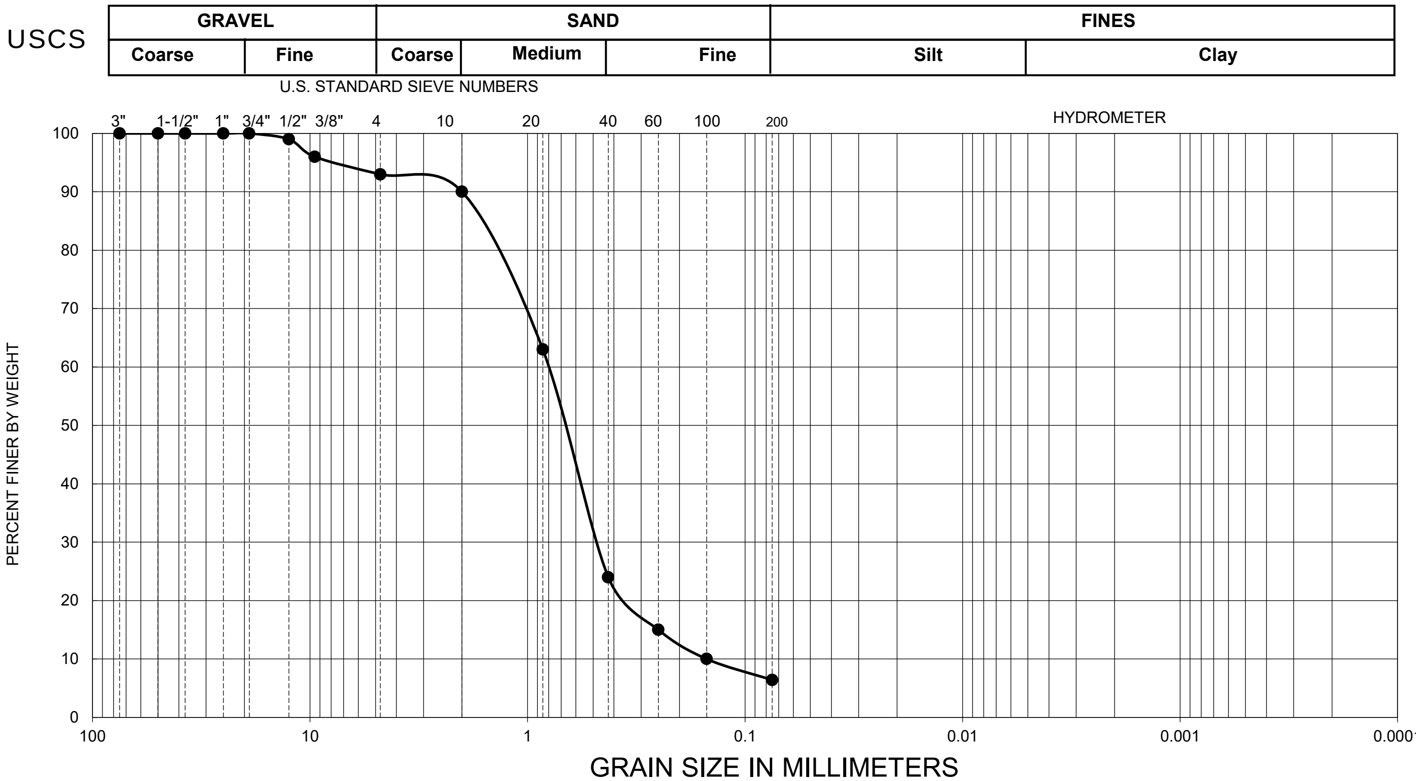
Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

GRADATION TEST RESULTS

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
B-5

Date Tested: 5/1/2018



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B5	S2	6-6.5	6.4	SP-SM

Sample Description	Poorly graded sand with silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	99
	3/8"	9.5 mm	96
	No. 4	4.75 mm	93
	No. 10	2.0 mm	90
	No. 20	0.85 mm	63
	No. 40	0.425 mm	24
	No. 60	0.25 mm	15
	No 100	0.15 mm	10
	No 200	.075 mm	6.4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
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Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

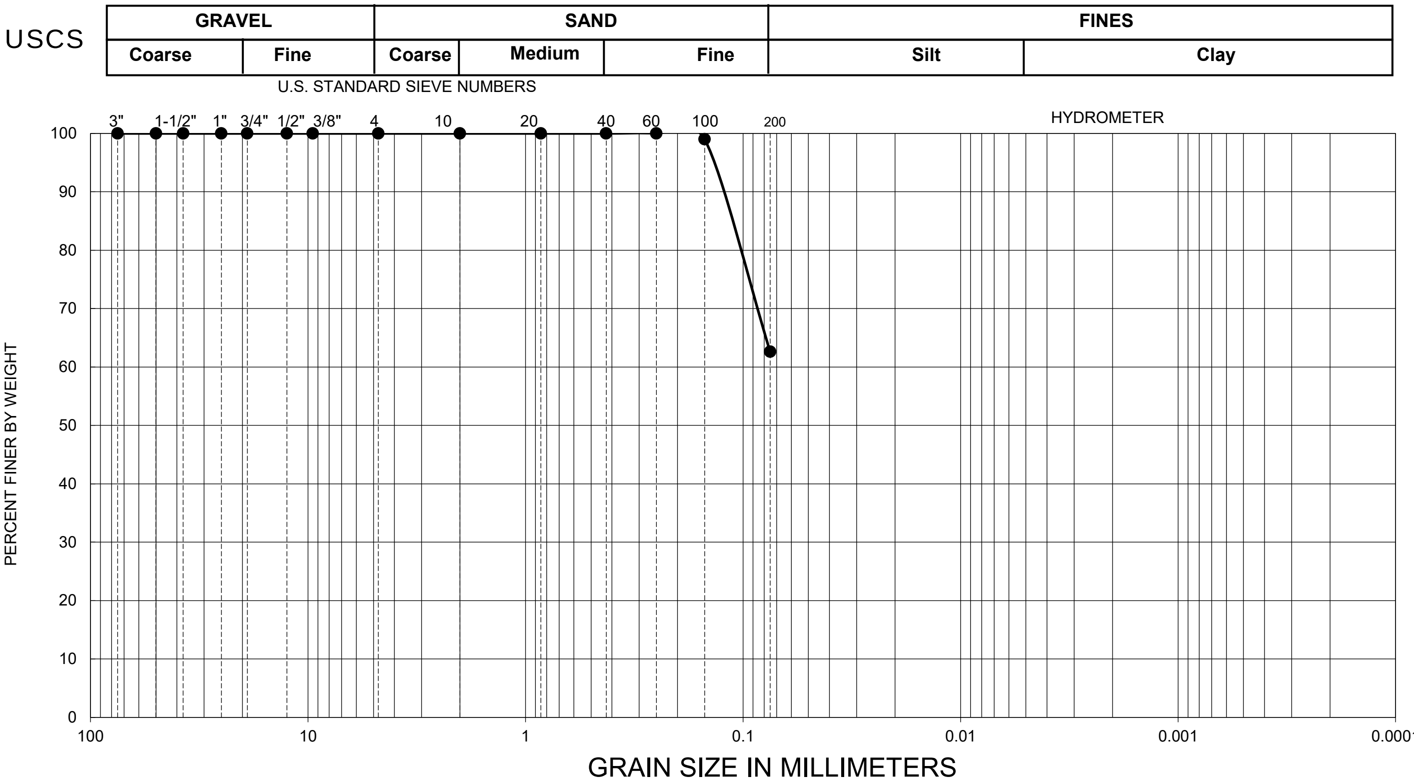
GRADATION TEST RESULTS

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-6

Date Tested: **5/1/2018**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B5	S4	16-16.5	62.6	ML

Sample Description	Sandy silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	100
	No. 60	0.25 mm	100
	No 100	0.15 mm	99
	No 200	.075 mm	62.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
Bright People. Right Solutions.

Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

GRADATION TEST RESULTS

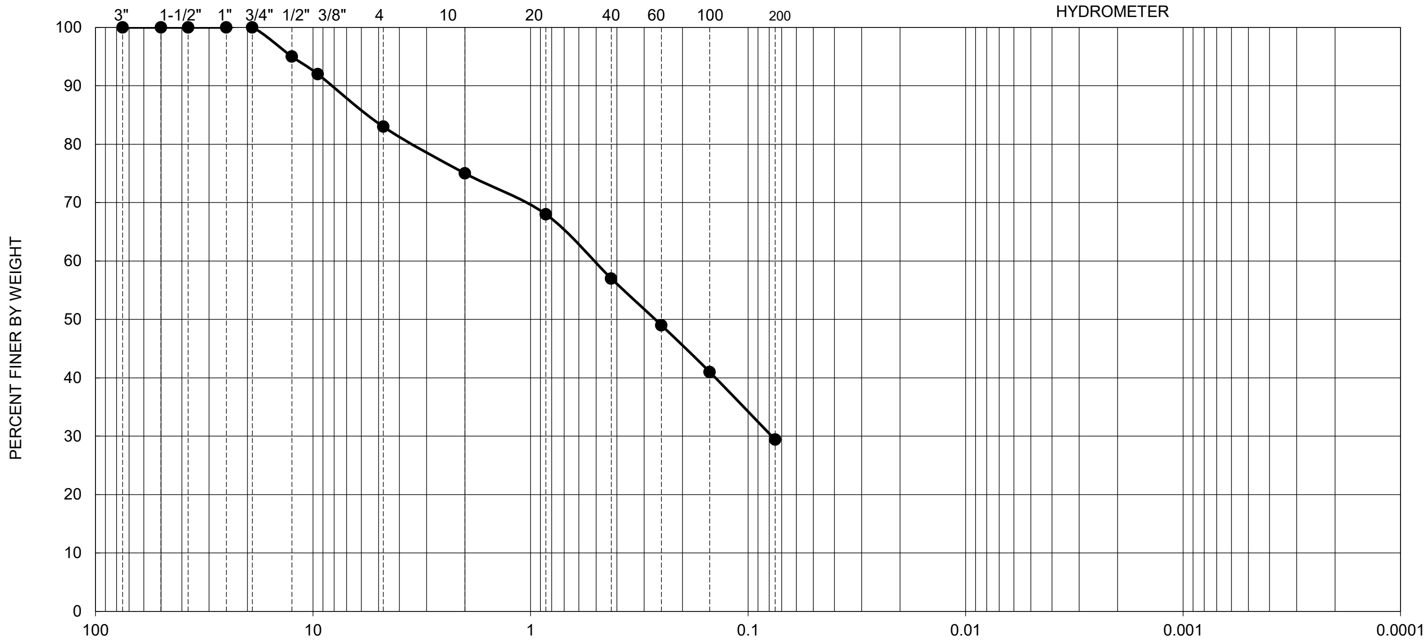
REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
B-7

Date Tested: **4/27/2018**

USCS	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

U.S. STANDARD SIEVE NUMBERS



GRAIN SIZE IN MILLIMETERS

Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
PERC1	S1	3-5	29.4	SM

Sample Description	Silty sand with gravel

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	95
	3/8"	9.5 mm	92
	No. 4	4.75 mm	83
	No. 10	2.0 mm	75
	No. 20	0.85 mm	68
	No. 40	0.425 mm	57
	No. 60	0.25 mm	49
	No 100	0.15 mm	41
	No 200	.075 mm	29.4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
Bright People. Right Solutions.

Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

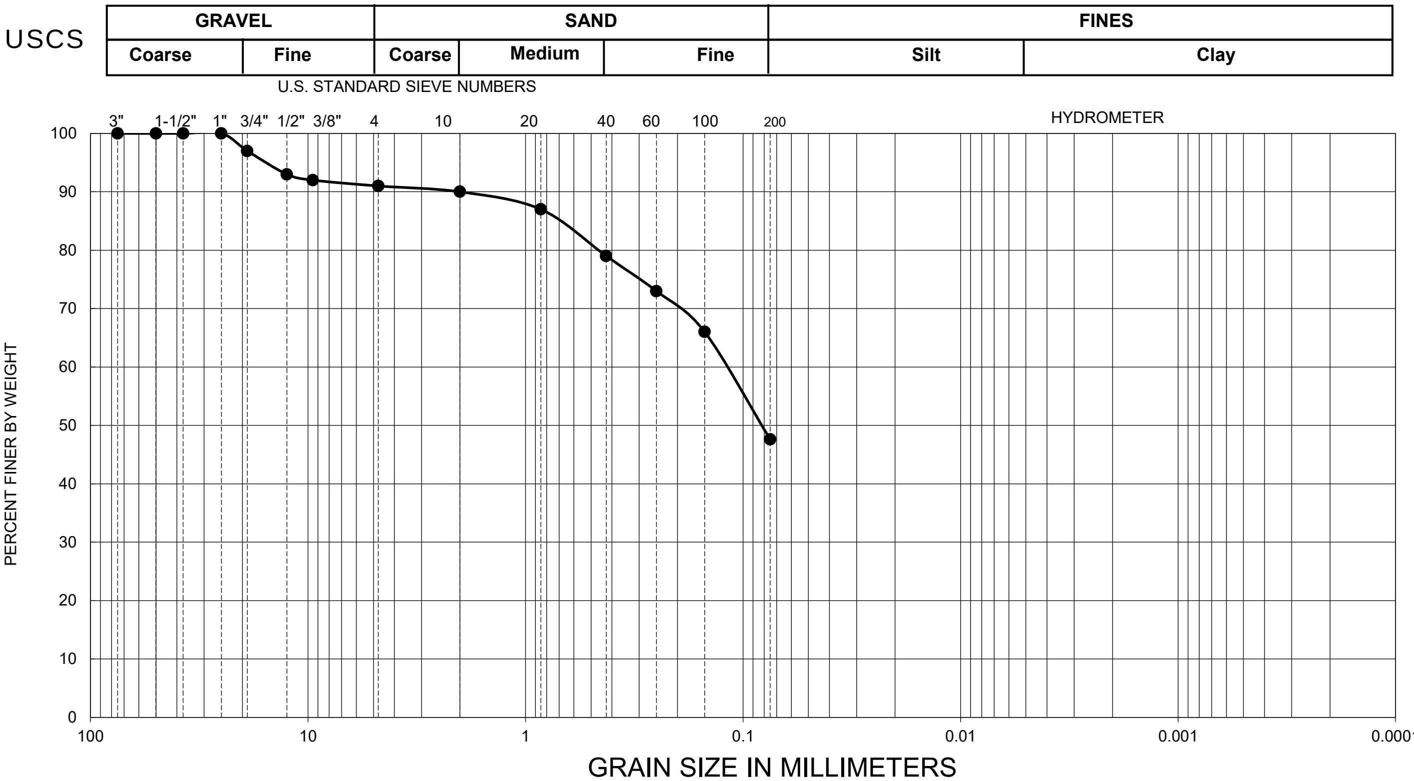
GRADATION TEST RESULTS

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-8

Date Tested: 4/27/2018



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
PERC2	S1	3-5	47.6	SM

Sample Description	Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	97
	1/2"	12.5 mm	93
	3/8"	9.5 mm	92
	No. 4	4.75 mm	91
	No. 10	2.0 mm	90
	No. 20	0.85 mm	87
	No. 40	0.425 mm	79
	No. 60	0.25 mm	73
	No 100	0.15 mm	66
	No 200	.075 mm	47.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
Bright People. Right Solutions.

Checked by:	Uly P.	Tech:	Carl D.
Project No.	20183768.001A	Date:	8-May-18

GRADATION TEST RESULTS

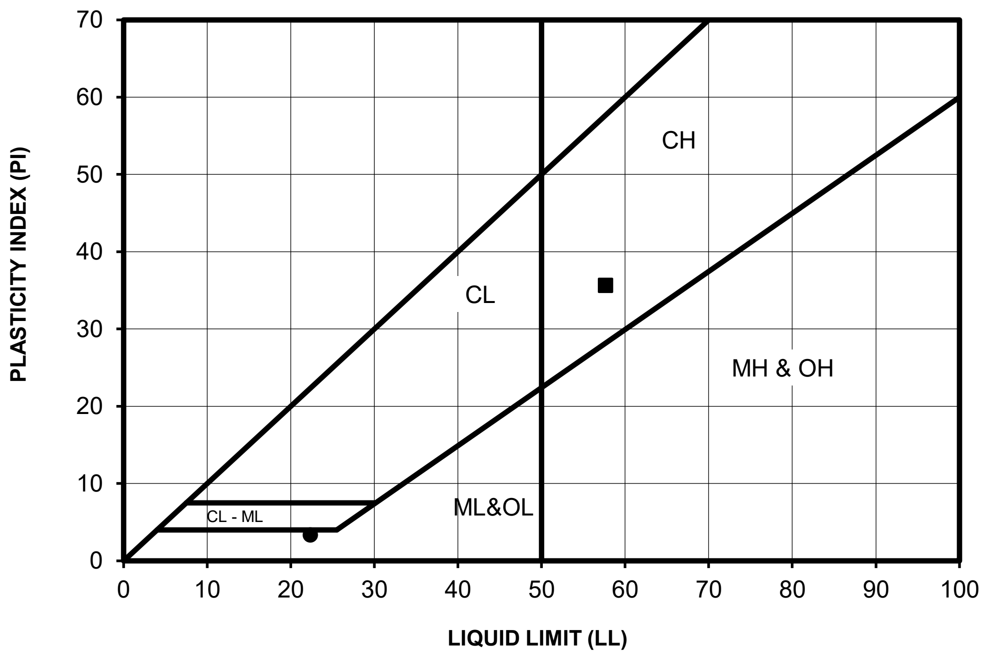
REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-9

Date Tested : 5/2/2018

SYMBOL	SAMPLE NAME	DEPTH (ft)	LL	PL	PI	USCS CLASSIFICATION (Minus No. 40 Sieve Fraction)	USCS (Entire Sample)
●	B2-S3	10.5-11	22	19	3	ML	SM
■	B3-S4	15-15.5	58	22	36	CH	SC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Limitations: Pursuant to applicable codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specification were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided. This report may not be reproduced, except in full, without written approval of Kleinfelder.

KLEINFELDER
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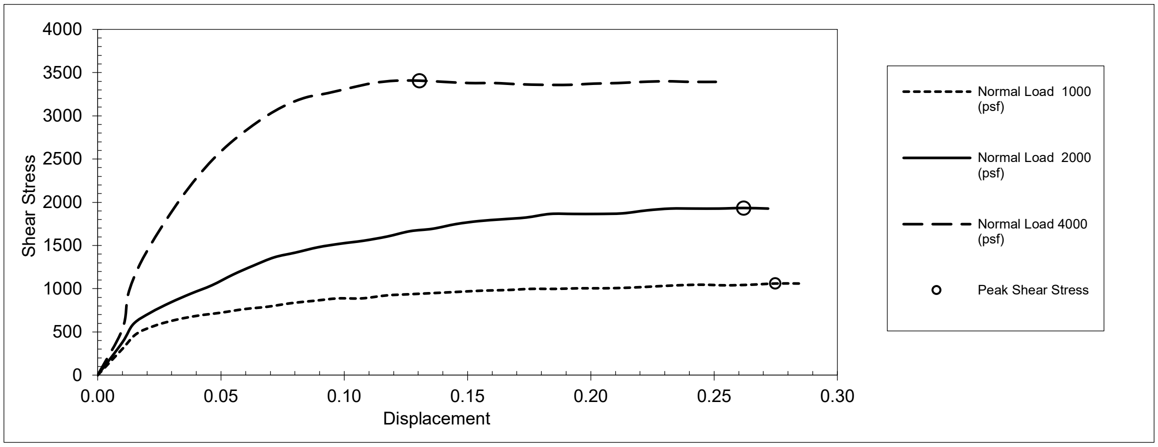
ATTERBERG LIMITS TEST RESULTS

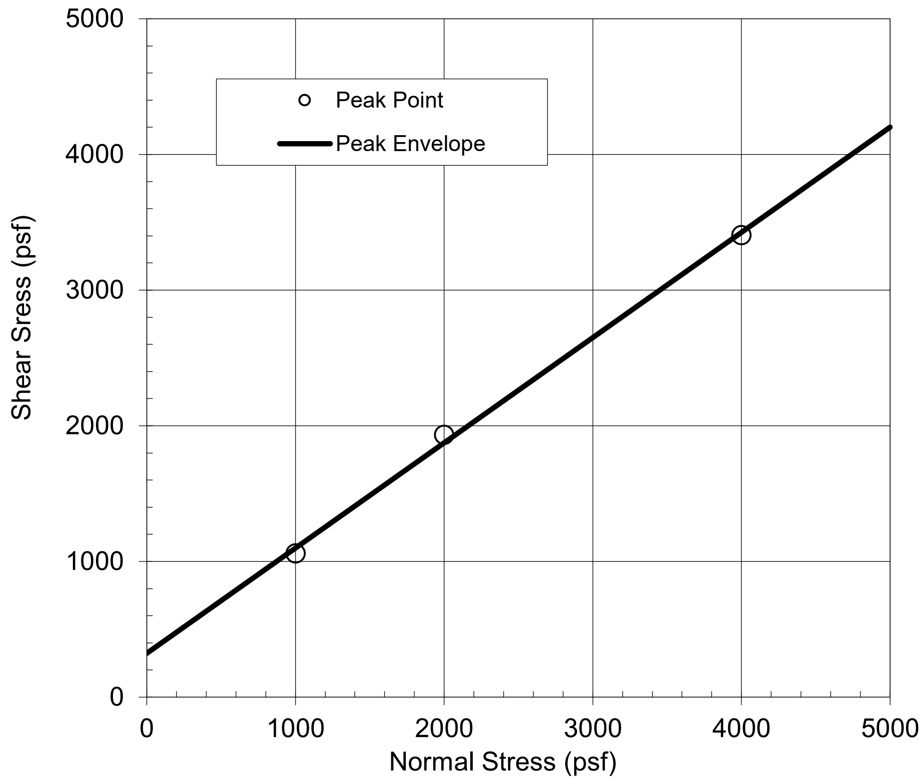
REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-10

Checked by	S.Tena	TECH:Uly P.
PROJECT NO: 20183768.001A		8-May-18



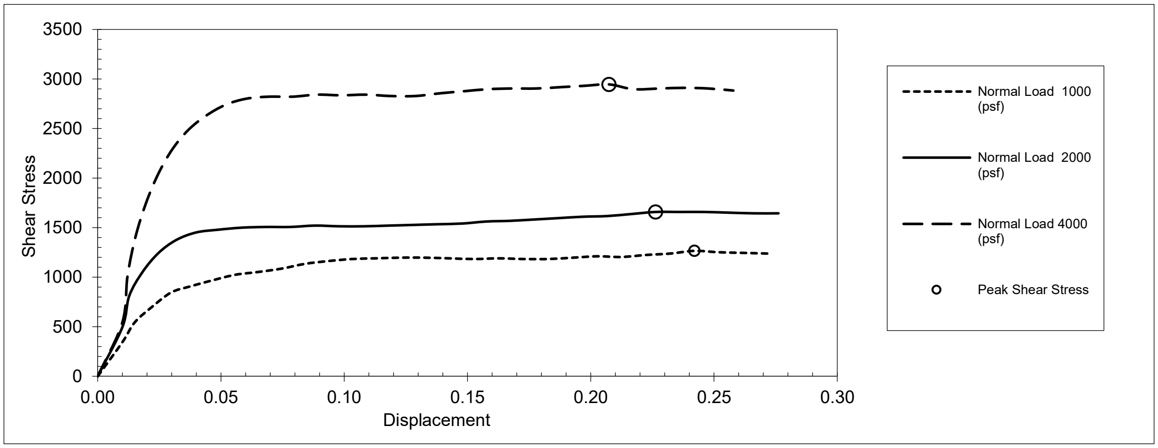


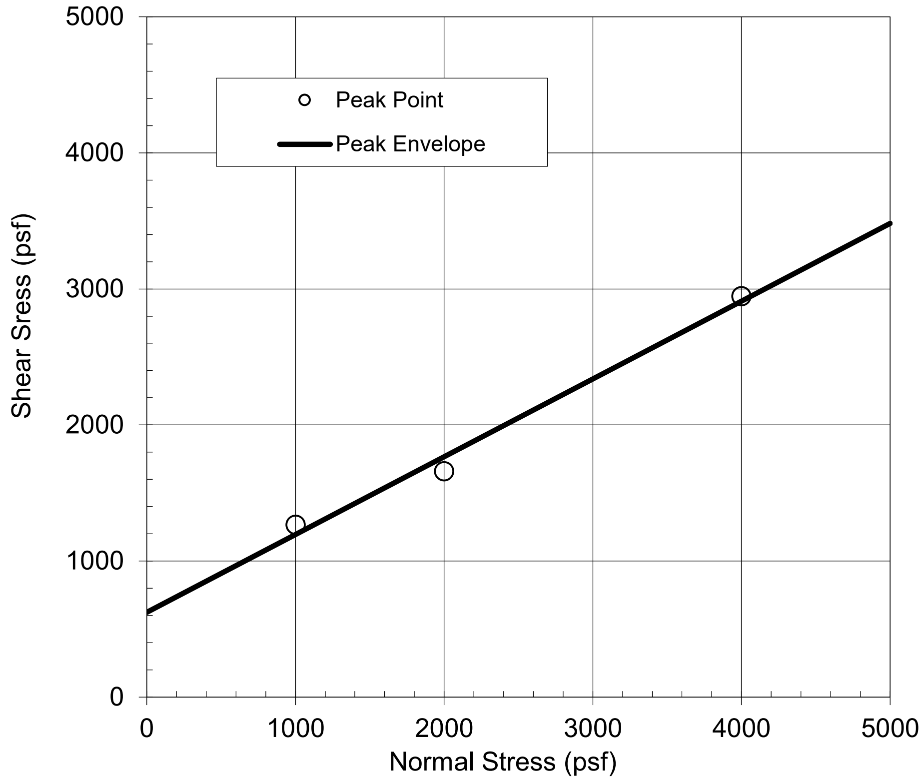
Strain Rate = 0.00975 inch/min
Date Tested: 4/25/2018

				Interpreted Shear Strength			
				Peak			
Boring No.	Sample No.	Depth	UCSC	Cohesion (psf)	Friction Angle (deg)		
B1	S4	15.5'-16'	ML	323	37.8		

Sample description: sandy silt

KLEINFELDER Bright People. Right Solutions.	**Direct Shear Test Results (ASTM D 3080)**	Figure
Checked By: S.Tena / Tech : Uly P.	**REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA**	**B-11**
Project # 20183768.001A / 9-May-18		





Strain Rate = 0.0118 inch/min
Date Tested: 4/9/2018

				Interpreted Shear Strength			
				Peak			
Boring No.	Sample No.	Depth	UCSC	Cohesion (psf)	Friction Angle (deg)		
B2	S1	0.5'-4'	SC	623	29.8		

Sample description: clayey sand

KLEINFELDER
Bright People. Right Solutions.

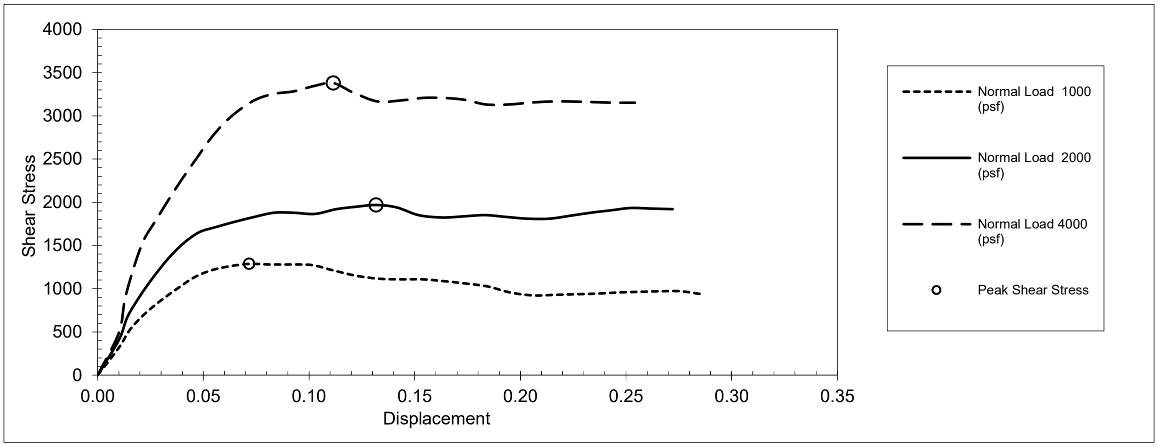
Remolded Direct Shear Test Results
90% Relative Compaction (ASTM D 3080)

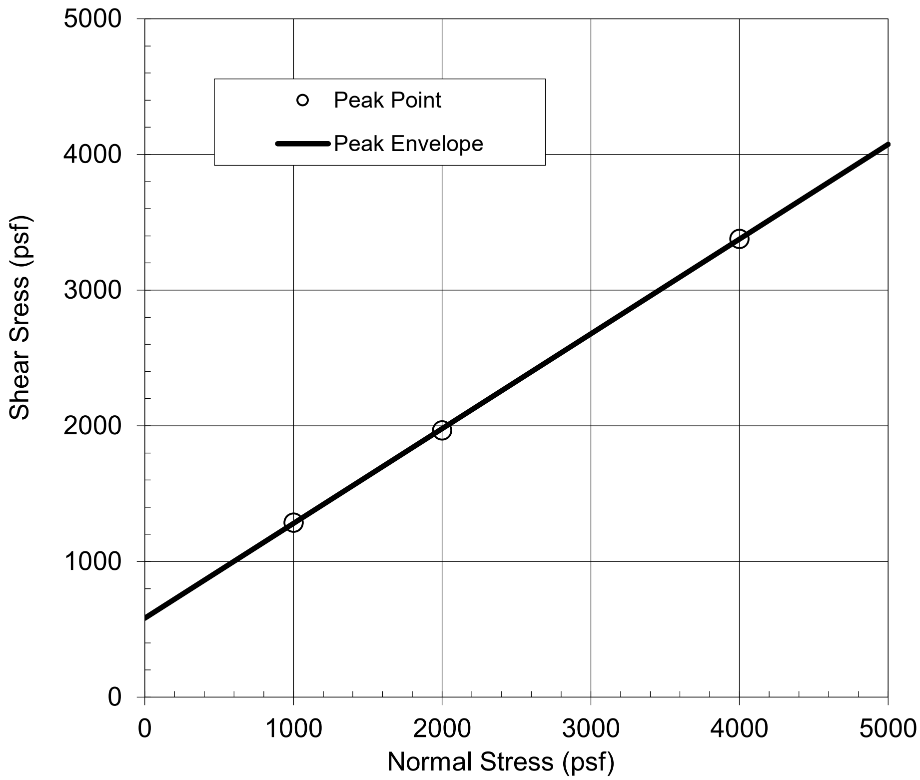
Figure

B-12

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

Checked By:	S.Tena	Tech : Uly P.
Project #	20183768.001A	9-May-18





Strain Rate =	0.00975	inch/min	
Date Tested:	4/26/2018		

				Interpreted Shear Strength			
				Peak			
Boring No.	Sample No.	Depth	UCSC	Cohesion (psf)	Friction Angle (deg)		
B5	S4	15.5'-16'	ML	582	34.9		

Sample description: sandy silt

KLEINFELDER Bright People. Right Solutions.		Direct Shear Test Results (ASTM D 3080)	Figure
Checked By: S.Tena	Tech : Uly P.	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	B-13
Project # 20183768.001A	9-May-18		

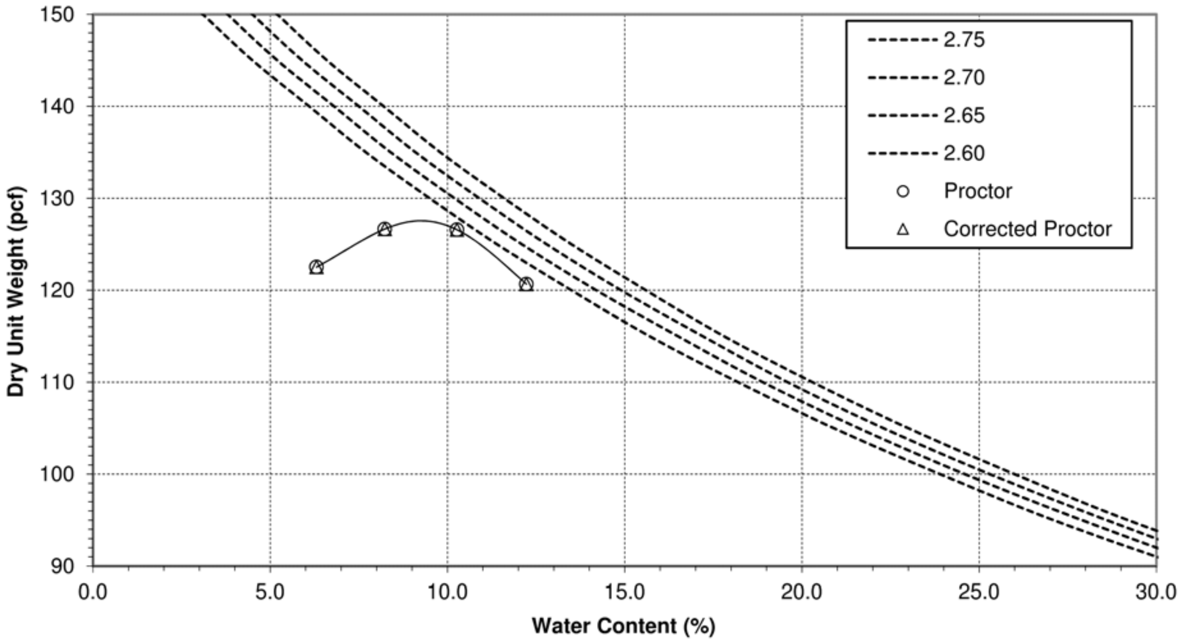
Laboratory Test Report

Client:	**Scripps Health**	Report No.:	**18-SAN-00332 Rev. 0**	Issued:	**5/4/2018**
Project:	**20183768.001A**			Field ID:	**various borings**
	Mercy Replacement Hospital	Sampled by:	**S. Tena**	Date:	**4/4/2018**
	01-000L - Laboratory Testing	Submitted by:	**S. Tena**	Date:	**4/16/2018**

Tested on **4/18/2018** by **C. De La Torre**

Material Description: **dark brown silty sand**

Location: **B2 / S1 at 1'-5'**



Test Method: ASTM D1557 A	Uncorrected	Corrected
Maximum Dry Unit Weight (pcf)	**127.6**	**na**
Optimum Water Content (%)	**9.3**	**na**
Oversize Fraction, retained on (%)		**<5**
Bulk Specific Gravity of Oversize Fraction		**na**

Rammer Type: Mechanical

Specimen Preparation: Moist

Remarks:

Passing No. 4 of material for remolded direct shear.

Reviewed on 5/4/2018 by Ulysses Panuncialman,
Lab Supervisor

LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: May 3, 2018
Purchase Order Number: 20183768.001A
Sales Order Number: 40022
Account Number: KLE
To:
--
Kleinfelder Inc.
550 West C Street Ste 1200
San Diego, CA 92101
Attention: Uly Panuncialman

Laboratory Number: SO6851-1 Customers Phone: 831-4600
Fax: 831-4619
Sample Designation:
--
One soil sample received on 05/02/18 at 12:05pm,
Project: Scripps Mercy
Project #: 20183768.001A
Boring #: B2
Sample #: S1
Depth: 1'-5'
Sampled by S. Tena
Date Sampled 04/04/18

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.0

Water Added (ml)	Resistivity (ohm-cm)
15	3500
5	2800
5	1400
5	1300
5	1100
5	1000
5	1000
5	1200
5	1400

31 years to perforation for a 16 gauge metal culvert.
40 years to perforation for a 14 gauge metal culvert.
55 years to perforation for a 12 gauge metal culvert.
70 years to perforation for a 10 gauge metal culvert.
86 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.132% (1320ppm)
Water Soluble Chloride	Calif. Test 422	0.001% (11ppm)

Laura Torres
LT/ram

FIGURE B-15

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N L A B O R A T O R Y A N D S U P P L Y I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L A N D C O N S U L T I N G C H E M I S T S

Date: May 4, 2018
Purchase Order Number: 20183768.001A
Sales Order Number: 40022
Account Number: KLE
To:
--
Kleinfelder Inc.
550 West C Street Ste 1200
San Diego, CA 92101
Attention: Uly Panuncialman

Laboratory Number: SO6851-2   Customers Phone: 831-4600
Fax: 831-4619

Sample Designation:
--
One soil sample received on 05/02/18 at 12:05pm,
Project: Scripps Mercy
Project #: 20183768.001A
Boring #: Combined of B4/S4, B1/S4 & B1/S5
Sample #: None
Depth: 15'-16.5'
Sampled by S. Tena
Date Sampled 04/04/18.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 9.1

Water Added (ml)	Resistivity (ohm-cm)
10	5100
5	3300
5	2400
5	2100
5	1800
5	1800
5	2000
5	2200

39 years to perforation for a 16 gauge metal culvert.
51 years to perforation for a 14 gauge metal culvert.
70 years to perforation for a 12 gauge metal culvert.
89 years to perforation for a 10 gauge metal culvert.
109 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.008% (84ppm)
Water Soluble Chloride	Calif. Test 422	0.005% (53ppm)

Rosa M. Bernal
RMB/ram

APPENDIX C

Geophysical Survey

SOUTHWEST
GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

April 23, 2018
Project No. 118143

Mr. Salvador Tena
Kleinfelder, Inc.
550 West C Street, Suite 1200
San Diego, CA 92101

Subject: Geophysical Evaluation
4077 5th Avenue
San Diego, California

Dear Mr. Tena:

In accordance with your authorization, we have performed geophysical survey services pertaining to the Mercy Replacement Hospital – Scripps Mercy Hospital project site located at 4077 5th Avenue in San Diego, California (Figure 1). The purpose of our survey was to develop Shear-wave velocity profiles to be used for design and construction of site improvements. Our services were performed on April 2nd, 2018. This report presents the survey methodology, equipment used, analysis, and findings from our study.

Our scope of services included the performance of two refraction microtremor (ReMi) profiles (RL-1 and RL-2) at preselected areas of the project site (see Figures 2 and 3). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a Shear-wave velocity profile of the study area down to a depth, in this case, of approximately 125 feet. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The results of the ReMi method are displayed as a one-dimensional sounding, which represents the average condition across the length of the line. The ReMi method does not require an increase of material velocity with depth; therefore, low velocity zones (velocity inversions) are detectable with ReMi.

Our ReMi survey included the use of a 24-channel Geometrics Geode seismograph and 24 4.5-Hz vertical component geophones. The geophones along RL-1 were spaced 8 feet apart for a total line length of 184 feet. The geophone spacing for RL-2 was 10 feet for a total line length of 230 feet. Fifteen records, each 32 seconds long, were recorded and then downloaded to a computer. The data were later processed using SeisOpt® ReMi™ software (© Optim LLC, 2005), which uses the refraction microtremor method (Louie, 2001). The program generates phase-velocity dispersion curves for each record and provides an interactive dispersion modeling tool where the users determine the best fitting model. The result is a one-dimensional Shear-wave velocity model of the site with roughly 85 to 95 percent accuracy.

Figures 4a and 4b as well as Table 1 present the results from our survey. Based on our analysis of the collected data, the average characteristic site Shear-wave velocity down to a depth of 100 feet is 1,554 feet per second (ft/s) for RL-1 and 1,919 ft/s for RL-2 (CBC, 2016). These values correspond to a site classification of **C**. It should be noted that the ReMi results represent the average condition across the length of the line.

TABLE 1
ReMi Results

Line No.	Depth (feet)	Shear Wave Velocity (feet/second)
RL-1	0-12	748
	12-23	866
	23-34	1,314
	34-46	2,053
	46-125	2,542
RL-2	0-14	872
	14-38	2,336
	38-80	2,392
	80-123	2,326
	123-125	2,299

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

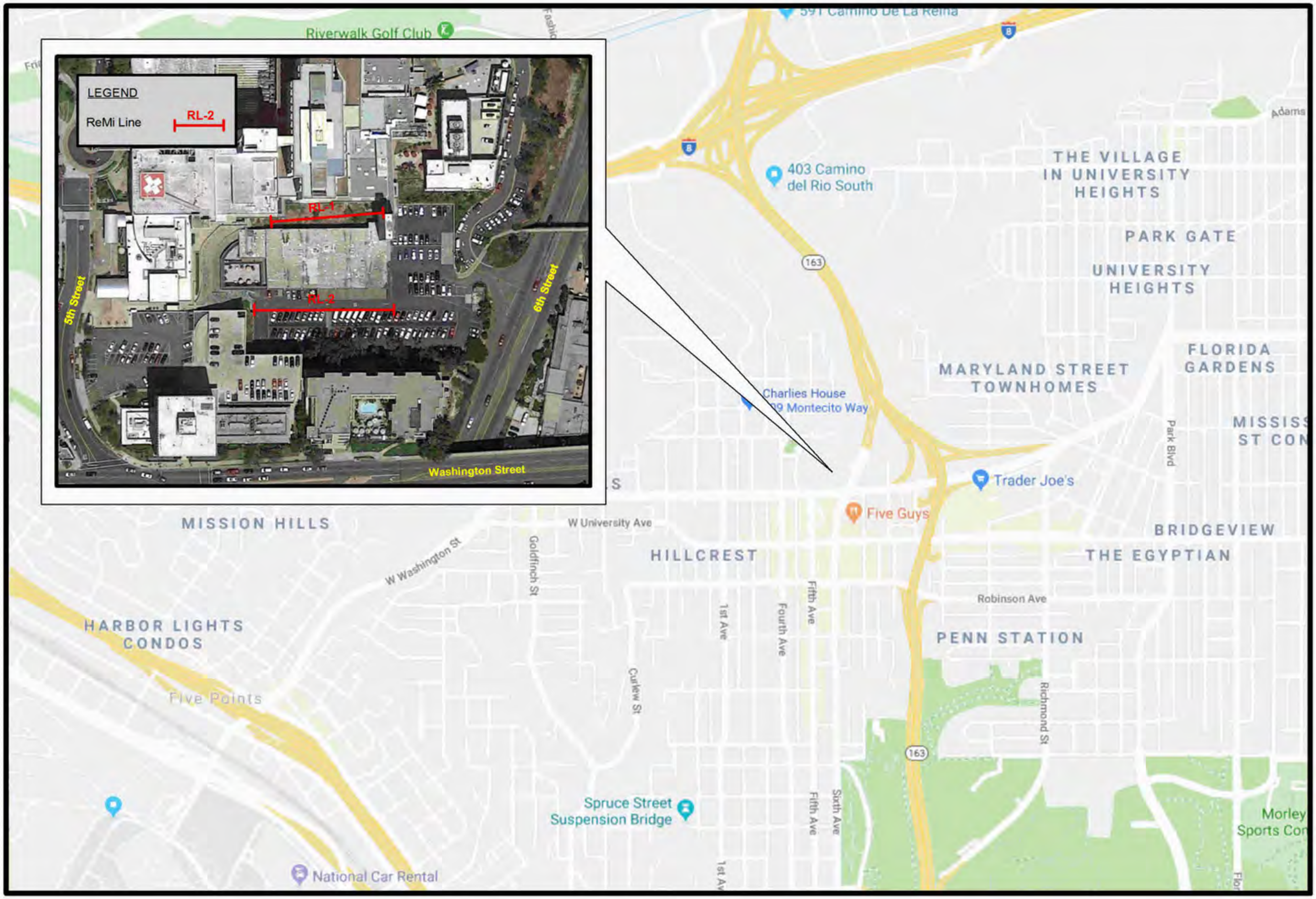
Evan Lavery
Staff Geologist/Geophysicist

Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

EML/AIS/HV/hv

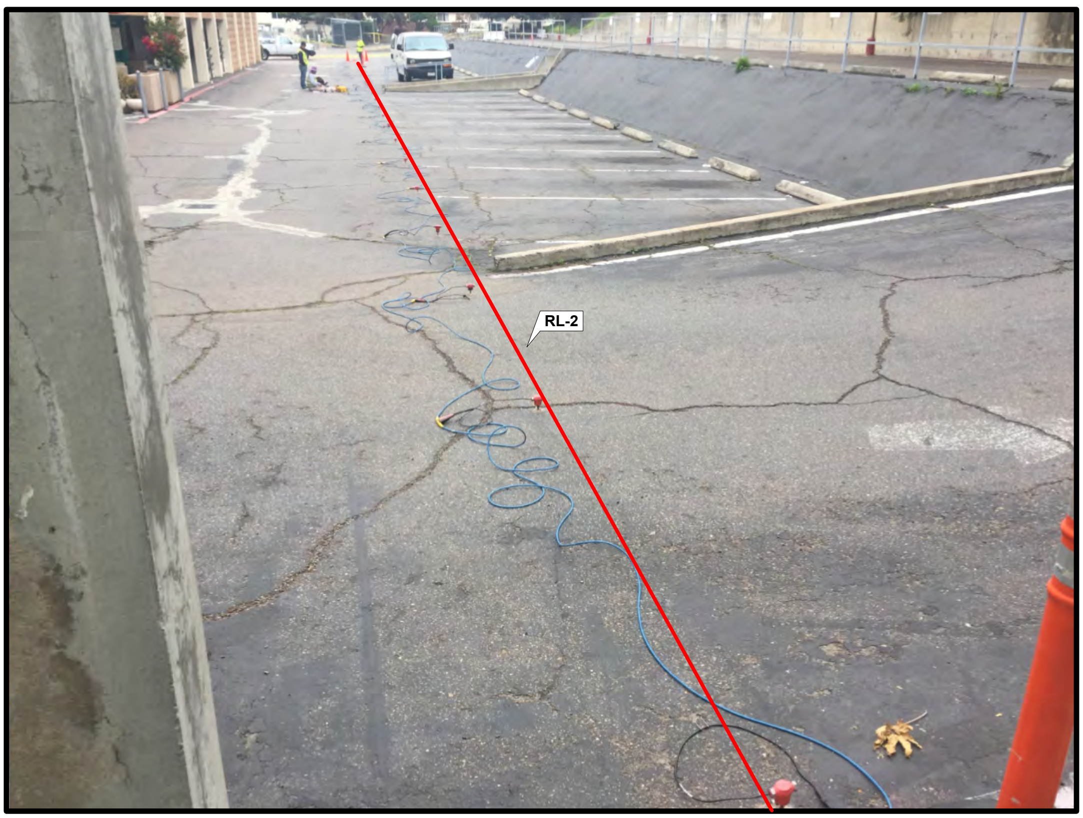
Attachments: Figure 1 – Site Location Map
Figure 2 – Line Location Map
Figure 3 – Site Photographs
Figure 4a – ReMi Results, RL-1
Figure 4b – ReMi Results, RL-2

Distribution: Addressee (electronic)

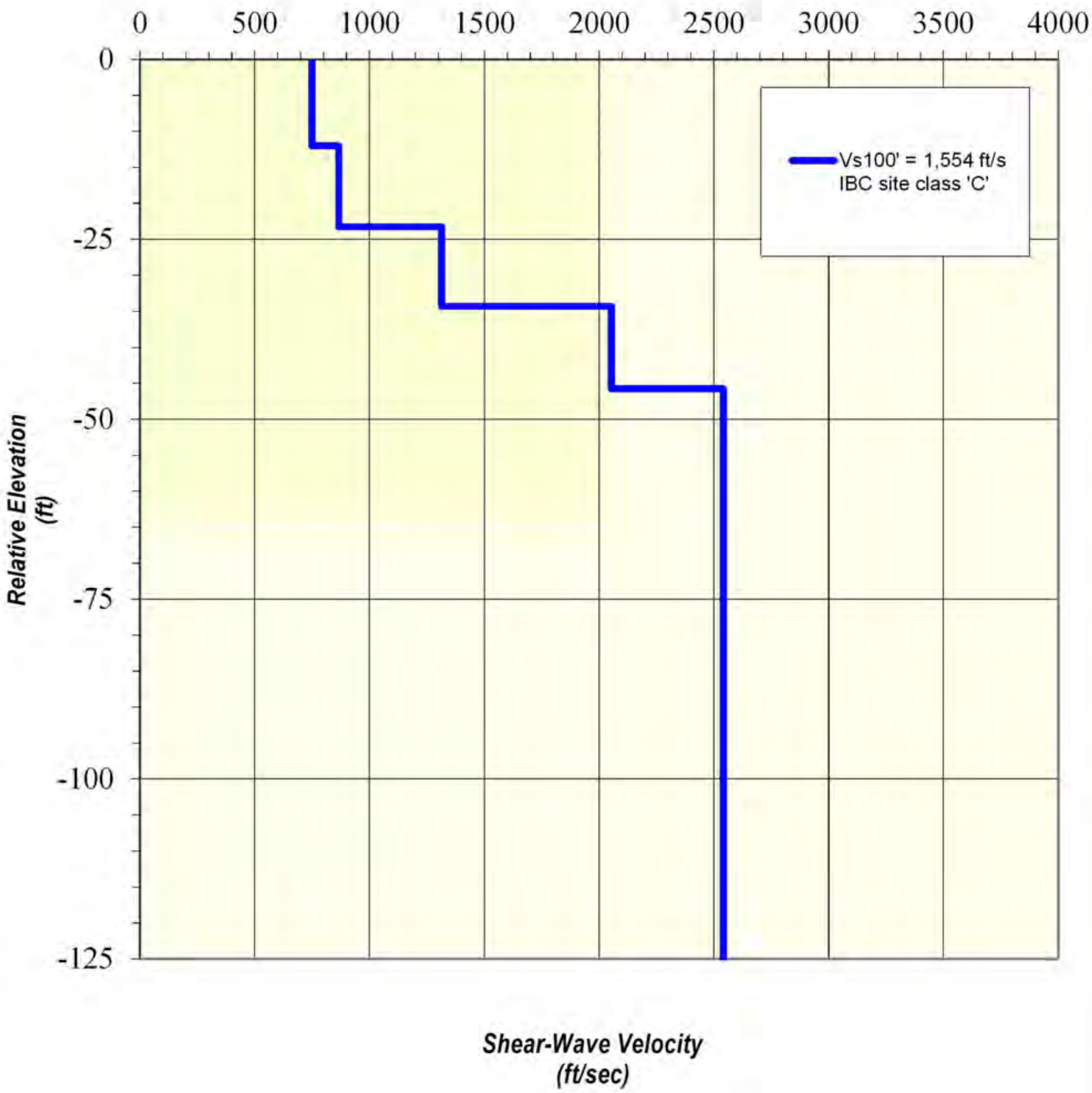


SITE LOCATION MAP	N	4077 5th Avenue San Diego, California Project No.: 118143 Date: 04/18	SOUTHWEST GEOPHYSICS INC. Figure 1

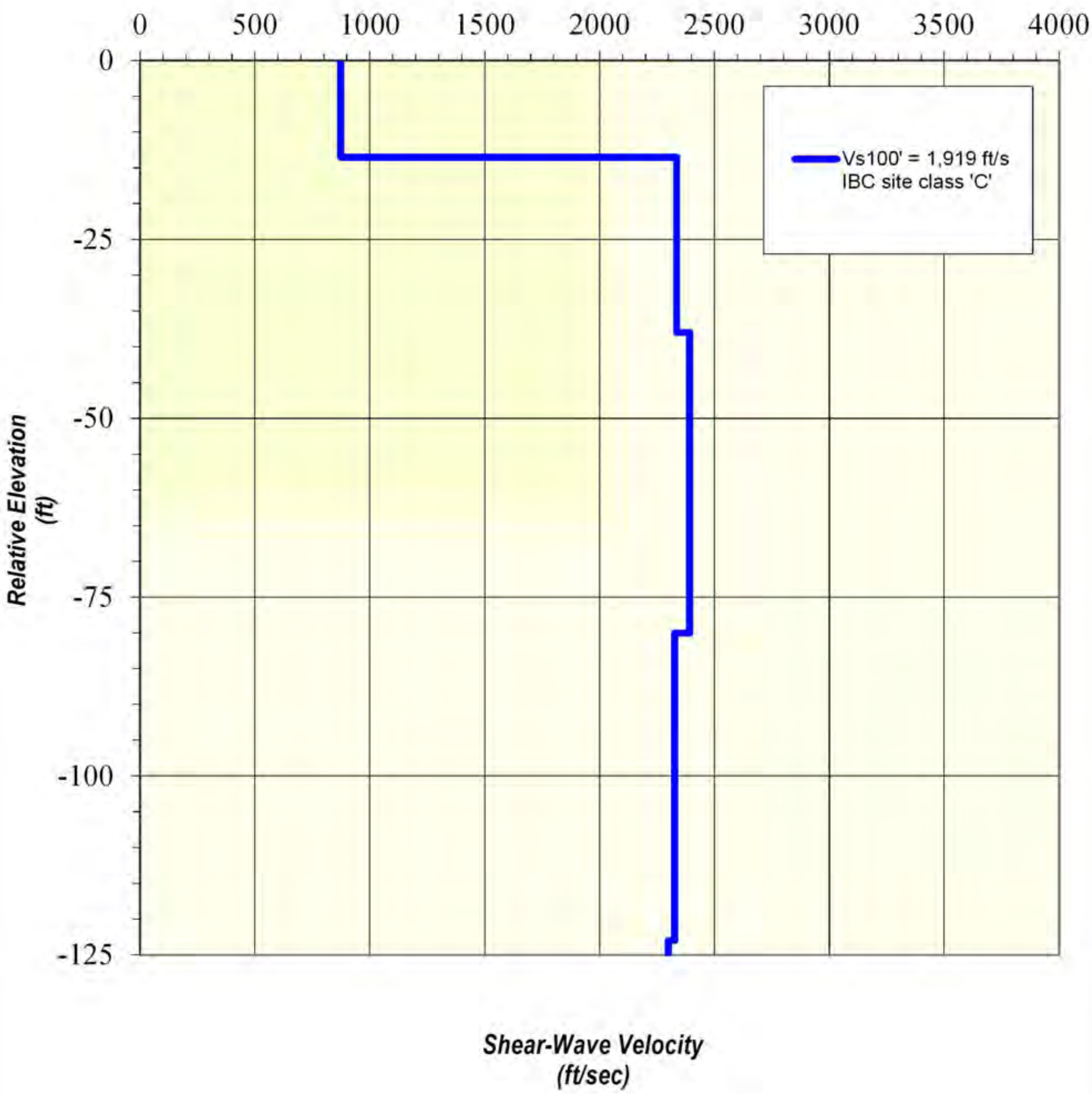
LINE LOCATION MAP	N	4077 5th Avenue San Diego, California	SOUTHWEST GEOPHYSICS INC. Figure 2	0 100 200 **approximate scale in feet**
		Project No.: 118143 Date: 04/18		



SITE PHOTOGRAPHS	4077 5th Street San Diego, California	SOUTHWEST GEOPHYSICS INC. Figure 3
	Project No.: 118143 / Date: 04/18	



ReMi RESULTS RL-1	4077 5th Avenue San Diego, California		SOUTHWEST GEOPHYSICS INC. Figure 4a
	Project No.: 118143	Date: 04/18	



ReMi RESULTS RL-2	4077 5th Avenue San Diego, California	SOUTHWEST GEOPHYSICS INC. Figure 4b
	Project No.: 118143 / Date: 04/18	

APPENDIX D

Infiltration Study

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
		Planning Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ⊙ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ○ Yes; continue to Step 1F. ○ No; conduct appropriate number of tests.	
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ○ Yes; continue to Step 1G. ○ No; select appropriate factor of safety.	
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ○ Yes; answer "Yes" to Criteria 1 Result. ○ No; answer "No" to Criteria 1 Result.	
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ○ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ◉ No; full infiltration is not required. Skip to Part 1 Result.	
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. One borehole percolation test was performed and the unfactored infiltration rate was 0.22 inches per hour. The soils at the location were described as silty sand with gravel.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	○Yes	○No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○Yes	○No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○Yes	○No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○Yes	○No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○Yes	○No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	○ Yes	● No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Summarize findings and basis; provide references to related reports or exhibits.

One borehole percolation test was performed and the unfactored infiltration rate was 0.22 inches per hour. The soils at the location were described as silty sand with gravel.

Part 1 Result – Full Infiltration Geotechnical Screening[4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	○ Full infiltration Condition ◉ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
		Planning Phase
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⦿ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ⦿ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ○ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ⦿ Yes; Continue to Criteria 4. ○ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). One borehole percolation test was performed and the unfactored infiltration rate was 0.22 inches per hour.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]		
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	◉ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◉ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◉ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◉ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◉ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	◉ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	◉ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	◉ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	◉ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	◉ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	◉ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits. One borehole percolation test was performed and the unfactored infiltration rate was 0.22 inches per hour.		
Part 2 – Partial Infiltration Geotechnical Screening Result[5]		**Result**
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.		◉ Partial Infiltration Condition ○ No Infiltration Condition

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

Percolation Test Data Sheet

Project:		Tested By:	Salvador Tena
	Mercy Replacement Hospital	Date:	4/27/2018
Project No:	20183768.001A	Checked By:	

Borehole ID:	**PERC-1**	
Depth of Borehole:	5	feet
Diameter of Borehole:	8	inches
USCS Soil Classification:	SM	
PVC Pipe Hieght above Surface	0	ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	8:48	9:18	30	1.87	1.93	0.06	41.67
2	9:18	9:48	30	1.93	2.00	0.07	35.71
3	9:48	10:18	30	2.00	2.15	0.15	16.67
4	10:18	10:48	30	2.15	2.46	0.31	8.06
5	10:48	11:18	30	1.95	1.97	0.02	125.00
6	11:18	11:48	30	1.97	2.09	0.12	20.83
7	11:48	12:18	30	2.09	3.25	1.16	2.16
8	12:18	12:48	30	1.85	1.92	0.07	35.71
9	12:48	13:18	30	1.92	1.99	0.07	35.71
10	13:18	13:48	30	1.99	2.10	0.11	22.73
11	13:48	14:18	30	2.10	3.24	1.14	2.19
12	14:18	14:48	30	2.03	2.20	0.17	**14.71**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t \, (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	35.64	inches
Hf	33.60	inches
ΔH	2.04	inches
Havg	34.62	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.22**	in/hr

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
		Planning Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ○ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ⦿ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ⦿ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ⦿ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ○ Yes; continue to Step 1F. ○ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ○ Yes; continue to Step 1G. ○ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ○ Yes; answer "Yes" to Criteria 1 Result. ○ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ◉ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ○ No; full infiltration is not required. Skip to Part 1 Result.
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. One borehole percolation test was performed and the unfactored infiltration rate was 2.47 inches per hour.	

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	◉ Yes	○ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◉ Yes	○ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◉ Yes	○ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◉ Yes	○ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◉ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	◉ Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	◉ Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	◉ Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	◉ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	◉ Yes	○ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	◉ Yes	○ No

Summarize findings and basis; provide references to related reports or exhibits.

One borehole percolation test was performed and the unfactored infiltration rate was 2.47 inches per hour.

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	◉ Full infiltration Condition ○ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
		Planning Phase
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ○ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ○ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	○ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.		
Part 2 – Partial Infiltration Geotechnical Screening Result[5]		Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.		◯ Partial Infiltration Condition ◯ No Infiltration Condition

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

Factor of Safety and Design Infiltration Rate Worksheet					Worksheet D.5-1: Form I-9
Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
A	Suitability Assessment	Soil assessment methods	0.25	2	0.50
		Predominant soil texture	0.25	3	0.75
		Site soil variability	0.25	2	0.50
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			2.00
B	Design	Level of pretreatment/ expected sediment loads	0.5	1	0.50
		Redundancy/resiliency	0.25	2	0.50
		Compaction during construction	0.25	1	0.25
		Design Safety Factor, $S_B = \Sigma p$			1.25
Combined Safety Factor, $S_{total} = S_A \times S_B$ [Minimum of 2 and Maximum of 9]					2.50
Observed Infiltration Rate, inch/hr., $K_{observed}$ (corrected for test-specific bias) Note: This worksheet is only applicable when the observed infiltration rate is greater than or equal to 1 inch/hr.					2.50
Design Infiltration Rate, in/hr., $K_{design} = K_{observed} / S_{total}$ Note: If the estimated design infiltration rate is less than or equal to 0.5 inch/hr. then the applicant may choose to implement partial infiltration BMPs.					1.00
Supporting Data					
Briefly describe infiltration test and provide reference to test forms:					

Note: Worksheet D.5-1: Form I-9 is only applicable to design BMPs in "full infiltration condition". This form is not applicable for categorization of infiltration feasibility (Worksheet C.4-1: Form I-8) and/or for designing BMPs in "partial infiltration condition" or "no infiltration condition".

Percolation Test Data Sheet

Project:		Tested By:	Salvador Tena
	Mercy Replacement Hospital	Date:	4/27/2018
Project No:	20183768.001A	Checked By:	

Borehole ID:	**PERC-2**	
Depth of Borehole:	5	feet
Diameter of Borehole:	8	inches
USCS Soil Classification:	SM	
PVC Pipe Hieght above Surface	0	ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	9:45	9:55	10	2.32	3.60	1.28	0.65
2	9:55	10:05	10	2.18	2.96	0.78	1.07
3	10:05	10:15	10	2.06	2.77	0.71	1.17
4	10:15	10:25	10	2.14	2.85	0.71	1.17
5	10:25	10:35	10	2.05	2.97	0.92	0.91
6	10:39	10:49	10	2.10	3.12	1.02	0.82
7	10:49	10:59	10	2.01	2.60	0.59	1.41
8				0.00	0.00	0	NA
9				0.00	0.00	0	NA
10				0.00	0.00	0	NA
11				0.00	0.00	0	NA
12				0.00	0.00	0	**NA**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t \, (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	35.88	inches
Hf	28.80	inches
ΔH	7.08	inches
Havg	32.34	inches
Δt	10.00	minutes
r	4.00	inches
It	**2.47**	in/hr

Soil Map—San Diego County Area, California

Map Scale: 1:3,380 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 50, 100, 200, 300

Feet: 0, 150, 300, 600, 900

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 12, Sep 13, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 7, 2014—Jan 4, 2015

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
TeF	Terrace escarpments	18.9	43.7%
Ur	Urban land	24.5	56.3%
Totals for Area of Interest		**43.4**	**100.0%**

Hydrologic Soil Group and Surface Runoff

This table gives estimates of various soil water features. The estimates are used in land use planning that involves engineering considerations.

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The four hydrologic soil groups are:

Group 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.

Group 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.

Group 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.

Group 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.

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.

Surface runoff refers to the loss of water from an area by flow over the land surface. Surface runoff classes are based on slope, climate, and vegetative cover. The concept indicates relative runoff for very specific conditions. It is assumed that the surface of the soil is bare and that the retention of surface water resulting from irregularities in the ground surface is minimal. The classes are negligible, very low, low, medium, high, and very high.

Report—Hydrologic Soil Group and Surface Runoff

Absence of an entry indicates that the data were not estimated. The dash indicates no documented presence.

Hydrologic Soil Group and Surface Runoff–San Diego County Area, California			
Map symbol and soil name	**Pct. of map unit**	**Surface Runoff**	**Hydrologic Soil Group**
TeF—Terrace escarpments			
Terrace escarpments	100	Very high	—

Hydrologic Soil Group and Surface Runoff–San Diego County Area, California			
Map symbol and soil name	**Pct. of map unit**	**Surface Runoff**	**Hydrologic Soil Group**
Ur—Urban land			
Urban land	100	Very high	—

Data Source Information

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 12, Sep 13, 2017

Engineering Properties

This table gives the engineering classifications and the range of engineering properties for the layers of each soil in the survey area.

Hydrologic soil group is a group of soils having similar runoff potential under similar storm and cover conditions. The criteria for determining Hydrologic soil group is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Listing HSGs by soil map unit component and not by soil series is a new concept for the engineers. Past engineering references contained lists of HSGs by soil series. Soil series are continually being defined and redefined, and the list of soil series names changes so frequently as to make the task of maintaining a single national list virtually impossible. Therefore, the criteria is now used to calculate the HSG using the component soil properties and no such national series lists will be maintained. All such references are obsolete and their use should be discontinued. Soil properties that influence runoff potential are those that influence the minimum rate of infiltration for a bare soil after prolonged wetting and when not frozen. These properties are depth to a seasonal high water table, saturated hydraulic conductivity after prolonged wetting, and depth to a layer with a very slow water transmission rate. Changes in soil properties caused by land management or climate changes also cause the hydrologic soil group to change. The influence of ground cover is treated independently. There are four hydrologic soil groups, A, B, C, and D, and three dual groups, A/D, B/D, and C/D. In the dual groups, the first letter is for drained areas and the second letter is for undrained areas.

The four hydrologic soil groups are described in the following paragraphs:

Group 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.

Group 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.

Group 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.

Group 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.

Depth to the upper and lower boundaries of each layer is indicated.

Texture is given in the standard terms used by the U.S. Department of Agriculture. These terms are defined according to percentages of sand, silt, and clay in the fraction of the soil that is less than 2 millimeters in diameter. "Loam," for example, is soil that is 7 to 27 percent clay, 28 to 50 percent silt, and less than 52 percent sand. If the content of particles coarser than sand is 15 percent or more, an appropriate modifier is added, for example, "gravelly."

Classification of the soils is determined according to the Unified soil classification system (ASTM, 2005) and the system adopted by the American Association of State Highway and Transportation Officials (AASHTO, 2004).

The Unified system classifies soils according to properties that affect their use as construction material. Soils are classified according to particle-size distribution of the fraction less than 3 inches in diameter and according to plasticity index, liquid limit, and organic matter content. Sandy and gravelly soils are identified as GW, GP, GM, GC, SW, SP, SM, and SC; silty and clayey soils as ML, CL, OL, MH, CH, and OH; and highly organic soils as PT. Soils exhibiting engineering properties of two groups can have a dual classification, for example, CL-ML.

The AASHTO system classifies soils according to those properties that affect roadway construction and maintenance. In this system, the fraction of a mineral soil that is less than 3 inches in diameter is classified in one of seven groups from A-1 through A-7 on the basis of particle-size distribution, liquid limit, and plasticity index. Soils in group A-1 are coarse grained and low in content of fines (silt and clay). At the other extreme, soils in group A-7 are fine grained. Highly organic soils are classified in group A-8 on the basis of visual inspection.

If laboratory data are available, the A-1, A-2, and A-7 groups are further classified as A-1-a, A-1-b, A-2-4, A-2-5, A-2-6, A-2-7, A-7-5, or A-7-6. As an additional refinement, the suitability of a soil as subgrade material can be indicated by a group index number. Group index numbers range from 0 for the best subgrade material to 20 or higher for the poorest.

Percentage of rock fragments larger than 10 inches in diameter and 3 to 10 inches in diameter are indicated as a percentage of the total soil on a dry-weight basis. The percentages are estimates determined mainly by converting volume percentage in the field to weight percentage. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Percentage (of soil particles) passing designated sieves is the percentage of the soil fraction less than 3 inches in diameter based on an ovendry weight. The sieves, numbers 4, 10, 40, and 200 (USA Standard Series), have openings of 4.76, 2.00, 0.420, and 0.074 millimeters, respectively. Estimates are based on laboratory tests of soils sampled in the survey area and in nearby areas and on estimates made in the field. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Liquid limit and *plasticity index* (Atterberg limits) indicate the plasticity characteristics of a soil. The estimates are based on test data from the survey area or from nearby areas and on field examination. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

References:

American Association of State Highway and Transportation Officials (AASHTO). 2004. Standard specifications for transportation materials and methods of sampling and testing. 24th edition.

American Society for Testing and Materials (ASTM). 2005. Standard classification of soils for engineering purposes. ASTM Standard D2487-00.

Report—Engineering Properties

Absence of an entry indicates that the data were not estimated. The asterisk '*' denotes the representative texture; other possible textures follow the dash. The criteria for determining the hydrologic soil group for individual soil components is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Engineering Properties–San Diego County Area, California														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				L-R-H	L-R-H	L-R-H	L-R-H	L-R-H	L-R-H	L-R-H	L-R-H
TeF—Terrace escarpments														
Terrace escarpments	100		0-60	Variable	—	—	—	—	—	—	—	—	—	—
Ur—Urban land														
Urban land	100		0-6	Variable	—	—	—	—	—	—	—	—	—	—

Data Source Information

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 12, Sep 13, 2017

APPENDIX E

Site Specific Ground Motion Hazard Analysis

January 31, 2019
Kleinfelder Project No. 20183768.001A

Mr. Bruce A. Rainey
Corporate Vice President
Scripps Health
10140 Campus Point Drive, Suite 210
San Diego, California 92121

SUBJECT: Technical Memorandum for Supplemental Site-Specific Ground Motion Hazard Analysis
Replacement Hospital Building
Scripps Mercy Hospital Campus
4077 5th Avenue
San Diego, California

Dear Mr. Rainey,

Kleinfelder previously submitted a Draft Geologic and Geotechnical Engineering Evaluation report on June 7, 2018 for the design and construction of the proposed replacement hospital building to be located within the Scripps Mercy Hospital campus at 4077 5th Avenue, San Diego, California. In our report, we provided code-based seismic design parameters for the design of the structure in accordance with Section 1613A of the 2016 California Building Code (CBC). Subsequent to issuing our report, the design team requested that a site-specific ground motion hazard analysis be performed to provide the site-specific response spectra for the proposed project site. This technical memorandum presents the results of our site-specific ground motion hazard analysis, in terms of site-specific response spectra and seismic design parameters, as well as the methodologies and criteria used to perform our ground motion hazard analysis. The results of our site-specific ground motion hazard analysis will also be provided as an appendix to our final geotechnical report to be provided at a future date.

INTRODUCTION

As requested by the project design team and Scripps Health, we performed a site-specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-10 as specified by the 2016 CBC. The seismic ground motion parameters provided in our June 2018 draft report are based on the code in accordance with Section 11.4 of ASCE 7-10 and Section 1613A of the 2016 CBC and are not considered to be site specific.

The results of our current 2018 geotechnical investigation for the proposed hospital replacement building, as well as the results of previous geotechnical investigations performed by Kleinfelder and by others within our project site limits, were utilized for our site-specific ground motion hazard analyses. These investigations included the completion of geophysical surveys consisting of two refraction microtremor (ReMi) surveys performed within our site which resulted in average shear wave velocities in the upper 100 feet (30 meters) of the soil profile (V_{s30}) of 1,554 ft/s (474 m/s) and 1,919 ft/s (585 m/s). Based on the results of the seismic refraction survey and the results of the current and previous field investigations performed at the project site, the site is classified as a Seismic Site Class C. Site Class C is defined as a very dense soil and soft rock profile with V_{s30} values between 1,200 and 2,500 ft/sec, an average standard penetration resistance greater than 50 blows/ft, or an average undrained shear strength greater than 2,000 psf. Further details of the current and previous geotechnical investigations performed at the site and used in our analyses are provided in our June 2018 geotechnical report.

PURPOSE AND SCOPE OF SERVICES

The purpose of our site-specific ground motion hazard analyses is to develop site-specific ground motion criteria in terms of site-specific seismic design parameters and response spectral accelerations by utilizing data obtained from the geotechnical investigations performed at the site. Specifically, our scope of services for the site-specific ground motion hazard analyses included the following:

- Review of readily available geologic and seismic data including review of previous geotechnical investigations performed at the site, the regional geologic and seismic setting, faulting locations with respect to the site, and historical seismicity;
- Development of applicable seismic source models, based on the 2016 CBC requirements, accounting for the site's proximity to active faults, data from major historical earthquakes, and regional seismicity;
- Performance of site-specific probabilistic and deterministic seismic hazard analyses (PSHA and DSHA) utilizing seismic source models and applicable ground motion prediction equations (GMPEs). The PSHA and DSHA were performed in accordance with ASCE 7-10 based on the current 2016 CBC requirements;
- Development of site-specific design horizontal response spectra for the Risk-Targeted Maximum Considered Earthquake (MCE_R) and the Design Earthquake (DE) in accordance with Sections 21.2 and 21.3 of the ASCE 7-10 Standard; and
- Preparation of this technical memorandum summarizing our analyses and results.

We will also provide the results of our site-specific ground motion hazard analysis as an appendix to our final geotechnical report for the project to be provided at a future date.

PROJECT LOCATION

The project site is located on the Scripps Mercy Hospital Campus located at 4077 5th Avenue in San Diego, California. We have used center of the proposed replacement hospital building as the site location and the approximate site coordinates used for the ground motion hazard analysis are:

Latitude: 32.7509° N
Longitude: -117.1598° W

REGIONAL FAULTING

According to Hart and Bryant (1997), the site is not located within an Alquist-Priolo Earthquake Fault Zone. Faults located close to the site include the Newport-Inglewood Connected fault (includes Rose Canyon, Newport-Inglewood offshore, and Newport-Inglewood faults) at about

4.3 km, the Rose Canyon fault at about 4.3 km, the Coronado Bank fault at about 23 km, and the Newport-Inglewood Offshore fault at about 52 km from the site. A seismic event on any of these faults could cause significant ground shaking at the site. Figure 1 shows the known faults within 100 km from the site. However, only independent seismogenic sources have been labeled. All other faults have been included in the background seismic sources.

HISTORICAL SEISMICITY

The project site is located in an area characterized by moderate seismic activity. Most of the high seismic activity near the San Diego region is associated with the Elsinore fault, located approximately 65 km northeast of the site, and the San Jacinto fault, located approximately 99 km northeast of the site. A number of large earthquakes have historically occurred surrounding the San Diego region that have been felt by the project site area. Based on our review of available earthquake catalogs, some of the significant nearby events include the November 22, 1800 (M6.5) San Diego Region earthquake, the May 27, 1862 (M6.0) earthquake associated with the Rose Canyon fault, the October 23, 1894 (M5.8) earthquake located east of San Diego County, the July 13, 1986 (M5.8) Oceanside earthquake associated with the San Diego Trough fault, and the July 7, 2010 (M5.4) and June 10, 2016 (M5.2) Borrego Springs earthquakes.

During the recent 2010 (M5.4) and 2016 (M5.2) Borrego Springs earthquakes on the San Jacinto fault, several California Strong Motion Instrumentation Program (CSMIP) stations throughout San Diego County recorded free-field horizontal peak ground accelerations ranging from approximately 0.01 to 0.04g (CESMD, 2019). Epicenters of significant earthquakes (M>4.0) within the vicinity of the site are shown on Figure 1.

SITE-SPECIFIC GROUND MOTION MODELS

Seismic Source Models

The seismic source model used in our site-specific ground motion hazard analysis was developed based on the Uniform California Earthquake Rupture Forecast, Version 2 (UCERF2) used in developing the 2008 Update of the United States National Seismic Hazard Maps (Petersen et al., 2008). Faults within 200 km of the site were used in our ground motion hazard analysis. The faults within 100 km and their seismic parameters are listed in Table 1 and shown on Figure 1. The locations of the faults and associated parameters presented in Table 1 are referenced from Petersen et al. (2008). Where multiple fault rupture scenarios are presented by Petersen et al. (2008), we have listed the scenario involving the maximum number of segments. The earthquake magnitudes presented in this table are based on the moment-area relationships developed by Ellsworth (Type B) (2003) and Hanks and Bakun (2002, 2008). The magnitudes developed for each fault rupture based on the Ellsworth-B and Hanks and Bakun relationships were used in the seismic source model with equal weights to determine the associated rupture magnitude.

Table 1
Seismic Source Model Significant Faults within 100 km of Project Site

Fault Name	Fault Length (km)	Approx. Closest Distance to Site (km)[1]	Ellsworth-B Magnitude	Hanks and Bakun Magnitude	Slip Rate (mm/year)
Rose Canyon	70	2.3	6.9	6.7	1.5
Newport-Inglewood (Connected)	208	2.3	7.5	7.5	1.3
Coronado Bank	180	22	7.4	7.3	3.0
Palos Verdes (Connected)	285	22	7.7	7.7	3
Newport-Inglewood (Offshore)	66	52	7	6.8	1.5
Elsinore (W+GI+T+J+CM)	241	65	7.79	7.85	2.5 - 5
Earthquake Valley	20	72	6.8	6.6	2
Palos Verdes	99	92	7.3	7.2	3
San Jacinto (SBC+SJV+A+CC+B+SM)	241	98	7.8	7.88	4 - 18

[1] Closest distance to potential rupture.

According to Petersen et al. (2008), characterizations of the Elsinore and San Jacinto faults are based on the following fault rupture segments and fault rupture scenarios:

- The Elsinore fault has been characterized by five fault segments and fifteen rupture scenarios. The five segments are the Whittier (W), Glen Ivy (GI), Temecula (T), Julian (J), and Coyote Mountain (CM) segments.
- The San Jacinto fault has been characterized by seven fault segments and twenty-five rupture scenarios. The seven segments are the San Bernardino Valley (SBV), San Jacinto Valley (SJV), Anza (A), Clark (C), Coyote Creek (CC), Borrego (B), and Superstition Mountain (SM) segments.

In addition, the Newport-Inglewood (Connected) fault represents a rupture scenario where the Newport-Inglewood, Newport-Inglewood (offshore), and Rose Canyon faults rupture together. The Palos Verdes (Connected) fault represents a rupture scenario where the Palos Verdes and Coronado Bank faults rupture together.

Magnitude-Frequency Distribution

The earthquake rates for identified fault sources (and their segments, if applicable) were developed using magnitude-frequency models as referenced in UCERF2. In the first model, earthquake recurrence is defined by a constrained form of the Gutenberg-Richter (G-R) magnitude-frequency relationship (Gutenberg and Richter, 1956) given by:

$$\log N = a - bM$$

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where N is the cumulative number of earthquakes of magnitude "M" or greater per year and "a" and "b" are constants based on recurrence analyses derived from seismicity catalogs and fault slip rates. The G-R relationship is truncated at the maximum magnitude earthquake determined by a magnitude-area relationship. This model generally implies that, during the time interval between each large-magnitude earthquake on a particular fault, seismic events of all sizes along that fault occur continually.

The second model, referred to as the Characteristic model (Schwartz and Coppersmith, 1984), implies that faults have a “characteristic behavior” which includes factors such as termination of ruptures at persistent locations and the assumption that the time between large-magnitude earthquakes along a particular fault is generally quiescent with the exception of foreshocks, aftershocks, and low-level background activity.

The 2008 National Seismic Hazard Maps and UCERF2 categorized faults into two types (Type A and Type B faults) and different magnitude-frequency distributions were applied dependent of the type of fault. Type A faults were defined as “well-known” faults that generally have slip rates greater than 5 mm/yr and well-constrained paleoseismic data (e.g. the San Andreas, San Jacinto, and Elsinore faults). Type B faults include all other “lesser-known” faults lacking the paleoseismic data necessary to constrain the recurrence intervals of large events. In accordance with UCERF2 and the 2008 National Seismic Hazard Maps, the Characteristic model is applied to Type A faults. For Type B faults, a weighted average of the Characteristic and G-R models are applied using weights of two-thirds and one-third; respectively, with a minimum magnitude of 6.5 for the G-R model and a maximum magnitude determined by magnitude-area relationships.

G-R model b-values of 0.8 and 0.0, with a one-half weight assigned to each, were generally used for faults in this study. The b-value of 0.8 is based on paleoseismic data of the California region and a b-value of 0.0 is used to reduce over-predicting M6.5 to M7 events due to background seismicity. The most likely a-values were estimated for each seismic source based on the recurrence rates of earthquakes associated with that seismic source as reported by Petersen et al. (2008). It should be noted that aftershocks were removed from seismicity catalogs using the Gardner and Knopoff (1974) method and not included the rate model presented in UCERF2.

Background Seismicity

In addition to the independent seismogenic sources, our seismic analyses also include background seismicity. In general, background seismicity accounts for earthquakes, both on and off identified fault sources, with generally lower magnitudes ranging between M5 and M7 based on the methodology described in Petersen et al. (2008). Some of the local faults in the San Diego region are not modeled as independent seismogenic sources based on UCERF2 and the update for the 2008 National Seismic Hazard Maps; however, the seismicity of these faults was incorporated into our analysis by including background seismicity in our model. The background seismicity hazard is calculated using a-values as determined by Weichert (1980), a b-value of 0.8, minimum magnitude of M5, maximum magnitude of M7, and by applying an exponential distribution as described by Herrmann (1977). Petersen et al. (2008) reduced the rate of background seismicity sources with magnitudes of M6.5 to M7 by one third to account for overlap of earthquakes already modeled on major faults.

Ground Motion Prediction Equations

Site-specific ground motions can be influenced by several factors including types of faulting, earthquake magnitudes, and local soil conditions. Ground motion prediction equations (GMPEs) account for these effects and are used to estimate ground motions at a site resulting from a scenario earthquake.

Three Next Generation Attenuation of Ground Motions West 1 (NGA West 1) relationships for shallow crustal earthquakes were used in developing the 2008 National Seismic Hazard Maps (Petersen et al., 2008) included in ASCE 7-10. For this project, we used the same three NGA West 1 models which include Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008) with equal weights applied for all crustal faults and for background seismicity. All three of these NGA West 1 relationships use estimates of V_{S30} as an input. Based on the locations of the ReMi lines performed at the site, we used an average V_{S30} value of 1,737 feet per second (530 m/s) in our analysis. In addition, some of the GMPEs require inputs for $Z_{1.0}$, defined as the depth to a layer having a shear wave velocity of 1,000 m/s, and $Z_{2.5}$, defined as the depth to a layer having a shear wave velocity of 2,500 m/s, which are intended to capture basin effects on site response. For these parameters, we utilized correlations developed for the NGA West 1 project to determine best estimate depths of $Z_{1.0}$ and $Z_{2.5}$ (Chiou and Youngs, 2008; Campbell and Bozorgnia, 2007). Based on these NGA West 1 correlations, we used a $Z_{1.0}$ value of 90.6 meters and a $Z_{2.5}$ value of 1.13 km.

The GMPEs provide geometric mean values of the two horizontal components of ground motions associated with varying parameters such as magnitude, distance to rupture, site soil conditions, and mechanism of faulting. The uncertainty in the predicted ground motions is taken into consideration by including a magnitude-dependent standard error in the analyses.

SITE-SPECIFIC SEISMIC HAZARD ANALYSES

Probabilistic Seismic Hazard Analysis

A probabilistic seismic hazard analysis (PSHA) procedure was used to estimate the spectral ground motions corresponding to a 2 percent probability of exceedance in 50 years in accordance with 2016 CBC and ASCE 7-10 Standard requirements. The PSHA approach is based on earthquake characteristics and the associated causative faults. These characteristics include such parameters as magnitude, distance to the fault rupture, and the length and activity of the fault. The effects of site soil conditions and mechanism of faulting are accounted for in the GMPEs used for the site.

The theory behind PSHA is based on the "total probability theorem" and on the assumption that earthquakes are events that are independent of time and space from one another (Cornell, 1968, 1971; Merz and Cornell, 1973). According to this approach, the probability of exceeding a given level of ground motion, Z, at the site within a specified time period, T, is given by the following equation:

$$PE(Z) = 1 - e^{-\vartheta(Z)T}$$

where $\vartheta(Z)$ is the mean annual rate of exceedance of ground motion level Z. Different probabilities of exceedance may be selected depending on the level of risk and performance required.

PSHA can be explained through a four-step procedure as follows:

- Identification and characterization of seismic sources and the probability distribution of potential rupture within the sources. Usually, uniform probability distributions are assigned to each source or rupture location. The probability distribution of the site distance to the rupture is obtained by combining potential rupture distributions with the source geometry.
- Characterization of seismicity distribution of earthquake recurrence. An earthquake recurrence relationship such as Gutenberg-Richter is used to characterize the seismicity distribution of each source.
- Use of GMPEs in assessing the ground motions produced at the site by considering the applicable seismic sources and the distance of the sources to site. The variability of attenuation relationships, the effects of site soil conditions, and the mechanism of faulting are accounted for in the PSHA analysis.
- Combination of all of these uncertainties to obtain the probability of ground motion exceedance during a particular time period.

A simplified mathematical expression of these steps is provided by the following:

$$\nu\ (Sa > z) = \sum_{i=1}^{Nsource} N_i(M_{\min}) \int_{r=0}^{\infty} \int_{m=M_{\min}}^{M_{\max_i}} f_{m_i}(M) f_{r_i}(r) P(Sa > z \mid M, r) dr dM$$

where $\nu(Sa>z)$ is the mean annual rate of a spectral acceleration (Sa) exceeding a test value (z); N_{source} is the number of seismic sources; $N_i(M_{min})$ is the rate of earthquakes with magnitude greater than M_{min} on the i^{th} seismic source; $f_{m,i}(M)$ is the probability distribution of earthquake magnitude (M) of the i^{th} source; $f_{r,i}(r)$ is the probability distribution of the fault rupture location (r); and P(Sa>z|M,r) is the probability that Sa is greater than the test value (z) given the magnitude, M, and distance to rupture, r.

We have used the computer program EZ-FRISK version 8.00 (Risk Engineering, 2017) for our probabilistic seismic hazard analysis. Elastic horizontal response spectral values for a probability of exceedance of 2 percent in 50 years, assuming a damping factor of 5 percent, were calculated using the probabilistic analysis approach described above.

Deterministic Seismic Hazard Analysis

Deterministic seismic hazard analysis (DSHA) is based on the characteristics of the controlling earthquake and of the causative fault associated with the controlling earthquake. These characteristics include such items as magnitude and distance from the site to the causative fault. The effects of local soil conditions and mechanism of faulting are accounted for in the GMPEs used in the analysis. Per the ASCE 7 Standard, the 84^{th}-percentile (i.e. median plus one standard deviation) deterministic spectral acceleration values are to be used.

The deterministic procedure was used to estimate the 84^{th}-percentile, 5-percent damped elastic response spectral accelerations for the controlling earthquake at the site. The controlling earthquake is the M6.9 event along the Rose Canyon fault located approximately 2.3 km (1.4 miles) west of the site. Although the Newport-Inglewood Connected fault is at the same distance and has a higher earthquake magnitude than the Rose Canyon fault, we believe that due to a long recurrence interval associated with the Newport-Inglewood Connected rupture, the Rose Canyon fault with M6.9 is the controlling fault for deterministic evaluation.

DETERMINATION OF SITE-SPECIFIC HORIZONTAL MCE_R and DE RESPONSE SPECTRA

To develop the site-specific spectral response accelerations, we first obtained the code-based seismic design parameters based on the site class, site coordinates, and the risk category of the building using the Office of Statewide Health Planning and Development (OSHPD) online tool (https:/seismicmaps.org). These values are summarized in Table 2.

Table 2
Code-Based Ground Motion Parameters Based on ASCE 7-10

Parameter	Value	ASCE 7-10 Reference
S_S	1.184g	Fig 22-1
S_1	0.453g	Fig 22-2
Site Class	C	Table 20.3-1
F_a	1.00	Table 11.4-1
F_v	1.344	Table 11.4-2
C_{RS}	0.854	Fig 22-3
C_{R1}	0.892	Fig 22-4
S_{MS}	1.184g	Eq. 11.4-1
S_{M1}	0.613g	Eq. 11.4-2
S_{DS}	0.790g	Eq. 11.4-3
S_{D1}	0.409g	Eq. 11.4-4
PGA	0.525g	Fig 22-7
F_{pga}	1.00	Table 11.8-1
PGA_M	0.525g	Eq. 11.8-1

The site-specific MCE_R response spectrum is developed by comparing probabilistic, deterministic, deterministic lower limit (DLL), and 80 percent of the code values. The NGA West 1 GMPEs present the spectral accelerations in terms of geometric mean values of the rotated two horizontal ground motions. To estimate both the deterministic and probabilistic spectral accelerations in the direction of the maximum horizontal response at each period from geometric mean values, we have used the scale factors as used by USGS. To obtain spectral acceleration values in the maximum direction, a factor of 1.1 for periods of 0.2 seconds and less and a factor of 1.3 for periods of 1.0 second and greater were used. Linear interpolation between 1.1 and 1.3 was used for periods between 0.2 seconds and 1.0 second. In addition, the probabilistic spectrum was adjusted for targeted risk using risk coefficients C_{RS} and C_{R1}. C_{RS} and C_{R1} were estimated from Figures 22-3 and 22-4 of ASCE 7-10 to be 0.854 and 0.892, respectively. C_{RS} is applied to spectral acceleration values for periods of 0.2 seconds and less and C_{R1} is applied for periods of 1.0 second and greater. Linear interpolation of the C_{RS} and C_{R1} values was used for periods between 0.2 seconds and 1.0 second.

Site-specific deterministic (84th-percentile) spectrum for the Rose Canyon fault is compared with the DLL spectrum per Figure 21.2-1 of ASCE 7-10 as shown on Figure 2 and in Table 3 for some specific periods. The controlling deterministic values are taken as the maximum of the 84th-percentile site-specific deterministic spectrum or the DLL spectrum. Figure 2 and Table 3 show that the controlling deterministic spectrum is governed by the 84th-percentile site-specific deterministic values for all periods used in our analysis.

Table 3
Comparison of Spectral Accelerations (g) based on ASCE 7-10

Period (s)	Deterministic Max Rot	DLL	Probabilistic Max Rot Risk Adj	DE	80% Code DE
PGA (0.01)	0.874	0.600	0.538	0.359	0.245
0.2	2.105	1.500	1.220	0.814	0.631
0.3	2.054	1.500	1.167	0.778	0.631
0.5	1.747	1.500	1.002	0.668	0.631
1.0	1.094	0.780	0.630	0.420	0.327
2.0	0.515	0.390	0.298	0.198	0.163

Site-specific probabilistic spectrum is compared with the controlling deterministic spectrum as shown on Figure 3 and in Table 3 for some specific periods. Figure 3 and Table 3 show that the probabilistic values are less than the controlling deterministic values for all periods in our analysis. Therefore, site-specific MCE_R spectrum is governed by the probabilistic spectrum for all periods. The DE spectrum was developed by taking two-thirds of the MCE_R spectrum. Comparison of the DE spectrum with the 80% of the code spectrum is shown on Figure 4 and in Table 3 for some specific periods. Figure 4 and Table 3 show that the DE spectrum is higher than the 80% of the code spectrum for all periods. Therefore, the recommended horizontal DE spectrum is governed by the site-specific DE spectrum. Site-specific MCE_R spectrum is taken as 1.5 times the DE spectrum. Figure 5 shows the recommended site-specific, 5%-damped DE and MCE_R spectra. Recommended site-specific horizontal spectral acceleration values in units of g for the DE and MCE_R are presented in Table 4.

Table 4
Recommended Site-Specific Horizontal MCE_R and DE Spectral Accelerations (g) based on ASCE 7-10

Period (sec)	DE	MCE_R
	5% Damping	5% Damping
0.01	0.359	0.538
0.020	0.367	0.550
0.030	0.394	0.592
0.050	0.471	0.707

Table 4 (continued)
Recommended Site-Specific Horizontal MCE_R and DE Spectral Accelerations (g) based on ASCE 7-10

Period (sec)	DE	MCE_R
	5% Damping	5% Damping
0.075	0.590	0.885
0.1	0.677	1.015
0.15	0.781	1.171
0.2	0.814	1.220
0.25	0.802	1.202
0.3	0.778	1.167
0.4	0.732	1.098
0.5	0.668	1.002
0.75	0.514	0.771
1	0.420	0.630
1.5	0.279	0.419
2	0.198	0.298
3	0.120	0.179
4	0.084	0.126
5	0.067	0.101

SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Site-specific ground motion parameters for S_{DS} and S_{D1} were estimated using the site-specific design response spectrum presented in Table 4. According to Section 21.4 of ASCE 7-10, the S_{DS} value should be taken as the spectral acceleration value at a period of 0.2 seconds but should not be less than 90 percent of any spectral acceleration after that period. Based on this, the S_{DS} value is governed by the spectral acceleration at a period of 0.2 seconds as shown in Table 4. Additionally, the S_{D1} value should be taken as the greater of the spectral acceleration value at a period of 1.0 second or two times the spectral acceleration value at a period of 2.0 seconds. Based on this, the S_{D1} value is governed by the spectral acceleration at a period of 1.0 second as shown in Table 4. The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. Site-specific S_{DS}, S_{D1}, S_{MS}, and S_{M1} values are presented in Table 5.

Table 5
Site-Specific Design Acceleration Parameters

Parameter	Value (5% Damping)
S_{DS}	0.814g
S_{D1}	0.420g
S_{MS}	1.220g
S_{M1}	0.630g

Site-specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA_M) was estimated using Section 21.5 of ASCE 7-10. According to Section 21.5 of ASCE 7-10, the site-specific PGA_M shall be taken as the lesser of the site-specific probabilistic geometric mean peak ground acceleration of Section 21.5.1 and the site-specific deterministic geometric mean peak ground acceleration of Section 21.5.2. Additionally, the site-specific PGA_M shall not be taken as less than 80% of PGA_M determined from Eq. 11.8-1 (code-based). Based on this procedure, the site-specific PGA_M value is 0.573g and is controlled by the probabilistic results.

CLOSURE

We have completed this technical memorandum for the exclusive use of Scripps Health and their consultants for specific application to the subject project. The findings and conclusions presented in this report were prepared in accordance with generally accepted geotechnical engineering practice and are subject to the limitations of Kleinfelder's June 2018 Draft Geologic and Geotechnical Engineering Evaluation Report.

We appreciate this opportunity to be of service and look forward to continuing to work with you in the future. If you have any questions about this memo, please contact us at 619.831.4600.

Sincerely,

KLEINFELDER

Janna Bonfiglio, PE 89334
Project Engineer

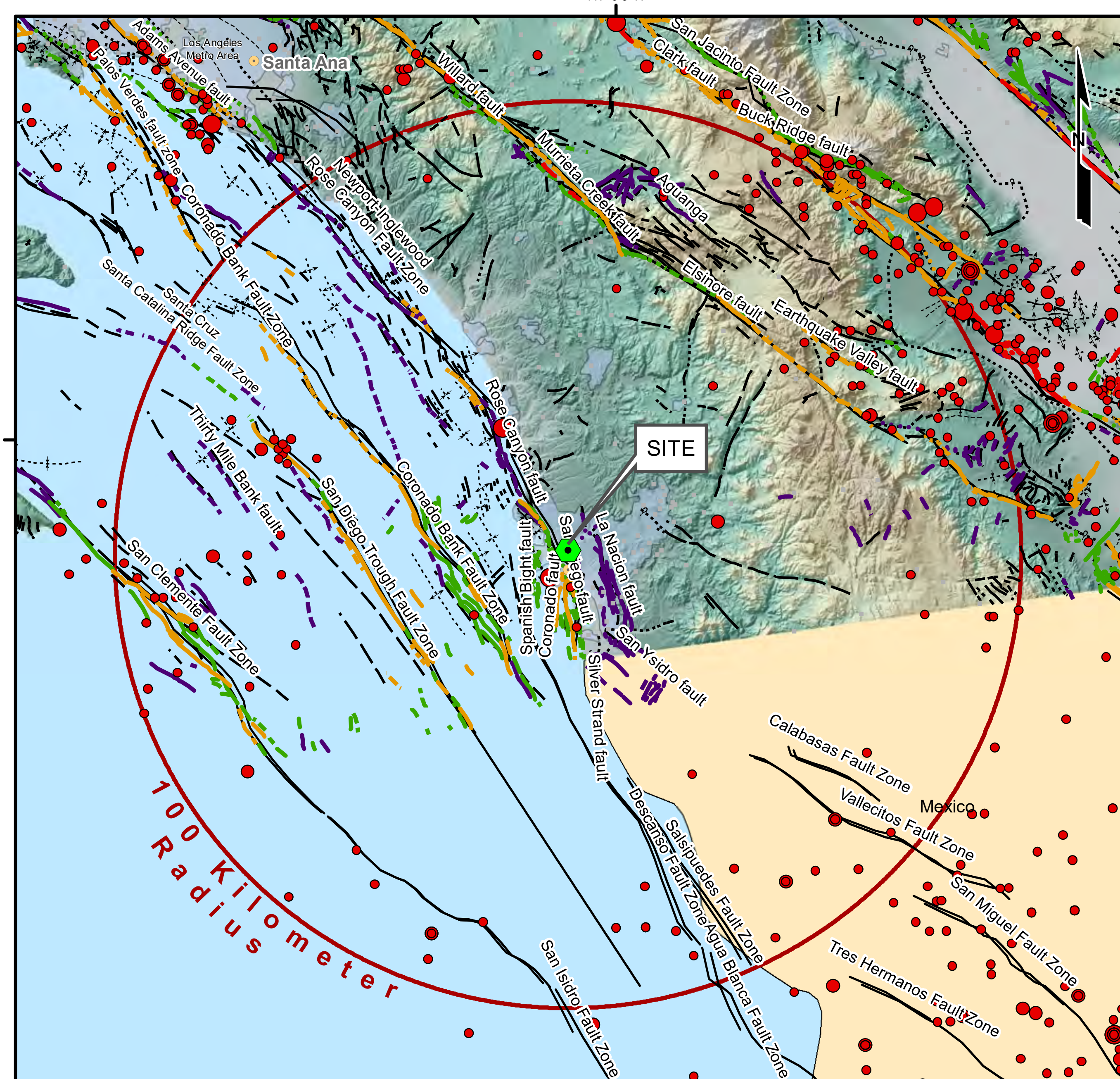
Kevin Crennan, GE 2511
Senior Project Manager

Reviewed By:

Zia Zafir, PhD, PE, GE
Senior Technical Manager

Attachments: Figure 1 – Regional Fault Map and Earthquake Epicenters
Figure 2 – Comparison of Deterministic Spectra
Figure 3 – Comparison of Probabilistic and Deterministic Spectra
Figure 4 – Comparison of DE and 80% of Code Spectra
Figure 5 – Site-Specific Horizontal Spectra for MCE_R and DE
Appendix A - References

FIGURES



Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2009)

Historic displacement (< 200 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Holocene displacement (< 11,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Quaternary displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

ANSS Earthquakes

Magnitude
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7.0 - 7.9
- 8.0 - 8.9

35 17.5 0 35
Kilometers

\\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20183768_001_Mercy\mxd\20183768_EQ_Merc.mxd

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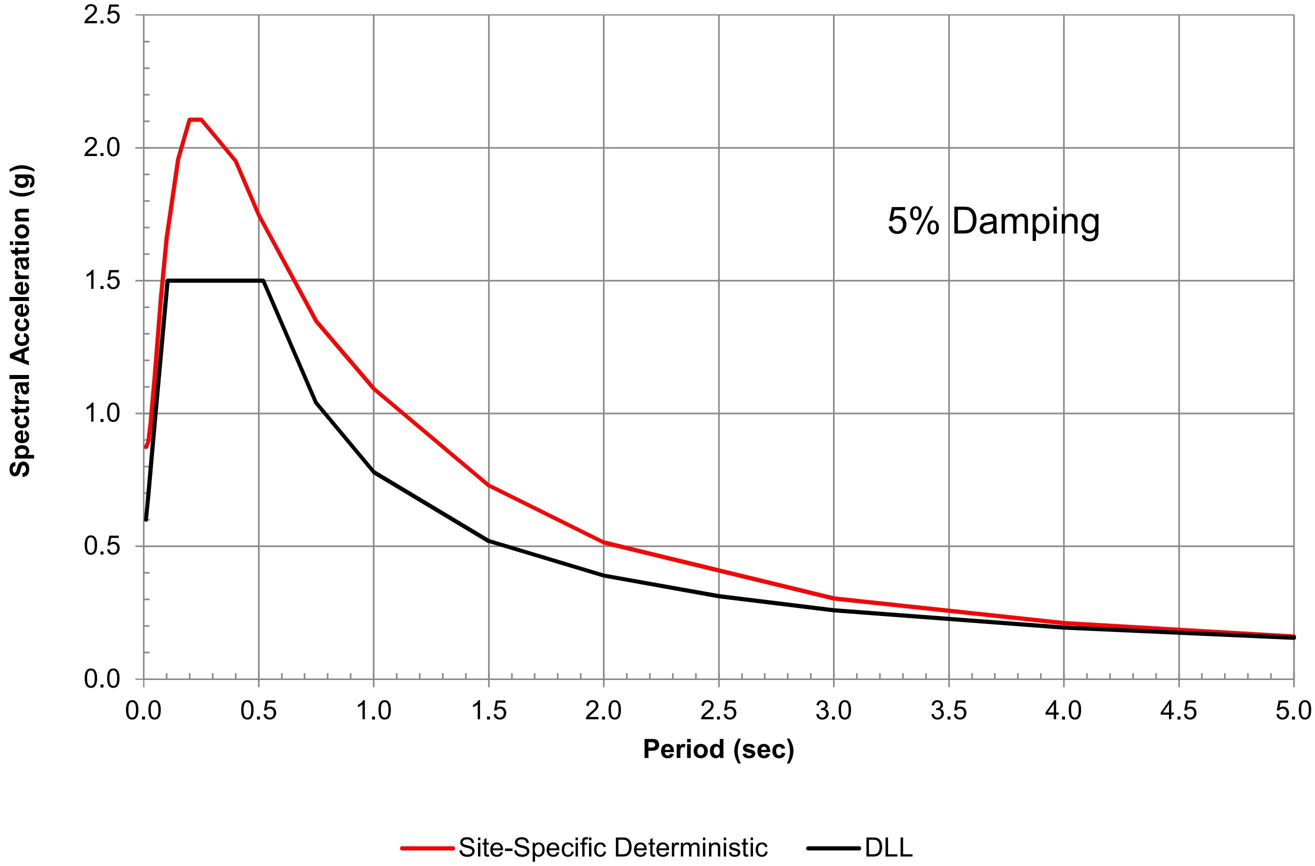
PROJECT NO.	20183768
DRAWN:	4/13/2018
DRAWN BY:	JP
CHECKED BY:	JB
FILE NAME:	20183768_EQ_Merc.MXD

REGIONAL FAULT MAP AND EARTHQUAKE EPICENTERS (1800 - JUNE 2016)

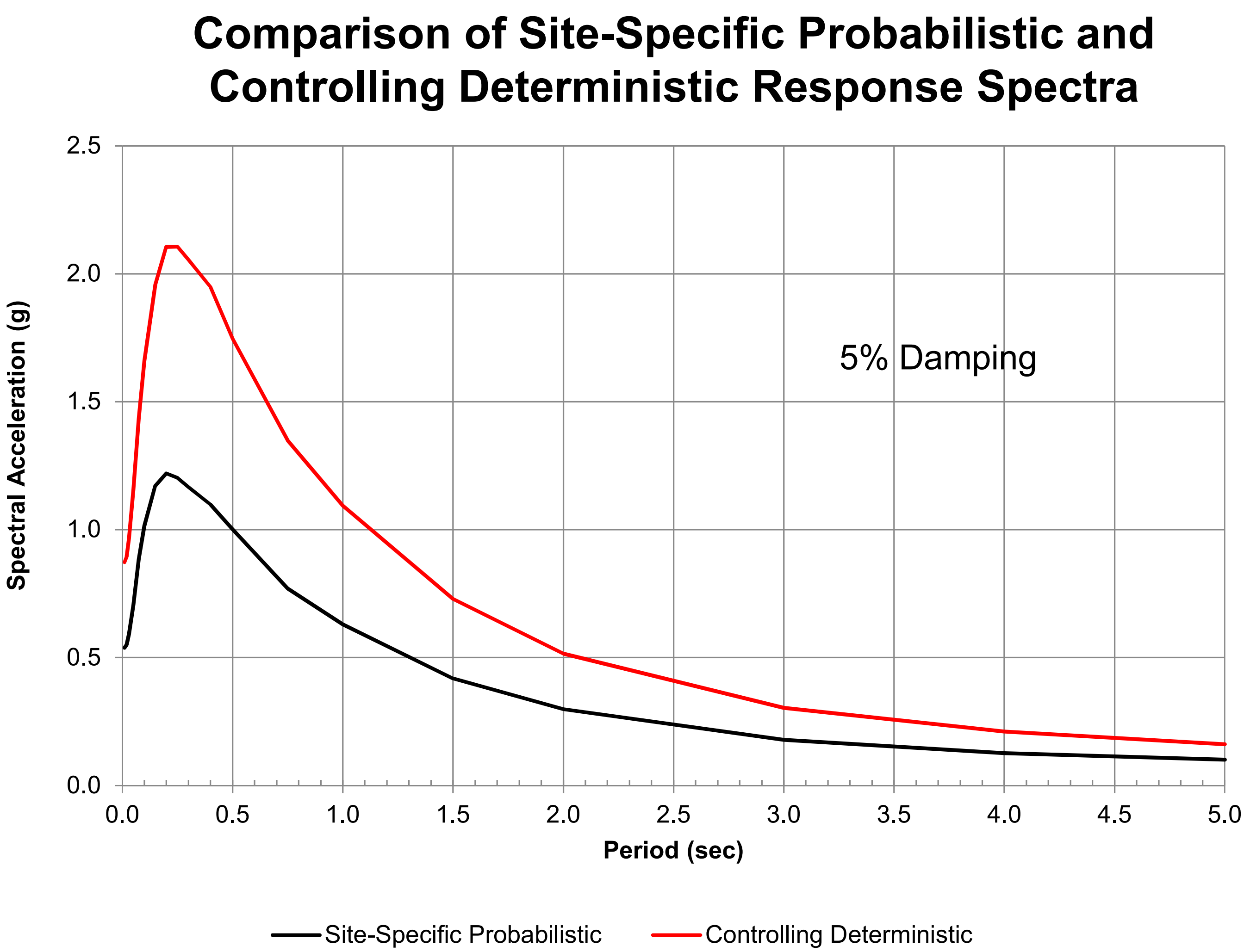
REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
1

Comparison of Site-Specific Deterministic and Deterministic Lower Limit Response Spectra



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20183768.001A DATE: 01/2019 DRAWN: JLB CHECKED BY: ZZ File Name: 2016 CBC Determ & DLL.ppt	**COMPARISON OF DETERMINISTIC SPECTRA 2016 CBC/ASCE 7-10** REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE, SAN DIEGO, CALIFORNIA	FIGURE **2**



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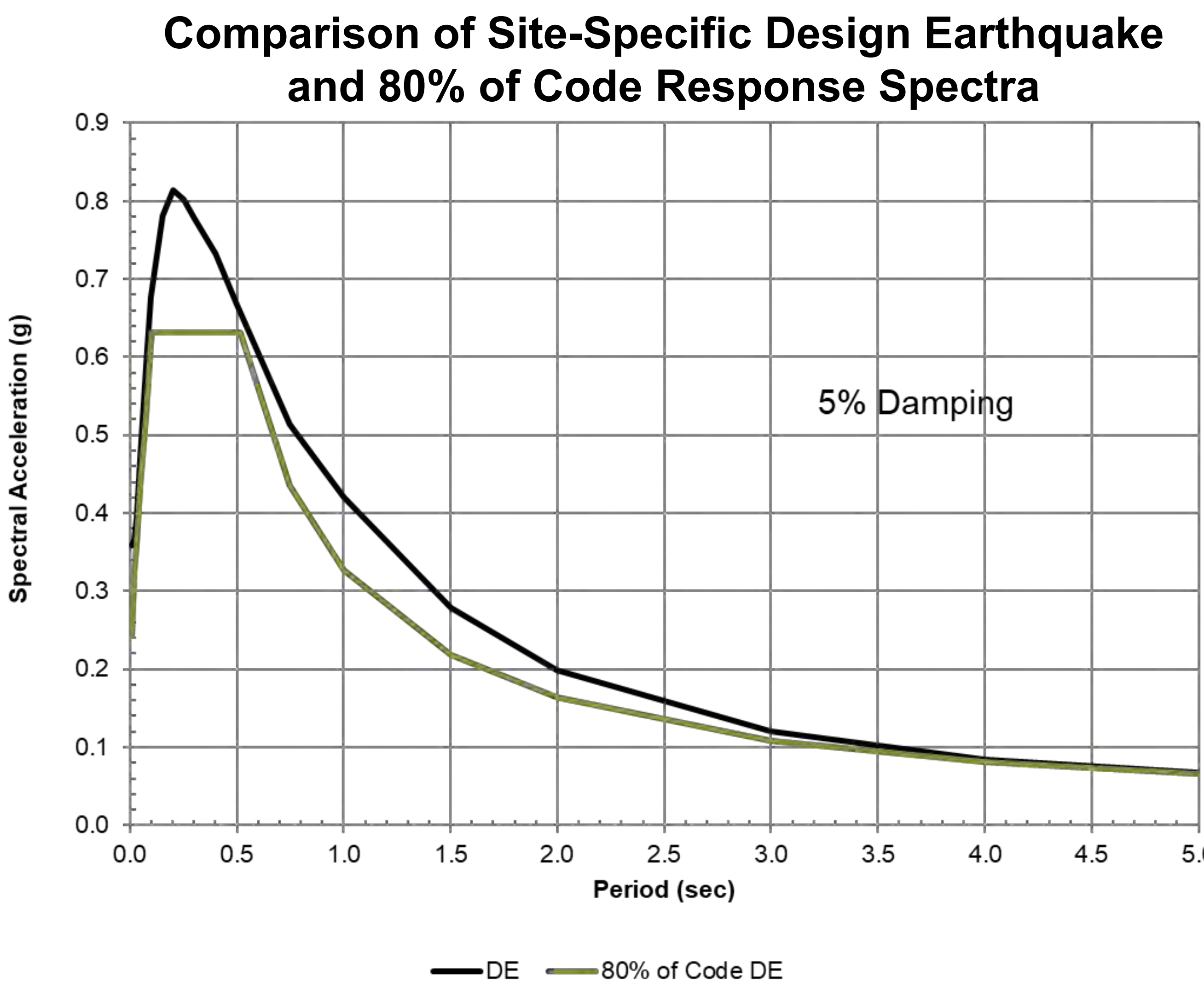
File Name:	2016 CBC Prob & Determ.ppt

COMPARISON OF PROBABILISTIC AND DETERMINISTIC SPECTRA 2016 CBC/ASCE 7-10

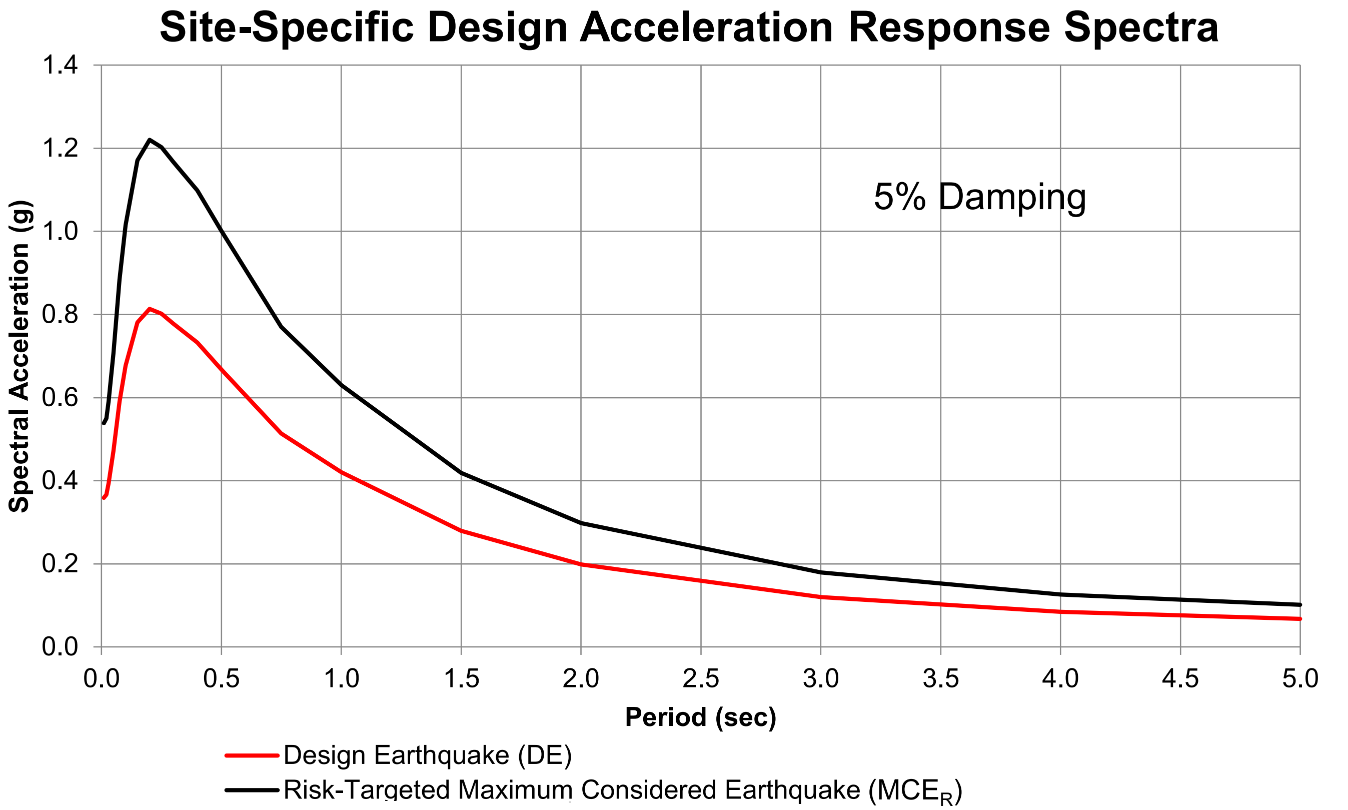
REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE, SAN DIEGO, CALIFORNIA

FIGURE

3



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20183768.001A	**COMPARISON OF DE AND 80% OF CODE SPECTRA 2016 CBC/ASCE 7-10**	FIGURE
	DATE: 01/2019		4
	DRAWN: JLB		
	CHECKED BY: ZZ	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE, SAN DIEGO, CALIFORNIA	
	File Name: 2016 CBC DE & 80%Code.ppt		



Final Site-Specific Design Acceleration Response Spectrum

Period (sec)	Spectral Acceleration (g)	
	DE	MCE$_R$
0.010	0.359	0.538
0.020	0.367	0.550
0.030	0.394	0.592
0.050	0.471	0.707
0.075	0.590	0.885
0.100	0.677	1.015
0.150	0.781	1.171
0.200	0.814	1.220
0.250	0.802	1.202
0.300	0.778	1.167
0.400	0.732	1.098
0.500	0.668	1.002
0.750	0.514	0.771
1.000	0.420	0.630
1.500	0.279	0.419
2.000	0.198	0.298
3.000	0.120	0.179
4.000	0.084	0.126
5.000	0.067	0.101

Final Site-Specific Design Acceleration Parameters

S_{DS} = 0.814 g S_{MS} = 1.220 g

S_{D1} = 0.420 g S_{M1} = 0.630 g

Notes:
1. The site-specific hazard analysis results presented herein are based on the requirements set forth in the 2016 California Building Code (CBC) and the 2010 ASCE/SEI 7-10 Standard.

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DATE:	01/2019
DRAWN:	JLB
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File Name:	2016 CBC SHA Results.ppt

SITE-SPECIFIC HORIZONTAL SPECTRA FOR MCE$_R$ AND DE 2016 CBC/ASCE 7-10

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE, SAN DIEGO, CALIFORNIA

FIGURE

5

APPENDIX A
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Appendix A
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APPENDIX F

Suggested Guidelines for Earthwork Construction

APPENDIX F
SUGGESTED GUIDELINES FOR EARTHWORK CONSTRUCTION

1.0 GENERAL

1.1 **Scope** - The work done under these specifications shall include clearing, stripping, removal of unsuitable material, excavation, preparation of natural soils, placement and compaction of onsite and imported fill material and placement and compaction of pavement materials.

1.2 **Contractor's Responsibility** - The Contractor shall attentively examine the site in such a manner that he can correlate existing surface conditions with those presented in the geotechnical study report. He shall satisfy himself that the quality and quantity of exposed materials and subsurface soil or rock deposits have been satisfactorily represented by the Geotechnical Engineer's report and project drawings. Any discrepancy of prior knowledge to the Contractor to that is revealed through his study shall be made known to the Owner. It is the Contractor's responsibility to review the report prior to construction. The selection of equipment for use on the project and the order of the work shall similarly be the Contractor's responsibility. The Contractor shall be responsible for providing equipment capable of completing the requirements included in the following sections.

1.3 **Geotechnical Engineer** - The work covered by these specifications shall be observed and tested by Kleinfelder, the Geotechnical Engineer, who shall be hired by the Owner. The Geotechnical Engineer will be present during the site preparation and grading to observe the work and to perform the tests necessary to evaluate material quality and compaction. The Geotechnical Engineer shall submit a report to the Owner, including a tabulation of tests performed. The costs of re-testing unsuitable work installed by the Contractors shall be deducted by the Owner from the payments to the Contractor.

1.4 **Standard Specifications** - Where referred to in these specifications, "Standard Specifications" shall mean the State of California Standard Specifications for Public Works Construction, with Regional Supplement Amendments for San Diego County, 2000 Edition.

1.5 **Compaction Test Method** - Where referred to herein, relative compaction shall mean the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D 1557 Compaction Test Procedure. Optimum moisture content shall mean the moisture content at the maximum dry density determined above.

2.0 SITE PREPARATION

2.1 **Clearing** - Areas to be graded shall be cleared and grubbed of all vegetation and debris. The Contractor shall remove these materials from the site.

2.2 **Stripping** - Surface soils containing roots and organic matter shall be stripped from areas to be graded and stockpiled or discarded as directed by the Owner. In general, the depth of stripping of the topsoil will be approximately 3 inches. Deeper stripping, where required to remove weak soils or accumulations of organic matter, shall be performed when determined necessary by the Geotechnical Engineer. Stripped material shall be removed from the site or stockpiled at a location designated by the Owner.

2.3 **Removal of Existing Fill** - Existing fill soils, trash and debris in the areas to be graded shall be removed prior to the placing of any compacted fill. Portions of any existing fills that are suitable for use in new compacted fill may be stockpiled for future use. All organic materials, topsoil, expansive soils, oversized rock or other unsuitable material shall be removed from the site by the Contractor or disposed of at a location onsite, if so designated by the Owner.

2.4 **Ground Surface** - The ground surface exposed by stripping shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction and compacted as required for compacted fill. Ground surface preparation shall be approved by the Geotechnical Engineer prior to placing fill.

3.0 EXCAVATION

3.1 **General** - Excavations shall be made to the lines and grades indicated on the plans. The data presented in the Geotechnical Engineer's report is for information only and the Contractor shall make his own interpretation with regard to the methods and equipment necessary to perform the excavation and to obtain material suitable for fill.

3.2 **Materials** - Soils which are removed and are unsuitable for fill shall be placed in nonstructural areas of the project, or in deeper fills at locations designated by the Geotechnical Engineer.

All oversize rocks and boulders that cannot be incorporated in the work by placing in embankments or used as rip-rap or for other purposes shall be removed from the site by the Contractor.

3.3 **Treatment of Exposed Surface** - The ground surface exposed by excavation shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction and compacted as required for compacted fill. Compaction shall be approved by the Geotechnical Engineer prior to placing fill.

3.4 **Rock Excavation** - Where solid rock is encountered in areas to be excavated, it shall be loosened and broken up so that no solid ribs, projections or large fragments will be within 6 inches of the surface of the final subgrade.

4.0 COMPACTED FILL

4.1 **Materials** - Fill material shall consist of suitable onsite or imported soil. All materials used for structural fill shall be reasonably free of organic material, have an Expansion Index of 30 or less, 100% passing the 3 inch sieve and less than 30 percent passing the #200 sieve.

4.2 **Placement** - All fill materials shall be placed in layers of 8 inches or less in loose thickness and uniformly moisture conditioned. Each lift should then be compacted with a sheepsfoot roller or other approved compaction equipment to at least 90 percent relative compaction in areas under structures, utilities, roadways and parking areas. No fill material shall be placed, spread or rolled while it is frozen or thawing, or during unfavorable weather conditions.

4.3 **Compaction Equipment** - The Contractor shall provide and use sufficient equipment of a type and weight suitable for the conditions encountered in the field. The equipment shall be capable of obtaining the required compaction in all areas.

4.4 **Recompaction** - When, in the judgment of the Geotechnical Engineer, sufficient compactive effort has not been used, or where the field density tests indicate that the required compaction or moisture content has not been obtained, or if pumping or other indications of instability are noted, the fill shall be reworked and recompacted as needed to obtain a stable fill at the required density and moisture content before additional fill is placed.

4.5 **Responsibility** - The Contractor shall be responsible for the maintenance and protection of all embankments and fills made during the contract period and shall bear the expense of replacing any portion which has become displaced due to carelessness, negligent work or failure to take proper precautions.

5.0 UTILITY TRENCH BEDDING AND BACKFILL

5.1 **Material** - Pipe bedding shall be defined as all material within 4 inches of the perimeter and 12 inches over the top of the pipe. Material for use as bedding shall be clean sand, gravel, crushed aggregate or native free-draining material, having a Sand Equivalent of not less than 30.

Backfill should be classified as all material within the remainder of the trench. Backfill shall meet the requirements set forth in Section 4.0 for compacted fill.

5.2 **Placement and Compaction** - Pipe bedding shall be placed in layers not exceeding 8 inches in loose thickness, conditioned to the proper moisture content for compaction and compacted to at least 90 percent relative compaction. All other trench backfill shall be placed and compacted in accordance with Section 306-1.3.2 of the Standard Specifications for Mechanically Compacted Backfill. Backfill shall be compacted as required for adjacent fill. If not specified, backfill shall be compacted to at least 90 percent relative compaction in areas under structures, utilities, roadways, parking areas and concrete flatwork.

6.0 SUBSURFACE DRAINAGE

6.1 **General** - Subsurface drainage shall be constructed as shown on the plans. Drainage pipe shall meet the requirements set forth in the Standard Specifications.

6.2 **Materials** - Permeable drain rock used for subdrainage shall meet the following gradation requirements:

Sieve Size	Percentage Passing
3"	100
1-1/2"	90 - 100
3/4"	50 - 80
No. 4	24 - 40
No. 100	0-4
No. 200	0 - 2

6.3 **Geotextile Fabric** - Filter fabric shall be placed between the permeable drain rock and native soils. Filter cloth shall have an equivalent opening size greater than the No. 100 sieve and a grab strength not less than 100 pounds. Samples of filter fabric shall be submitted to the Geotechnical Engineer for approval before the material is brought to the site.

6.3 **Placement and Compaction** - Drain rock shall be placed in layers not exceeding 8 inches in loose thickness and compacted as required for adjacent fill, but in no case, to be less than 85 percent relative compaction. Placement of geotextile fabric shall be in accordance with the manufacturer's specifications and shall be checked by the Geotechnical Engineer.

7.0 AGGREGATE BASE BENEATH CONCRETE SLABS

7.1 **Materials** - Aggregate base beneath concrete slabs shall consist of clean free-draining sand, gravel or crushed rock conforming to the following gradation requirements:

Sieve Size	Percent Passing
1"	100
3/8"	30 – 100
No. 20	0 – 10

7.2 **Placement** - Aggregate base shall be compacted and kept moist until placement of concrete. Compaction shall be by suitable vibrating compactors. Aggregate base shall be placed in layers not exceeding 8 inches in loose thickness. Each layer shall be compacted by at least four passes of the compaction equipment or until 95 percent relative compaction has been obtained.

APPENDIX G

Geotechnical Business Council Insert

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

June 27, 2019
Kleinfelder Project No. 20194096.001A

Mr. Bruce A. Rainey
Corporate Vice President
Scripps Health
10140 Campus Point Drive, Suite 210
San Diego, California 92121

SUBJECT: **Geologic and Geotechnical Engineering Report**
Hospital Support Building
Scripps Mercy Hospital
4077 5th Avenue
San Diego, California

Dear Mr. Rainey:

This report presents the results and recommendations of our geologic and geotechnical engineering investigation for the design and construction of the proposed hospital support building. The proposed project is to be constructed on the northeastern corner of Washington Street and 5th Avenue at the existing Scripps Mercy Hospital campus located on 4077 5th Avenue in San Diego, California.

Based on the results of our geotechnical investigation and engineering evaluation, it is our opinion that the project is feasible from a geotechnical standpoint. Our geotechnical investigation and evaluation are based on the provided documents, discussions with Mr. Dylan Williams of Scripps Health and our proposed scope of work.

We appreciate this opportunity to be of service and look forward to working with you in the future. If you have any questions about this report, please contact us.

Respectfully submitted,

KLEINFELDER

Kevin M. Crennan GE 2511
Senior Engineer

Scott H. Rugg CEG 1651
Engineering Geologist

GEOLOGIC AND GEOTECHNICAL ENGINEERING REPORT
HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA
KLEINFELDER PROJECT NO. 20194096.001A

JUNE 27, 2019

A Report Prepared for:

Mr. Bruce A. Rainey
Corporate Vice President
Scripps Health
10140 Campus Point Drive, Suite 210
San Diego, California 92121

GEOLOGIC AND GEOTECHNICAL ENGINEERING REPORT
HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

Prepared by:

Kevin M. Crennan GE 2511	Scott H. Rugg CEG 1651
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June 27, 2019
Kleinfelder Project No: 20194096.001A

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EXECUTIVE SUMMARY

The Scripps Mercy Hospital San Diego is located at 4077 5th Avenue in San Diego, California (Figure 1). The proposed hospital support building is located in the southwestern corner of the hospital campus on the northeastern corner of Washington Street and 5th Avenue. The approximate footprint of the proposed building site is shown in Figure 2.

Preliminary plans indicate that the proposed structure will have four subterranean levels with a four story above-grade building in the eastern portion of the site. Temporary shoring is anticipated along most of the building perimeter to protect existing structures, streets and improvements. However, the northern side of the eastern half of the building will daylight to a depressed area which is approximately 50 feet lower in elevation.

Kleinfelder's field exploration for the project consisted of advancing six hollow stem auger borings, infiltration testing and geologic reconnaissance of the site. Boring data from previous nearby studies on the Scripps Mercy campus were also evaluated in this study. The Logs of Borings for this study are included in Appendix A, pertinent borings from nearby studies are in Appendix A.1 and the locations are shown in Figure 2.

The results of our field exploration and review indicate that fill up to approximately 8 feet in depth is present within the building area. The very old paralic deposits underly the fill and overlay the San Diego Formation. These formational units are typically very dense and cemented. The very old paralic deposits also contain gravel and cobble and were very difficult to penetrate with 8-inch diameter hollow stem augers. Based on previous boring logs in the area, the Pomerado Conglomerate is present approximately 60 to 70 feet below the surface, well below the depths of excavation. No static groundwater was observed in the borings. A more detailed description of each unit is provided in the attached report.

Based on our review of the data collected during the study, it is our opinion that the proposed development is feasible from a geotechnical perspective if the recommendations contained herein are followed. The following key items are conclusions developed from our study:

- Undocumented fill to depths of up to approximately 8 feet were encountered within our borings in the proposed building area. Deeper fill may be associated with subsurface utilities or buried structures.
- Based on the results of our geotechnical evaluation, it is our opinion that the undocumented fill is not suitable for support of structural loads in their current condition. However, it is anticipated that all of this material will be removed for the building excavation which will be on the order of 15 feet in depth.
- The lower level of the building can be supported on spread footings or a mat foundation system bearing entirely on the San Diego Formation.
- Temporary shoring will be required around most of the building perimeter to protect existing improvements during construction of the subterranean levels, with the exception of the northeastern side which will daylight to area below.
- The replacement hospital building site is located in a seismically active area and could be subject to relatively strong ground shaking due to earthquakes on active faults in the region. The structures should be designed to tolerate seismic shaking in accordance with the 2019 California Building Code (CBC).
- The site is not located within a State of California Alquist-Priolo fault zone.

This summary of findings should not be relied upon without consulting the attached report for a more detailed description of the geotechnical study performed by Kleinfelder for Scripps Mercy Hospital San Diego. This report is subject to the limitations included in Section 7.0 and the Geotechnical Business Council insert in Appendix E.

1 INTRODUCTION

1.1 PURPOSE

In accordance with your authorization, we have completed a geologic and geotechnical investigation for the proposed hospital support building at Scripps Mercy Hospital located at 4077 5th Avenue, San Diego, California (see Vicinity Map, Figure 1). The purpose of our study was to evaluate the surface and subsurface conditions at the site, evaluate potential geologic hazards, and provide geotechnical recommendations for design and construction. This report presents the results of our background review, subsurface exploration, laboratory testing, geotechnical analyses, conclusions regarding the geotechnical conditions at the project site, and our recommendations.

1.2 PROJECT DESCRIPTION

We understand the project is in the early stages of design and preliminary building plans provided by CO Architects were provided for review. The proposed project will have four subterranean levels over the entire site with the overlying four story hospital support building in the eastern portion of the site. Vehicle access will be from Washington Street and the western at-grade portion of the building will have a vehicular dropoff for patients and the entrance to the lower level parking. Finish floor elevations are 291 feet for the surface elevation and 243 for the lowest level, with the parking level ramping down to about 237 feet MSL. A 4-level corridor bridge will overly the lower levels and connect the support building to the future replacement hospital to the north.

Additional improvements will likely include hardscape, landscaping, lighting, underground utilities and possibly stormwater BMPs. Pavement is not anticipated due to the structure occupying the majority of the site. Temporary shoring is anticipated along most of the building perimeter to protect existing structures, streets and improvements. However, the northern side of the eastern half of the building will daylight to a depressed area which is approximately 50 feet lower in elevation. Current plans indicate that this area will connect to the future replacement hospital to the north.

The address and latitude/longitude coordinates for the hospital support building are listed below, and the vicinity map is shown on Figure 1. The site and exploration plan is shown on Figure 2.

Address:	Scripps Mercy Hospital San Diego 4077 5th Avenue San Diego, California 92103
Latitude:	32.75047^{O} N
Longitude:	117.16052^{O} W

1.3 GENERAL SITE DESCRIPTION

The Mercy Hospital campus is generally located north of Washington Street, south of Montecito Way, west of 6th Avenue and east of 4th Avenue, as shown on the Vicinity Map, Figure 1. The site is bordered by 5th Avenue on the west, Washington Street on the south, the 5-story residential Warwick building on the east, the emergency building on the northwest and a parking lot at a lower elevation on the northeast. The proposed structure will be located within an approximate 68,000 square foot area that currently contains an existing at-grade parking lot in the northwest corner, the Scripps O'Toole Breast Care Center and Surgery Pavilion on the south, and a 4-level subterranean parking structure on the northeast. An approximate 5-foot high concrete retaining wall is present above the lower parking lot and below the exposed portion of the parking structure. A stairway along the northern side descends about 50 feet to the lower parking lot. The site also includes small landscape areas adjacent to the sidewalk along Washington Street. Surface elevations vary from about 288 feet on the southeast to 291 feet MSL on the northwest. The lower parking lot on the northeast is approximately 50 feet lower at an elevation of about 240 feet MSL.

1.4 SCOPE OF SERVICES

The scope of services for this study included the following;

- Review of readily available pertinent reports (including previous borings performed near the site), geologic maps and aerial photographs;
- Drilling and sampling six soil borings;

- Performing geotechnical laboratory testing of the soil samples;
- Providing a site plan with boring locations;
- Discussion of site surface and subsurface conditions encountered, groundwater levels and anticipated excavation characteristics of the materials;
- Discussion of regional geologic setting, geologic features, and geologic hazard potential including the potential for ground rupture due to surface faulting, liquefaction and seismically induced settlement;
- Providing 2019 California Building Code seismic parameters;
- Recommendations for temporary shoring;
- Lateral earth pressures for permanent retaining walls;
- Recommendations for foundation design, including foundation type, soil bearing pressures, and anticipated total and differential settlements;
- Performing a preliminary screening of soil corrosivity characteristics;
- Guidelines for earthwork construction including recommendations for site preparation, shoring, fill placement and compaction.
- Recommendations for slab-on-grade floor design and construction;
- Preliminary stormwater infiltration study; and
- Preparation of this report.

2 METHODS OF STUDY

2.1 FIELD EXPLORATION

Our current field exploration consisted of a geologic reconnaissance of the site, drilling four borings to evaluate subsurface conditions, drilling two borings for infiltration testing, and collection of soil samples for laboratory testing. The exploration borings were drilled on May 11 and 18, 2019 to depths up to approximately 71 ½ feet and the two borings for infiltration testing were drilled to a depth of 5 feet. Due to access constraints at the site, the drilling contractor, Pacific Drilling of San Diego, used a limited access Fraste PL-G drill rig with the exception of a truck-mounted Marl-10 which was used in the parking lot at boring B-1. The rig was equipped with 6-inch diameter hollow stem augers, and a 140-pound hammer dropped from a height of 30 inches to drive the sampler into the soil.

A Kleinfelder engineer from our office logged the subsurface soil conditions at each boring location and collected soil samples for laboratory testing. Driven samples were obtained from the borings using Standard Penetration Test and California split spoon samplers lined with three 6-inch brass sleeves. Soil samples obtained from the borings were packaged, sealed in the field to reduce moisture loss and disturbance, and returned to our laboratory for testing. Upon completion, all borings were backfilled with soil cuttings or bentonite chips. A more detailed description of the Kleinfelder boring exploration program, the logs of the exploratory borings are presented in Appendix A.

2.2 GEOLOGIC RECONNAISSANCE AND MAPPING

The site was geologically evaluated by our engineering geologist. Part of this work included reviewing historic maps and aerial photography to evaluate the extent of previous earthwork at the site for infilled drainages. Data from our geologic review is incorporated into this report.

2.3 LABORATORY TESTING

Geotechnical laboratory testing was performed on representative bulk and driven samples to substantiate field classifications and to provide engineering parameters for geotechnical design. Testing performed consisted of moisture/density measurement, sieve analyses, direct shear, Atterberg limits, pH, resistivity, soluble sulfates, and chlorides. Laboratory testing procedures and test results are provided in Appendix B.

2.4 BACKGROUND REVIEW

We have reviewed readily available unpublished geotechnical reports relevant to the subject site. The most pertinent reports to the current project include the following reports for the adjacent emergency department to the north and the proposed hospital replacement building below the site to the northeast:

- "Geologic and Geotechnical Engineering Evaluation, Scripps Mercy Hospital Expansion, New Emergency Department, 4077 5th Avenue, San Diego California," prepared by Kleinfelder, dated January 20, 2006.
- "Supplemental Geotechnical Investigation, Proposed Emergency Department Expansion, Scripps Mercy Hospital, 4077 5th Avenue, San Diego California," prepared by Kleinfelder, dated March 6, 2007.
- "Geologic and Geotechnical Engineering Evaluation, Replacement Hospital, Scripps Mercy Hospital San Diego, 4077 5th Avenue, San Diego California," prepared by Kleinfelder, dated April 26, 2019.

Pertinent borings logs from these reports are included with Appendix A.1. We have also reviewed historic aerial photography and maps of the site as part of our work.

3 GEOLOGY AND SOILS

3.1 REGIONAL GEOLOGIC AND GEOTECTONIC SETTING

The site is located on a coastal terrace within the Peninsular Range geomorphic province (Norris and Webb, 1990). This province stretches for several hundreds of miles south from the Los Angeles area to the tip of Baja California. It is dominated by basement material of Cretaceous-age igneous rocks of the Southern California Batholith and various Jurassic-age metamorphic rocks that are often situated as isolated blocks within the igneous rocks. This igneous/metamorphic complex occupies the regions of central and eastern San Diego County.

The western coastal zone of San Diego County is underlain by a westward thickening wedge of sedimentary units that were deposited on the basement rocks described above. These sedimentary units can be divided into three series of deposits based on their sequence and age of deposition. The oldest sequence consists of claystone, siltstone, sandstone, and conglomerate deposited during late Cretaceous time as an apparent submarine fan (Abbott, 1999). These units crop out on Mt. Soledad in La Jolla, Point Loma, and Carlsbad. The second sequence of sediments was deposited during the Tertiary (Eocene and Pliocene) period within an embayment that stretched from northern San Diego County into Mexico (Kennedy, 1975). The sediments consist of a variety of claystone, siltstone, sandstone, and conglomerate.

The most recent sedimentary deposits consist of early to late Pleistocene near shore marine, estuarine, and delta deposits, also typically identified as terrace deposits. Most of these sediments were deposited on wave cut surfaces (terraces) developed in response to sea level fluctuations during the Pleistocene. The oldest terrace deposits are typically identified as the Lindavista Formation and consist of conglomerate and sandstone with minor clay and silt strata. The youngest terrace deposits (late Pleistocene) are known as the Bay Point Formation. More recent geologic maps (Kennedy and Tan, 2007) have subdivided both the Lindavista and Bay Point Formations into numerous very old paralic deposits (Qvop1 – Qvop13) and old paralic deposits (Qop1 – Qop8) and dropped the previous formal names. The Regional Geologic Map is presented as Figure 3 and shows the local extent of the geologic units described herein.

3.2 REGIONAL FAULTING AND SEISMICITY

The San Andreas fault delineates the boundary between two global tectonic plates consisting of the North American Plate on the east and the Pacific Plate on the west and dominates the seismicity of the Southern California region (Wallace, 1990; Weldon and Sieh, 1985). It stretches from the Gulf of California in Mexico along a northwest alignment through the desert region of Southern California up to Northern California, where it eventually trends offshore north of San Francisco. Within Southern California, the mostly right-lateral strain associated with the plate boundary movement extends well westward for up to 150 miles (241 kilometers) from the main San Andreas fault in the Imperial Valley to well offshore of San Diego.

The major faults east of San Diego (from east to west) include the San Andreas fault, the San Jacinto fault, and the Elsinore fault (see Regional Fault Map, Figure 4). Major faults west of San Diego include the Palos Verdes-Coronado Bank fault, the San Diego Trough fault, and the San Clemente fault. The most dominant zone of faulting within the San Diego region are several faults associated with the Rose Canyon Fault Zone (RCFZ).

Most of the seismic energy and associated fault displacement occurs along the fault structures closest to the plate boundary on the Elsinore, San Jacinto, and San Andreas Faults. Approximately 49 mm/yr (1.9 inches/yr) of overall lateral displacement has been measured geodetically as fault slip across the plate boundary. The Elsinore, San Jacinto, and San Andreas Faults combined account for up to 41 mm/yr (1.6 inches/yr) of the total plate displacement (84%), meaning that the remaining 8 mm/yr (0.3 inches/yr) is accommodated across the faults to the west and east (Bennett, et al, 1996). Recent GPS measurements from the offshore islands to the peninsular ranges indicate about 5 to 7 mm/yr of plate movement is accommodated by the coastal and offshore system of faults, including the Rose Canyon.

Historically, San Diego County has long been considered as a region of negligible seismic hazard. Except for a probable local event in 1862 (Legg and Agnew, 1979), there has been a lack of significant seismic activity within the recorded human history of San Diego County. More recent studies have recognized that the potential for significant seismic events is much greater than earlier believed. This potential has been recognized by the discovery of many active fault traces associated with structures within the RCFZ. Studies within Rose Canyon (east of Mt.

Soledad) have revealed fault strands that have clearly displaced Holocene soil horizons with slip rates from 1 to 2 mm/yr (Lindvall and Rockwell, 1995).

These results indicated that at least the northern onshore portion of the RCFZ is active. Additional studies (Testing Engineers and other, 1985; Patterson and others, 1986; and Kleinfelder, 1998) within downtown San Diego revealed additional fault structures offsetting Holocene soil horizons, suggesting the possibility that the entire mapped onshore alignment of the RCFZ may be active.

More regionally, data has been presented that indicates that the RCFZ may be structurally connected to the Newport-Inglewood Fault Zone (Grant and Rockwell, 2002; Grant and Shearer, 2004) on the north and the San Miguel-Vallecitos fault or the offshore Descanso fault on the south, all of which are active faults. Sahakein, et. al. (2017) processed previously collected seismic reflection and bathymetric data, which indicated relatively narrow (2 kilometer) step-overs fault segments in offshore strands between the two major fault systems. This not only provides additional support of the structural connectively between the two fault systems but also indicates the possibility that they could erupt together with greater magnitude events of up to 7.5M. This larger fault system is thus over 150 miles in length.

3.3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

The results of our investigation indicate that the site is underlain by artificial fill over very old paralic deposits and the San Diego Formation. As part of our engineering analysis we reviewed historic aerial photography and topographic maps and compared these to the present conditions at the site. Information from this work in conjunction with our current and previous nearby boring data was used to estimate the depth and extent of geologic units below the site. Detailed descriptions of these materials are provided in the boring logs (Appendix A). Generalized descriptions are provided in the subsequent sections.

3.3.1 Artificial Fill (af)

Artificial fill material was encountered in all of the current boring locations and extended to depths of about 3 to 8 feet below the ground surface within the borings. It is possible that deeper fill is present at the site and associated with utility trenches, wall backfill or buried structures. We were not provided with information regarding whether full height temporary shoring or a combination of shoring and temporary slopes were used for construction of the existing 4-level subterranean parking structure. However, the fill depth of 7 feet in boring B-1 suggests that shoring may have extended most if not all of the full height. Review of our previous boring B-4 for the northern replacement hospital (Kleinfelder, 2019) indicates that about 4 ½ feet of fill was encountered down to approximate elevation 235.5 feet.

The fill material consisted of clayey sand and silty sand. Due to the lack of compaction records from previous site grading operations and location within landscaped or paved areas, the fill material is considered undocumented and therefore unsuitable for supporting structural loads. It is likely that the fill was derived from nearby materials during site grading and construction for previous site development.

3.3.2 Very Old Paralic Deposits (Qvop9)

Very old paralic deposits underlie the fill at a depth of 3 to 8 feet or at upper contact approximate elevations of 283 to 288 feet MSL in five of the six borings. This unit ranged from 3 to 8 feet thick in our borings and extended to lower contact elevations of 277 to 281 feet MSL. This unit is comprised of a very dense, light reddish yellow and pale brown, silty sand and clayey sand with gravel and cobble. This material is typically moderately cemented and was very difficult to drill, with near refusal of the boring auger at all locations. Refusal was encountered in multiple attempts at boring B-3. Excavations within this unit will be difficult, particularly for trenching operations of utilities and other shallow excavations.

3.3.3 San Diego Formation (T_{sd})

The San Diego Formation was encountered below the very old paralic deposits at depths of 10 to 13 feet or approximate elevations of 277 to 281 feet MSL in three of our deeper borings. This unit was also encountered below fill in our previous boring B-4 for the northern replacement hospital (Kleinfelder, 2019) at an approximate elevation 235.5 feet. This formation consists primarily of yellow to olive brown, silty fine sand which transitions to a fine sandy silt at a depth of about 20 feet. It also contains some scattered gravel and cobble. The unit is in primarily in a very dense and weakly to moderately cemented condition. The sampler blow counts in this material obtained during drilling was typically in excess of 50 blows per foot, with a few isolated blow counts of 23 and 32 blows per foot near the top of the unit. Based on our review of nearby borings for the replacement hospital to the northeast (Kleinfelder, 2019), this unit is anticipated to have a thickness up to about 50 to 60 feet.

3.3.4 Pomerado Conglomerate (T_p)

The Pomerado Conglomerate was not encountered within our current borings and is present well below the depths of construction. However, this unit was encountered directly below the San Diego Formation at approximate elevation +220 to +227 feet MSL in our previous borings northeast of the site for the replacement hospital. Observations of the slope exposures along 6th Avenue east of the site indicate the unit consists of a brown to yellowish brown, cemented cobble conglomerate. The cobbles are typically 3 to 6 inches in size, but larger cobbles and boulders greater than 12 inches in size are occasionally present on the slope outcrops to the east of site. The formation exposed on the slope outcrops typically contain between 20 to 50 percent cobbles.

3.3.5 Groundwater

Groundwater was not encountered in any of our borings. In general, most of the soils encountered are in a moist condition below saturation levels necessary for free groundwater conditions. It is possible that perched groundwater or seepage zones may be present at isolated locations. In particular, perched groundwater typically develops at the interface between more permeable fill and less permeable formational materials or between layers of variable permeability. It should be noted that groundwater levels at the site can fluctuate with time due to changes in weather, irrigation, construction, or other influences that were not present at the time the observations were made.

4 SEISMIC AND GEOLOGIC HAZARDS

We have reviewed the site with respect to the potential presence of geologic and/or seismic hazards. These hazards include landslides, expansive soils, liquefaction, seismic compression, fault surface rupture, and flooding. The following sections discuss these hazards and their potential at this site in more detail.

4.1 EXPANSIVE SOILS

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade.

Visual classification of the soils near anticipated subgrade elevations indicates that these soils primarily consist of non-plastic sandy silt and silty sand, and non- to low-plastic clayey sand and silty sand near the ground surface. Based on the results of our field investigation and previous experience in the site area, it is our opinion that the site soils generally have a very low to low expansion potential. Isolated zones of more expansive soils may also be encountered but are not anticipated. No special requirements for footing and floor slab reinforcement are recommended from a geotechnical perspective.

4.2 SEISMIC GROUND SHAKING

The project site, like all Southern California, is a seismically active area and is likely to experience ground shaking as a result of earthquakes on nearby or more distant faults. The Rose Canyon fault zone and Elsinore fault zones dominate the seismicity of the area. The Rose Canyon fault zone (CDMG, 1998) is located approximately 1.5 miles southwest of the project site.

We understand that the proposed structure will be designed in accordance with the requirements of the latest 2019 edition of the California Building Code (CBC). It should be noted that the seismic provision of the 2019 CBC are based on and refer to (for more requirements) "Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7" (referred to herein as "ASCE 7").

Based on the results of our borings, we classify the site as Site Class C. Site Class C is defined as a very dense soil and soft rock with average shear wave velocities within the upper 30 meters between 1,200 ft/sec. and 2,500 ft/sec., average SPT N>50, or average undrained shear strength Su > 2,000psf. The recommended seismic design parameters are presented in Table 1.

Table 1
Recommended 2019 CBC Seismic Design Parameters

Design Parameter	Symbol	Recommended Value	2016 CBC (ASCE 7) Reference(s)
Site Class	--	C	Section 1613A.5.2
Mapped spectral acceleration for short periods	S_s	1.38g	Section 1613A.5.1
Mapped spectral acceleration for a 1-second period	S_1	0.47g	Section 1613A.5.1
Site Coefficient	F_a	1.2	Table 1613A.5.3(1)
Site Coefficient	F_v	1.5	Table 1613A.5.3(2)
MCE* Peak Ground Acceleration (S_M at T=0)	PGA_M	0.75 g	n/a
MCE* spectral response acceleration for short periods	S_{MS}	1.66 g	Section 1614A.1.1 (Section 21.4)
MCE* spectral response acceleration at 1-second period	S_{M1}	0.71 g	Section 1613A.5.3 (Section 21.4)
Design Peak Ground Acceleration (S_D at T=0)	PGA_D	0.86 g	Section 1802A.2.7
Design spectral response acceleration (5% damped) at short periods	S_{DS}	1.11 g	Section 1613A.5.4 (Section 21.4)
Design spectral response acceleration (5% damped) at 1-second period	S_{D1}	0.47 g	Section 1613A.5.4 (Section 21.4)

*MCE : Maximum Considered Earthquake

4.3 LIQUEFACTION

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or "cyclic mobility", increased lateral earth pressures on retaining walls, post liquefaction settlement, lateral spreading and "flow failures" in slopes.

Based on the absence of near-surface groundwater, as well as the presence of dense to very dense formational soils, the potential for liquefaction of the subsurface soils at the site is considered low.

4.4 SEISMIC COMPRESSION

Seismic compression results from the accumulation of contractive volumetric strains in unsaturated soil during earthquake shaking. Loose to medium dense granular material with no fines or with low plasticity fines are most susceptible to seismic compression. The site will require excavation to proposed finish grade which we anticipate will be on the very dense San Diego Formation, therefore the hazard posed to the site by seismically induced settlement is considered low.

4.5 FAULT SURFACE RUPTURE

Review of readily available geologic and fault maps does not show any active or potentially active fault features passing through or nearby the site. The closest active fault to the site is the Rose Canyon fault, which is located approximately 1.5 miles to the southwest. The site is not within a State of California Alquist-Priolo Earthquake Fault Zone. In addition, published geologic maps do not show any faults crossing through or nearby the site. Finally, review of predevelopment aerial photographs do not show geomorphic features or lineaments indicative of faulting across the site. Based on this information, the geologic hazard with respect to fault rupture is considered low.

4.6 LANDSLIDES

Landslides are deep-seated ground failures (tens to hundreds of feet deep) in which a large section of a slope slides downhill. Landslides are not to be confused with minor surficial slope failures (slumps), which are usually limited to the topsoil zone and can occur on slopes composed of almost any geologic material. Landslides can cause damage to structures both above and below the slide mass. Undermining of foundations can damage structures above the slide area. Areas below a slide can be damaged by being overridden and crushed by the failed slope material.

The site is located in a relatively level with the exception of the historic drainage to the northeast. The proposed building is constructed about 50 feet below grade and well into cemented formational materials. Therefore, the potential for landsliding is considered slow.

4.7 TSUNAMIS AND SEICHES

A tsunami is a giant sea wave (which can reach over 50 feet in height) usually generated by catastrophic displacement on a submarine fault. Tsunamis can travel at speeds of hundreds of miles per hour over distances of thousands of miles. In the open ocean, tsunamis have large wavelengths and are difficult to detect. As the sea wave approaches shore, the wave decreases in wavelength and increases in amplitude (height). Large tsunamis can travel well beyond the normal wave break of the shoreline and cause damage to near shore structures.

A seiche is an oscillation (wave) of a body of water in an enclosed or semi-enclosed basin that varies in period, depending on the physical dimensions of the basin, from a few minutes to several hours, and in height from several inches to several feet. A seiche is caused chiefly by local changes in atmospheric pressure, aided by winds, tidal currents, and occasionally earthquakes.

The project site is located about 5 ½ miles from the Pacific Ocean and is located at an elevation of approximately 290 feet or more above mean sea level. Additionally, the site is not located adjacent to any large bodies of water that could adversely affect the site in the event of earthquake-induced failures or seiches. Therefore, the hazard with respect to a tsunami or seiche is considered low.

4.8 FLOODING

Flooding occurs as a result of several factors in developed areas. These factors include rainfall rates that exceed an area's ability to absorb or control the runoff; impounded water retained behind a flood control structure (upstream-inundation), failure of a flood control structure (downstream-indundation), Seiches, or tsunami.

The Federal Emergency and Management Administration (FEMA) maintains a collection of Flood Insurance Rate Maps (FIRM), which cover the entire United States. These maps identify those areas which may be subjected to 100-year and 500-year cycle floods. A set of these maps for the County of San Diego are available for viewing on the FEMA website (https://msc.fema.gov/portal). Based on our review of FEMA map panel 06073C1618G, the site is not within any designated flood zones and therefore the potential for flooding of the proposed development is considered low.

5 DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

Based on the results of our background review, subsurface exploration, laboratory test results, and engineering analyses, it is our opinion that the construction of the proposed MOB building is feasible from a geotechnical standpoint provided the recommendations of this report are incorporated into the design and construction of the project. The following recommendations were prepared based on our understanding of the project as depicted on the progress plans titled "Scripps Mercy Hospital Support Building," prepared by CO Architecture and dated May 17, 2019.

5.1 SITE GRADING

The following recommendations were prepared based on our understanding of the project as previously described in this report. Kleinfelder should be provided with updated plans by the design team if design is modified.

5.1.1 General

All site preparation and earthwork operations should be performed in accordance with applicable codes including the current edition of the California Building Code, Section 1803A. All reference in this report to maximum dry density is established in accordance with American Society for Testing and Materials (ASTM) ASTM D 1557. We recommend that site earthwork and construction be performed in accordance with the following recommendations and guidelines presented in the Suggested Guidelines for Earthwork Construction included in Appendix D. In case of conflict, the following recommendations supersede those outlined in Appendix D.

5.1.2 Site Preparation

Since the excavation for the proposed structure will be on the order of 55 feet in depth, all of the existing pavement, hardscape, site improvements and landscaping will be demolished and removed prior to construction of the proposed building. Man-made structures, including buried pipes, utilities, etc., should be completely removed within the building pad. Temporary shoring which was likely used for construction of the existing parking structure should be removed and the potential soldier beams should be removed or cut off at least 4 feet below new foundations. Subsurface utilities should be rerouted or plugged and capped at the building perimeter. Excavations for removal of any man-made items should be dish-shaped and backfilled with properly compacted engineered fill per Section 5.1.4. All surficial vegetation and deleterious material should be stripped and completely removed from the proposed site area.

5.1.3 Subgrade Preparation

Excavations for the building foundations are anticipated to be on the order of 55 feet and into native soils of the San Diego Formation. Depending on final foundation elevations, it is possible that shallow fill may be present at the extreme northeast corner of the structure. If encountered, fill should be completely removed and recompacted. The exposed surface in formational soils should may be left above finish subgrade elevation during foundation construction and disturbed soils excavated to expose undisturbed formational soils. Remaining disturbed soils may be moisture conditioned and compacted. The need for scarification to provide adequate moisture conditioning may be evaluated by the geotechnical engineer at the time of grading.

5.1.4 Engineered Fill

The majority of existing undocumented fill material is considered suitable for re-use as engineered fill outside of the building footprint, however the clay soils may be unsuitable considering the volume of granular soils which will be generated from the building excavation. The onsite San Diego Formation materials may be used as engineered fill, as they are not anticipated to contain oversized rock, organic materials, expansive clay, and deleterious debris. If encountered, oversize material in excess of 6 inches in diameter should not be used in engineered fill and material larger than 3 inches should not be used within the upper 6 inches for foundation subgrade. The onsite soil placed as engineered fill should be moisture conditioned between 0 and 3 percent above optimum moisture content and compacted to a minimum of 95 percent relative compaction in structural areas, and 90 percent in non-structural areas, based on ASTM D 1557.

Import materials used as engineered fill (if any) should consist of clean, granular material that has less than 30 percent passing the #200 sieve, a minimum R-value of 20, and expansion index of 20 or less as evaluated by ASTM 4829. Imported engineered fill should be moisture conditioned between 0 to 3 percent above optimum moisture content and compacted to a minimum of 95 percent relative compaction in structural areas and 90 percent in non-structural areas, based on ASTM D 1557.

Although the optimum lift thickness for fill soils will be dependent on the size and type of compaction equipment utilized, fill should generally be placed in uniform lifts not exceeding approximately 8 inches in loose thickness. Oversized material, rocks, or hard clay lumps greater than 6 inches in dimension should not be used in compacted fills and greater than 3 inches in dimension should not be used in the upper 3 feet in structural areas. In pavement and exterior flatwork areas the upper 12 inches of subgrade soils should be moisture conditioned 0 to 3 percent above optimum content, and compacted to at least 95 percent relative compaction, just prior to placement of aggregate base.

5.1.5 Pipe Bedding and Trench Backfill

Pipe bedding should consist of sand or similar granular material having a sand equivalent value of 30 or more. The sand should be placed in a zone that extends a minimum of 6 inches below and 12 inches above the pipe for the full trench width. The bedding material should be compacted to a minimum of 90 percent of the maximum dry density. Trench backfill above pipe bedding may consist of approved, onsite or import soils placed in lifts no greater than 8 inches loose thickness and compacted to 90 percent of the maximum dry density.

5.1.6 Temporary Slopes for Shallow Excavations

Shallow, temporary utility trench excavations are anticipated for installation of the required utility lines. If very steep or vertical-sided excavations in excess of 5 feet deep are necessary, we recommend the sidewalls be shored in accordance with OSHA standards to provide temporary trench stability during construction. The contractor should be responsible for the structural design and safety of the temporary shoring system and we recommend that this design be submitted to Kleinfelder for review and approval.

For preliminary planning of OSHA sloping and shoring requirements, we recommend that fill materials be considered as Type C soils and that native formational materials be considered as Type B soils. The actual OSHA soil type should be determined by the contractor's "competent person" based on conditions exposed in the field.

Heavy construction loads, such as those resulting from stockpiles and heavy machinery, should be kept a sufficient distance away from the top of the excavation or shoring to prevent unanticipated surcharge loading. All surface water should be diverted away from excavations.

5.1.7 Excavation Considerations

Excavation in the existing fill and San Diego Formation can generally be accomplished with conventional, heavy earthmoving equipment in good operating condition. Excavation into the very old paralic deposits is anticipated to be difficult due to cemented layers of cobbles and gravel. Potential excavations for utility trenches, soldier beams for shoring, through this unit may be particularly difficult due to cementation, cobbles and limited size of excavation.

5.1.8 Dewatering

Static groundwater was not encountered within the exploratory borings performed by Kleinfelder. Elevated moisture is possible from the landscape areas overlying relatively impermeable permeable formation soils. The regional groundwater table is probably in excess of 100 feet below the hospital campus. However, some minor seepage may be encountered in the excavations due to perched groundwater that may be located near the contact of differing geologic units or soil types. Perched groundwater, if encountered, may require collection, control and disposal of minor amounts of water.

5.2 TEMPORARY SHORING

While the details of site excavation and temporary excavation support are not known at this time, we anticipate that the proposed excavation will require temporary shoring around most, of the perimeter of the site during construction to protect existing improvements such as buildings, roads and hardscape areas, utilities, power poles, etc. Shoring the northeastern side would not be required shoring as the excavation should daylight to the lower parking lot. Depending on the proximity to the existing building on the east, underpinning may also be necessary. Depending on final plans, the shoring height may be on the order of approximately 55 feet which would require several rows of tieback anchors.

Shoring is anticipated to consist of closely-spaced soldier piles and wooden lagging as it is typically the most feasible and economical method for soils in this area. While soil nailing is technically feasible, permanent encroachment into City streets is unlikely and the cemented cobble conglomerate of the old paralic deposits would be problematic. If tiebacks are required to limit lateral deflections due to the shoring height and surcharge loads, the City of San Diego would require removal of the upper portion of the anchors within their right-of way. Drilling holes for soldier beams through the very old paralic deposits should consider the appropriate diameter to remove cobbles and the effort to penetrate cemented zones. Based on our experience with nearby projects, the caving potential of the onsite soils is low to moderate for undocumented fill and low for formational materials. Recommendations for this system are provided in the following sections.

To accommodate installation of the soldier beams, wide-flange beam sections may be installed into pre-drilled holes surrounded by concrete below excavation depth and cement slurry within the lagging depths. In the unanticipated event that caving of the drilled holes occurs, drilling slurry or casing may be required. In addition, caving of drilled holes for tieback anchors is possible in undocumented fill.

5.2.1 Tieback Anchors

Tiebacks derive their load capacity through frictional resistance along the grouted “bonded zone” which is located beyond the active wedge. For design of tieback anchors, we recommend the active wedge may be assumed at an angle of 30 degrees from vertical, passing through a point located at least 5 feet behind the bottom of the excavation. We recommend the portion of the anchor within the “unbonded zone” within the active wedge either have a sleeve so that it is not bonded to the grout or be backfilled using sand/cement slurry. The shoring contractor should determine the suitable drilling method for tieback installation based on the subsurface conditions at the site and on their experience with similar materials. Tiebacks should be installed at angles between 15 and 30 degrees from horizontal.

Since the load-carrying capacity of the tieback anchors will depend on various site-specific equipment and method-related factors, design tieback capacities should be confirmed by performance testing. We recommend performance testing and proof testing of anchors be performed in accordance with the latest edition of the Post-Tensioning Institute's (PTI) Recommendations for Prestressed Rock and Soil Anchors. Performance test for the anchors shall be at a minimum of two (2) times the design loads and shall not exceed 80 percent of the specified minimum tensile strength of the anchor rod. A creep test is required for all pre-stressed anchors that are performance tested. All production anchors shall be tested at 150 percent of design loads and shall not be greater than 70 percent of the specified minimum tensile strength of the anchor rod

Based on experience from nearby projects, the allowable unit friction between grout and the soil may be assumed to be on the order of 2,000 to 3,000 psf for design of tiebacks if post-grouting is performed and depending on depth of overburden. However, it is important that the unbonded length not be grouted if post-grouting is performed. If post-grouting is not performed, we recommend an allowable unit friction of 1,000 psf be used. If tieback anchors are installed at an angle below the horizontal, tieback resistance should be taken as the horizontal component of the total anchor capacity. Additionally, the shoring designer should be aware that the vertical component of the total anchor capacity may act as a downward load on the shoring system.

5.2.2 Timber Lagging

Timber lagging may be used between the soldier piles to support the exposed soils. Since the lagging is generally left in place, treated lumber should be used. Lagging should be designed for the lateral pressures recommended in Section 5.2.3 of this report. The soil bridging between the stiffer soldier beam elements and the intermediate wooden lagging results in a reduced lateral earth pressure in the lagging area. The CBC allows soil arching effects to be considered in the design of lagging. We recommend 0.6 of the design earth pressure, or a uniform pressure of 400 psf for level ground without surcharge. It is our understanding that conventional design of earth pressures for soldier beam and wooden lagging walls incorporate this pressure into standard design tables for wooden lagging but do not include surcharge pressures. Therefore, additional surcharge pressures should be added if these tables are utilized.

If possible, structural walls should be cast directly against the shoring, eliminating the need for backfilling a narrow space. Voids between the soil and lagging should be grouted to mitigate the potential for the voids to propagate to the surface and to protect the existing improvements. Voids identified during lagging operations may also be immediately filled by compacting soil behind the lagging.

5.2.3 Lateral Pressures

Cantilever shoring can typically be used for retained shoring heights up to about 12 to 15 feet with level backfill and no surcharge loading, however this height could be increased by using larger beams. Tie-back anchors would likely be needed for surcharges, higher walls or sloping backfill. Cantilever shoring supporting undocumented fill should be designed to resist an equivalent fluid weight of 35 pcf for level ground conditions in fill and 30 pcf for level ground conditions in formational material. Thirty percent of any areal surcharge adjacent to the shoring (including soil stockpiles) may be assumed to act as a uniform horizontal pressure against the shoring for cantilever conditions.

Tied-back shoring systems should be designed to resist a trapezoidal horizontal soil pressure with an equivalent fluid weight of 25H (in psf), where H is the wall height in feet. Increased loads due to ascending slopes should also be added to this. As an approximation, thirty percent of any permanent or temporary surcharge loads adjacent to the shoring (including existing structures, temporary soil stockpiles, material staging, construction trailers, etc.) may be assumed to act as a uniform horizontal pressure against the shoring. A better estimate of horizontal pressure resulting from foundations adjacent to the shoring system can be estimated using the method presented on Figure 5 (based on US Navy, 1986). Strip footings can be represented as line loads and spread footings can be represented as point loads. Special cases such as combinations of sloping and shoring or other surcharge loads (not specified above) may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical engineer on a case-by-case basis. The above pressures do not include hydrostatic pressures since groundwater was not encountered within the depths explored at the site.

All soldier piles should extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. An equivalent fluid unit weight of 400 pcf may be used or allowable passive pressure against soldier piles that extend below the level of excavation. To account for three-dimensional effects, the passive pressure may be assumed to act on an area 2 times the width of the embedded portion of the pile, provided adjacent piles are spaced at least 3 pile diameters center-to-center. Additionally, we recommend a factor of safety of 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 4,000 psf.

Lateral movement of a shored excavation will depend on the type and relative stiffness of the system used, the workmanship of the contractor, and other factors beyond the scope of this study. The shoring engineer should design the system to limit lateral movements and settlements so that effects on adjacent structures, existing utilities to remain in place, and other existing site improvements are minimized.

In addition to monitoring of existing structures, horizontal and vertical movements of the shoring system should also be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor's responsibility. At a minimum, we recommend that the tops of the soldier beams be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the foundation is completed. The soldier beams should be surveyed at spacings no greater than 50 feet on-center.

5.3 FOUNDATION RECOMMENDATIONS

Foundations should be supported entirely on either formational material or compacted fill to mitigate the potential for differential settlement. Based on the currently proposed depth of excavation, very dense soils of the San Diego Formation should be present at foundation elevation. The structure may be supported on spread and continuous foundations, or a mat foundation.

5.3.1 Spread Foundations

For foundations entirely in formational soils, an allowable foundation pressure of 6,000 may be used. For foundations entirely in compacted fill soils such as near surface retaining walls or equipment pads, an allowable foundation pressure of 3,000 pounds per square foot (psf) can be used. The allowable design bearing values can be increased by one third for transient loading due to seismic and wind forces.

Anticipated total settlements will be evaluated when foundation loads are provided but are not expected to exceed 1 inch. Differential settlements over a 40-foot span are not expected to exceed 50 percent of the total settlement. Shallow foundations should contain reinforcing steel as determined by the project structural engineer.

Resistance to horizontal loadings can be developed by passive earth pressure on the sides of footings and frictional resistance developed along the footings bottoms. Passive resistance to lateral earth pressures may be calculated using an allowable equivalent fluid unit weight of 450 psf within formational soils or 350 psf within compacted fill. A frictional coefficient of 0.35 may be applied to vertical dead loads supported either formational or fill soils. The passive pressure and frictional resistance can be combined without reduction to resist lateral loads.

Footings may experience a reduction in bearing capacity or an increased potential to settle when located near existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse, and/or a loss of serviceability. To reduce this risk, utility excavations should not extend below a 2H:1V plane projected downward from 12 inches above the bottom of the outside edge of the footing. Also, no parallel utility excavations should be made within a lateral distance of 18 inches outside the footing.

Prior to placing reinforcing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. All footing excavations should be observed by the project geotechnical engineer immediately prior to placement of reinforcing steel and concrete to check that the recommendations contained herein are implemented during construction.

5.3.2 Mat Foundations

If utilized, a mat foundation bearing on formational materials may be designed using an allowable bearing pressure of 10,000 psf. We recommend an uncorrected vertical modulus of subgrade reaction for a one-foot square plate, k_1, of 275 pci be used for design of the mat foundation. The k_1 value is based on a settlement of 1 inch for a one-foot square plate and should be modified to account for the mat foundation width, B. The modified vertical modulus of subgrade reaction may be calculated using the following equation with actual mat dimensions:

$$k_s = k_1 \left(\frac{B+1}{2B} \right)^2$$

Where:

k_1 is the subgrade modulus for a one-foot square foundation (= 275 pci).

k_s is the subgrade modulus in pci for a foundation with width B (B is the least dimension of the mat in feet). This value approaches 0.25 k_1 for large dimensions.

The allowable design bearing value can be increased by one third for short-term loading due to seismic and wind forces. The mat foundation should have a minimum embedment of 18 inches and should be designed by a licensed structural engineer in the state of California for the specific loading conditions.

5.4 INTERIOR CONCRETE SLABS-ON-GRADE

Interior concrete slabs-on-grade may be used with structures not supported on mat foundations. Engineered fill or disturbed formational soils supporting concrete slabs should be scarified to a depth of 6 inches, moisture conditioned to within 3 percent above optimum and compacted to at least 95 percent relative compaction per ASTM D1557. A modulus of subgrade reaction, k, of 175 pounds per cubic inch (pci) may be used to design floors, pavements, and walkways on the compacted subgrades. For slabs subjected to pedestrian-type loadings, we recommend a minimum floor slab thickness of 4 inches.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, and/or curling of the slabs. High water-cement ratio and/or improper curing will increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

The floor slab should be underlain by at least 4 inches of clean coarse sand or fine gravel to provide a capillary moisture break and uniform support to the slab. In cases where the floor may have a vapor/moisture sensitive covering (e.g. tile, linoleum, carpet, wood), may be in a humidity controlled environment, or may likely have one or both of these conditions in the future, we recommend a polyolefin vapor barrier membrane be utilized between the prepared subgrade and the bottom of the floor slab.

Subsurface moisture and vapor naturally migrate upward through the soil. Where the soil is covered by a building or pavement, this subsurface moisture will collect and transmit through the concrete slab-on-grade. Traditional Visqueen vapor barriers may be considered marginally effective and have been shown to eventually disintegrate with time. To reduce the impact of this subsurface moisture and the potential impact of future introduced moisture (such as landscape irrigation or precipitation) we recommend utilizing a polyolefin vapor barrier membrane between the subgrade and slab-on-grade. This vapor barrier membrane should consist of a polyolefin sheeting at least 15 mil in thickness, have a water vapor permeance less than 0.01 perms (ASTM F1249), a puncture resistance of at least 2,200 grams (ASTM D1709), and a tensile strength of at least 45 lbf/in (ASTM D882).

The polyolefin vapor barrier membrane described above should be highly resistant to tearing, cracking, flaking, or puncturing during construction and should not disintegrate with time. A granular subbase below the membrane or a sand or gravel layer on top of the membrane is not required. In accordance with recommendations in ACI guidelines and many flooring companies, placement of the concrete slab may be directly on the vapor barrier. This eliminates the potential for water to be trapped in the blotter layer that could later be transmitted through the slab and adversely affect the flooring system. However, a reduced joint spacing, slab reinforcement, a low shrinkage mix design, and/or other measures to reduce the potential for slab curl should be implemented by the concrete slab designer.

We recommend that the vapor barrier be installed in accordance with ASTM E1643, "Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs". Some salient features of ASTM E1643 are discussed below. All joints and seams should have a minimum 6-inch overlap and be taped. The area of tape adhesion should be free from dust, dirt, and moisture. All penetrations must be sealed using a combination of membrane, tape, and mastic. The tape and mastic used should conform to the vapor barrier manufacturer's recommendations. Care should be taken at the lateral terminations so that vapors do not go around the membrane. This may be accomplished by placing the membrane on top of the footing and against the vertical wall so that the membrane will be sandwiched between the footing, vertical wall, and poured concrete floor slab. If damaged, the membrane should be repaired prior to placing concrete.

It is emphasized that we are not floor moisture-proofing experts. We make no guarantee nor provide any assurance that use of the capillary break will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level. The builder and designers should consider all available measures for slab moisture protection. Exterior grading and/or adjacent landscaping have an impact on the potential moisture beneath floor slabs. Exterior grading and/or adjacent landscaping should be designed to address the potential for increased moisture below moisture sensitive slabs and should at least reference the recommendations contained in Section 5.8 (Site Drainage) of this report.

5.5 PERMANENT BASEMENT AND SITE RETAINING WALLS

Permanent restrained retaining walls for the subterranean levels will likely be placed both against the temporary shoring in areas where used and be backfilled in potential areas where temporary excavation slopes are used. After permanent bracing such as floor slabs have been installed, it is important that the tieback anchors used for the shoring (if any) be detentioned and documented by the geotechnical engineer. The upper portion of the tiebacks will also need to be removed from City right-of-way. Cantilever retaining walls are not anticipated due to level ground conditions but recommendations are provided in the following sections if site walls are included in project design.

The basement walls should be dampproofed and permanent drainage provisions should be made against the walls. Vertically placed composite geosynthetic drainage panels are typically used rather than open graded gravel. The wall drains should be connected to closed conduits at the base of the walls and brought to a storm drain, sump or other suitable discharge location.

5.5.1 Wall Foundations

The perimeter basement walls will be supported on the foundation for the building. Recommendations for foundation design were previously provided in Section 5.3 of this report.

Although not anticipated, cantilevered masonry and poured-in-place concrete walls with shallow reinforced concrete footings are considered suitable for potential site retaining walls. Cantilevered concrete retaining walls may be supported on shallow continuous footings founded entirely on either engineered fill or undisturbed native formational soil. Where existing undocumented fill soils are present, the fill should be removed and recompacted so that retaining walls over 3 feet in height are supported by engineered fill. Shallow foundations supported on engineered fill or formation should be designed for an allowable bearing pressure of 3,000 psf. Estimated total settlements for retaining walls constructed in accordance with the recommendations contained herein are anticipated to be less than ¾ inch. Differential settlements are expected to be less than ½ inch within 40 feet.

Retaining wall foundations should be embedded at least two feet below the lowest adjacent grade or to the depth necessary to provide adequate factors of safety against sliding and overturning as determined by the retaining wall designer, whichever is greater. Reinforcement should be provided as required by the Regional Standard Drawings (if used) or as directed by the wall designer for load carrying purposes.

All footings should be extended in depth as necessary so that no existing or proposed utility trenches will extend below a plane having a downward slope of 2H:1V from 12 inches above the bottom edge of the footing. In addition, no parallel trenches should be within 18 inches from the closest edge of the footing. New footings should not be excavated below the bottom of adjacently located existing foundations. For all retaining walls, we recommend a minimum horizontal distance from the outside face of the footing to daylight of 6 feet.

5.5.2 Static Lateral Earth Pressures

Lateral pressures acting against masonry and poured-in-place concrete retaining walls can be calculated using soil equivalent fluid weight (efw). The efw value used for design depends on allowable wall movement. Walls that are free to rotate at least 0.5 percent of the wall height can be designed for the active efw. Retaining walls that are restrained at the top (such as basement walls), or are sensitive to movement and tilting should be designed for the at-rest efw.

Values given in the Table 2 below are in terms of equivalent fluid weight and assume a triangular distribution. These values assume that the wall is cast against the temporary shoring, or onsite or imported, sandy soils (SP, SM, SC) will be used as backfill and that the backfill is well drained and above the static water table. If walls with undrained backfill are to be used Kleinfelder should be consulted for additional evaluation and recommendations.

Table 2

Equivalent Fluid Weights (efw) for Calculating Lateral Earth Pressures

Wall Type	Conditions	Level Backfill (psf)
Restrained - Basement Wall	At-Rest	55
Cantilever - Site Wall	Active	35

Fifty and thirty percent of any uniform areal surcharge placed at the top of the wall may be assumed to act as a uniform horizontal pressure over the entire wall for the at-rest and active cases, respectively. As a minimum, we recommend that a traffic surcharge equivalent to 2 feet of soil backfill be assumed as a surcharge for the at-rest condition. For this condition a pressure of 120 psf may be assumed to act as a uniform horizontal pressure over the entire height of the wall, H.

For passive resistance on retaining wall foundations, we recommend using an allowable equivalent fluid weight of 450 psf for undisturbed formational material or 350 pcf for footings poured neat against properly compacted engineered fill. The upper 12 inches of material in areas not protected by adjacent concrete slabs or footings should not be included in design for passive resistance to lateral loads. The allowable coefficient of friction between the bottom of the footing and formational soil or engineered fill can be assumed as 0.35. The passive pressure and frictional resistance can be combined to resist lateral loads, provided that the larger value is reduced by 50 percent.

5.5.3 Seismic Design of Retaining Walls

Retaining walls should be designed to resist dynamic earth pressures from earthquake loading. For both cantilever and restrained conditions, walls can be designed using an incremental seismic force of $12H^2$ for the Design Earthquake PGA (in pounds per linear foot of wall length, with H as the wall height in feet), which are additive to the static active earth pressure described above. The incremental seismic force acts at 0.5H above the base of the wall.

Allowable bearing pressure values described in previous sections of this report can be increased by one-third when calculating resistance caused by loads of short duration, such as earthquake loads. Restraining passive pressure and friction values should not be increased by this amount, but a lower factor of safety than is normally applied to static loads could be used. This factor of safety for dynamic load conditions should not be less than 1.2.

5.5.4 Wall Drainage

The above-recommended values do not include lateral pressures due to hydrostatic water pressures generated by infiltrating surface water that may accumulate behind the walls. Therefore, wall backfill materials should be free draining and provisions should be made to collect and remove excess water that may accumulate behind earth retaining structures.

Wall drainage may be provided by free-draining gravel surrounded by non-woven synthetic filter fabric or by prefabricated, synthetic drain panels. In either case, drainage should be collected by collector pipes at the base and directed to a sump, storm drain, weep hole(s), or other suitable location for disposal. The drainage discharge should not be permitted to discharge over soil in a manner that would cause erosion. If utilized, we recommend that drainage gravel consist of durable stone having 100 percent passing the 1-inch sieve and zero percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a permeability of at least 0.02 centimeters per second, a minimum flow rate of 50 gallons per minute per square foot of fabric, and a minimum puncture strength of 50 pounds. The geotextile manufacturer's recommendations should be followed for installation of a drainage fabric system.

5.6 EXTERIOR CONCRETE FLATWORK

All flatwork and exterior concrete should be supported on at least 12 inches of compacted, low to very low expansive engineered fill. The concrete slabs for walkways and sidewalks should have a nominal thickness of 4 inches. The exposed subgrade should be scarified to a depth of 6 inches, uniformly moisture conditioned to between optimum and 3 percent above optimum moisture content and compacted to at least 95 percent relative compaction as per ASTM D1557.

5.7 PRELIMINARY CORROSIVE SOIL SCREENING

A preliminary corrosive soil screening for the onsite materials was completed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory testing on two representative soil samples obtained during our current subsurface evaluation and review of previous nearby results. Laboratory testing was performed to evaluate soluble chloride, soluble sulfate content, resistivity and pH of soil. Results of the tests are provided in Appendix B.

Caltrans defines a “corrosive site” as one where one or more representative soil and/or water samples contain concentrations of soluble chloride of 0.05 percent (by weight) or greater, soluble sulfate concentrations of 0.2 percent or greater or the pH is 5.5 or less. Based on the laboratory test results, this site is considered “non-corrosive” by the Caltrans definition. The Portland Cement Association (PCA, 1988) defines concrete exposure to sulfate attack as negligible for soil with a water soluble sulfate content of 0.00 to 0.10 percent (by weight), moderate for a sulfate content of 0.10 to 0.20 percent, severe for a sulfate content of 0.2 to 2.00 percent, and very severe for a sulfate content over 2.00 percent. Test results indicate concrete exposure to sulfate attack as low for this site.

Upon saturation, the minimum resistivity result of the test was 910 ohm-cm. A test result from our 2019 investigation for the lower replacement hospital to the northeast indicated a minimum resistivities of 1,000 and 1,800 ohm-cm. Resistivity values between 500 and 1,000 ohm-cm are considered corrosive and those between 1,000 and 2,000 ohm-cm are considered moderately corrosive.

We recommend that the corrosion test results be reviewed and evaluated by the project designers considering the proposed improvements and project lifespan requirements. A corrosion engineer can be contacted for detailed evaluation of corrosion potential and corrosion resistant design.

5.8 SITE DRAINAGE AND EROSION CONTROL

Final elevations at the site should be planned so that positive drainage is established around structures. Positive drainage is defined as a slope of 2 percent or more for a distance of 5 feet or more away from structure foundations. Roof gutters and downspouts should be installed on structures. Downspouts should discharge to controlled drainage systems and drainage gradients should be maintained to carry all surface water off the site. Ponding should not occur on the site.

Planters should be built so that water exiting from them will not seep into the foundation areas or beneath slabs and pavement. In any event, the maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to properly sustain the landscaping plants. Should excessive irrigation, waterline breaks, or unusually high rainfall occur, saturated zones and perched groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the foundation or landscaped areas. Potential sources of water, such as water pipes, drains, garden ponds, and the like, should be frequently examined for signs of leakage or damage. Any such leakage or damage should be repaired promptly.

5.9 SIGN AND LIGHT POLE SUPPORT

Proposed sign structures and light standard foundations as columns embedded in earth or embedded in concrete footings in the earth to resist both axial and lateral loads, can be designed in general accordance with Section 1807 of the 2016 California Building Code (CBC). We have conservatively assumed that foundations will be embedded in fill materials with the foundation properties as Class 4 Material as defined by the CBC, Table 1806A.2. We recommend that a lateral soil-bearing pressure of 150 lbs/ft^2 per foot of depth below natural grade be used. An allowable soil-bearing pressure of 2,000 lbs/ft^2 may be used to support vertical loads. The allowable lateral soil-bearing pressure may be increased by a factor of 2 for short-term lateral loads, as allowed in Section 1806A.3.4 of the 2016 CBC, provided the structures will not be adversely affected by a ½ inch of motion at the ground surface.

5.10 STORMWATER INFILTRATION STUDY

We have evaluated the site in conformance with the City of San Diego 2018 BMP Design Manual. For the purpose of this report, infiltration is defined as the flow of water through the ground surface and percolation is defined as the downward flow of water through the subsurface soil layers. Infiltration may be controlled primarily by factors such as the type and porosity of the surface filtering media, maintenance of these media, surface slope, surface vegetation, and intensity, duration, and type of precipitation. Percolation may be controlled primarily by the soil types and properties such as grain size and density, soil layering, porosity, hydraulic head, and the proximity to the groundwater. Surface drainage and maintenance will largely determine the site's infiltration rate and the amount of water that will infiltrate for any given storm. The percolation rate will depend locally on the soil layering and will be controlled by the finer grained soil layers.

Borehole percolation testing was the selected method for field infiltration testing at the site. Two percolation tests were performed at two different locations of the site. The percolation tests were performed in general accordance with those set forth in California Test 750, "Method for Determining the Percolation Rate of Soils Using a 6-Inch-Diameter-Test Hole". The tests were performed in drilled holes advanced to a depth of 5 feet below existing site grades. The measured percolation rates have been converted to an adjusted short-term infiltration rate based on borehole geometry using the Porchet Method (Ritzema, 1994) and are presented in Table 3. These values are converted to long term design infiltration rates later in this report by using correction factors based on Worksheet D.5-1 of the BMP Design Manual.

Table 3

Summary of Adjusted Infiltration Rates

Boring	Tested Depth from Ground Surface (feet)	Adjusted Short Term Infiltration Rate (inch/hour)	Soil Description
PERC-1	5	0.02	Silty Sand with Gravel
PERC-2	5	0.05	Silty Sand with Gravel

Note that relatively clean water was used to perform the tests above. However, surface runoff water from the site would likely contain silt, clay, oil and/or other materials that would eventually decrease the percolation rates. The provided field percolation rates in Table 3 do not include reduction factors for long term performance. These values are converted to long term design infiltration rates later in this report by using correction factors based on Worksheet D.5-1 of the BMP Design Manual.

Based on visual soil classifications and laboratory testing of the two soil samples collected during our field exploration at the percolation test locations, subsurface materials mostly consist of clayey and silty sand within the depths of the test.

5.10.1 Mitigation Measures

The following bullets present typical considerations (geotechnical and other) for implementation of infiltration systems, along with site specific conditions.

- Presence of fill soils below building footprint. The site is underlain by about 3 to 8 feet of fill, however the proposed building foundation depth will be approximately 55 feet from existing grade.
- Presence of shallow formational material. The site has very old paralic deposits approximately 3 to 8 feet in depth from the ground surface and the San Diego Formation approximately 10 to 13 feet in depth from the ground surface. Water from overlying BMPs would likely perch on the less permeable formation materials and move laterally to the more permeable material in utility trenches or wall backfill for the subterranean level.
- Building sites located adjacent to or within landslide hazard areas or hillside grading areas. The site is not located near landslide hazard areas.
- Sites with initial seasonal high groundwater elevation within 10 feet of the invert of a proposed basin. The sites are not within 10 feet of high groundwater table.
- Site soils with a moderate or high potential for liquefaction. The sites have a low potential for liquefaction.
- Site soils with a moderate or high expansion potential. The majority of observed soils within the infiltration test areas appear to have low expansion potential.
- Sloping sites. The location of potential BMPs are not currently known. The southern and western portions of the site are in flat ground.

- Sites with soil and/or groundwater contamination. According to the California State Water Resources Control Board Geo Tracker Database, contamination has not been identified in the near the project site.

5.10.2 Data Evaluation

The results of the field testing program provide a design infiltration rate based on correction factors contained within Tables D.5-1 and D.5-2 of the 2018 BMP Design Manal. For preliminary feasibility of infiltration purposes, a factor of safety of 2.0 is used with the non-factored infiltration rates.

5.10.3 Design Infiltration Rates

Based on our evaluation of the percolation test data discussed in a preceding section of this report, the soils encountered exhibit infiltration rates for *short-term, non-factored infiltration rates between **0.02** and **0.05** inch/hour*. The long-term design infiltration rate was calculated by using the following correction factors based on Worksheet D.5-1 of the BMP Design Manual. The completed worksheets are presented in Appendix C. Design infiltration rates have been estimated for PERC-1 through PERC-2 and the values are presented in Table 4. However, feasibility of infiltration for preliminary purposes a factor of safety of 2.0 must be used with the non-factored infiltration rates. For preliminary purposes the design infiltration rates are **0.01** and **0.03** inch/hour.

Table 4
Design Infiltration Rates

Boring	Safety Factor	Long Term Design Infiltration Rate (Inch/hour)
PERC-1	2.0	0.01
PERC-2	2.0	0.03

5.10.4 Recommendations and Conclusions

Based on the design infiltration rates less than 0.5 in/hr and the completed Geotechnical and Groundwater Investigation Requirement Worksheet C.4-1 contained in the BMP Design

Manual, we classify the site as a feasibility screening category of “No Infiltration”. The completed C.4-1 worksheets for each BMP location proposed at the site are included in Appendix C of this report.

Based on the field percolation testing, geotechnical observations, laboratory data, and completion of the BMP Manual Worksheets, it is our opinion that the project site is categorized as not suitable for infiltration. The site is underlain by very dense and cemented very old paralic deposits and the San Diego Formation. In our opinion, the underlying formation material has low void ratio and cementation therefore low permeability characteristics. We recommend that BMPs be lined with an impermeable liner and located as far away from proposed building foundations and utility trenches as feasible in order to minimize the potential effects of lateral migration of water.

6 ADDITIONAL STUDIES

The recommendations within this report were based on progress drawings and should be updated if final design differs from that assumed herein. The review of plans and specifications, and the observation and testing by Kleinfelder of earthwork related construction activities, are an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client will be assuming our responsibility for any potential claims that may arise during or after construction. The required tests, observations, and consultation by Kleinfelder during construction include, but is not limited to:

- A review of plans and specifications;
- Observation of site clearing;
- Construction observation and density testing of fill material placement, trench backfill, subgrade preparation, and aggregate base for pavements; and
- Observation of foundation excavations and foundation construction.

7 LIMITATIONS

This report has been prepared for the exclusive use of Scripps Health and their consultants for specific application to the subject project. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No warranty, express or implied, is made.

The scope of services was limited to the field exploration program described in this report. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions presented herein are based on field explorations, laboratory testing, engineering analyses and professional judgement.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues addressed in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, laboratory tests, and our understanding of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, or locations of the improvements, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid until the

changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, ground improvement, preparation of foundations, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on site and off site), or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

Kleinfelder will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials. Additional important information about this report is presented in the attached Geotechnical Business Council insert in Appendix G.

8 REFERENCES

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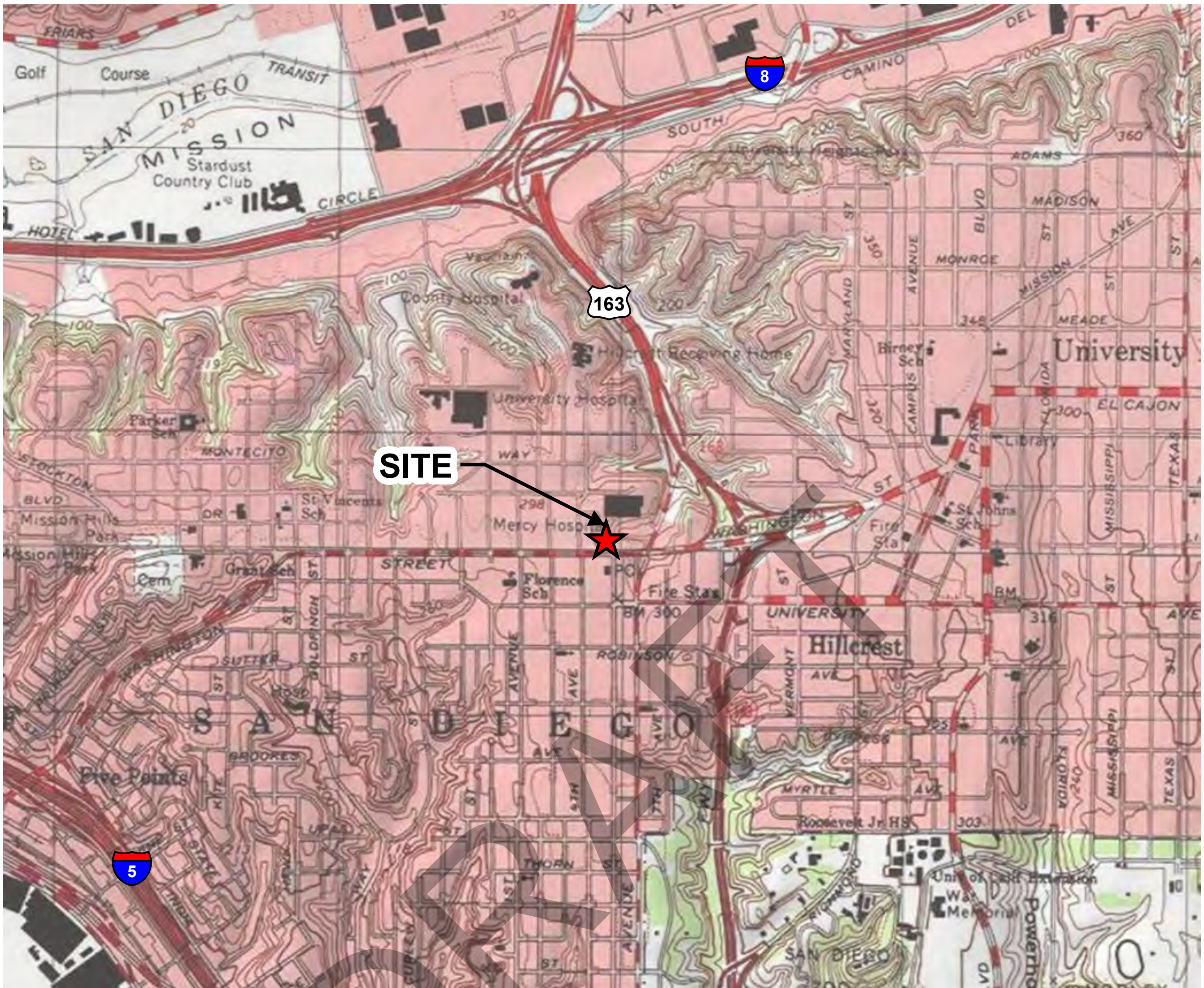
Kleinfelder, "Supplemental Geotechnical Investigation, Proposed Emergency Department Expansion, Scripps Mercy Hospital, 4077 5th Avenue, San Diego California," dated March 6, 2007.

Kleinfelder, "Report Geologic and Geotechnical Engineering Evaluation, Replacement Hospital, Scripps Mercy Hospital, 4077 5th Avenue, San Diego, California," dated April 26, 2019.

FIGURES



N

0 1,000 2,000 4,000
Feet

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PROJECT NO.	20194096
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FILE NAME:	20194096_Vic_Merc.mxd

VICINITY MAP

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
1

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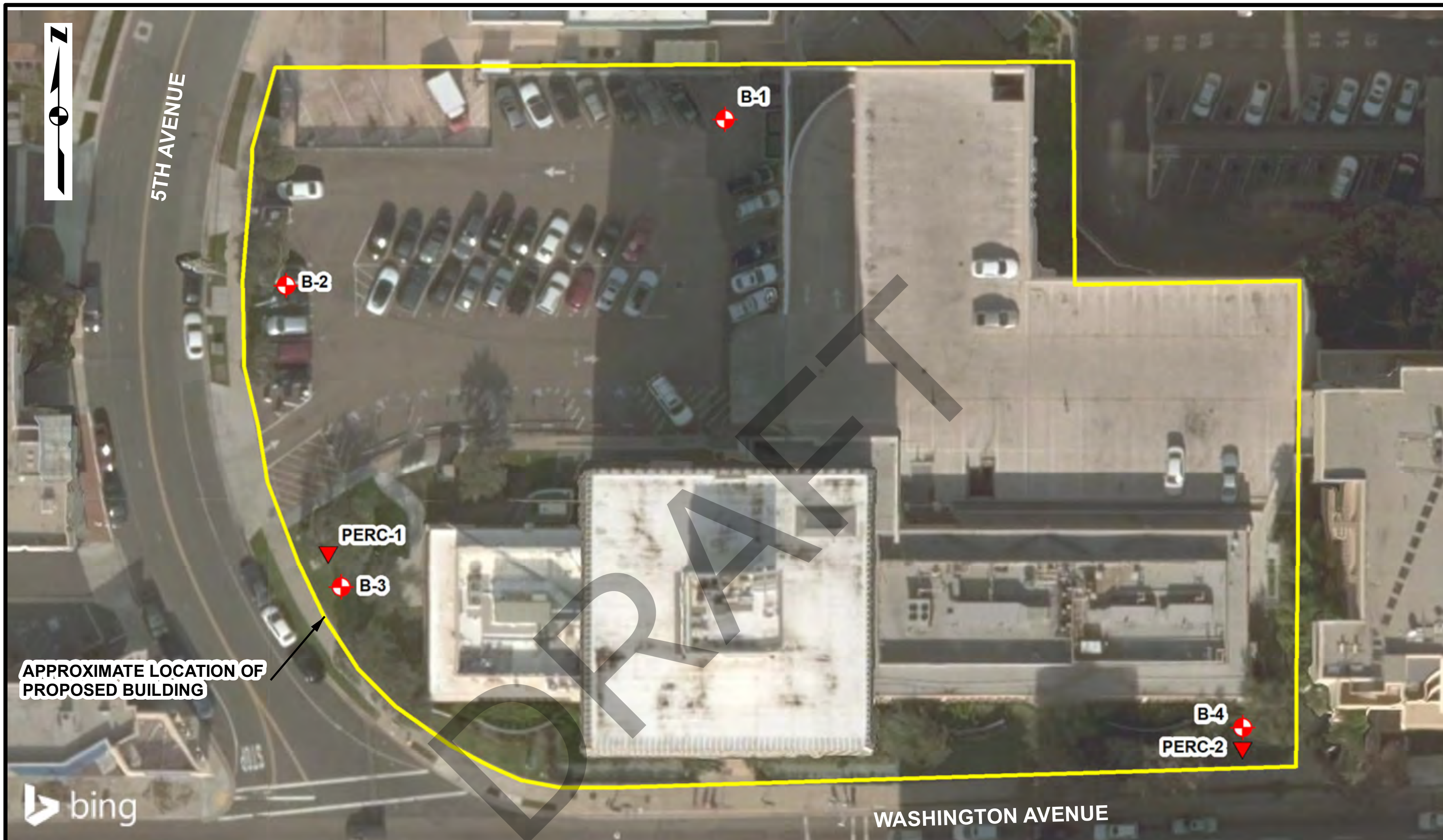

LEGEND

APPROXIMATE LOCATION OF BORING

APPROXIMATE LOCATION OF PERCOLATION TEST

0 25 50 100 Feet

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FILE NAME:	20194096_Site_Merc.mxd

EXPLORATION MAP

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
2

LEGEND

- af - ARTIFICIAL FILL
- Qya - YOUNG ALLUVIAL FAN DEPOSITS
- Qvop - VERY OLD PARALIC DEPOSITS, UNITS 6, 8, 9, AND 10
- Tsd - SAN DIEGO FORMATION
- Tmv - MISSION VALLEY FORMATION
- Tp - POMERADO CONGLOMERATE

0 1,000 2,000 Feet

N

SOURCE:
GEOLOGIC MAP OF THE SAN DIEGO 30'X60' QUADRANGLE, CALIFORNIA, KENNEDY AND TAN 2008.

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REGIONAL GEOLOGIC MAP

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE 3

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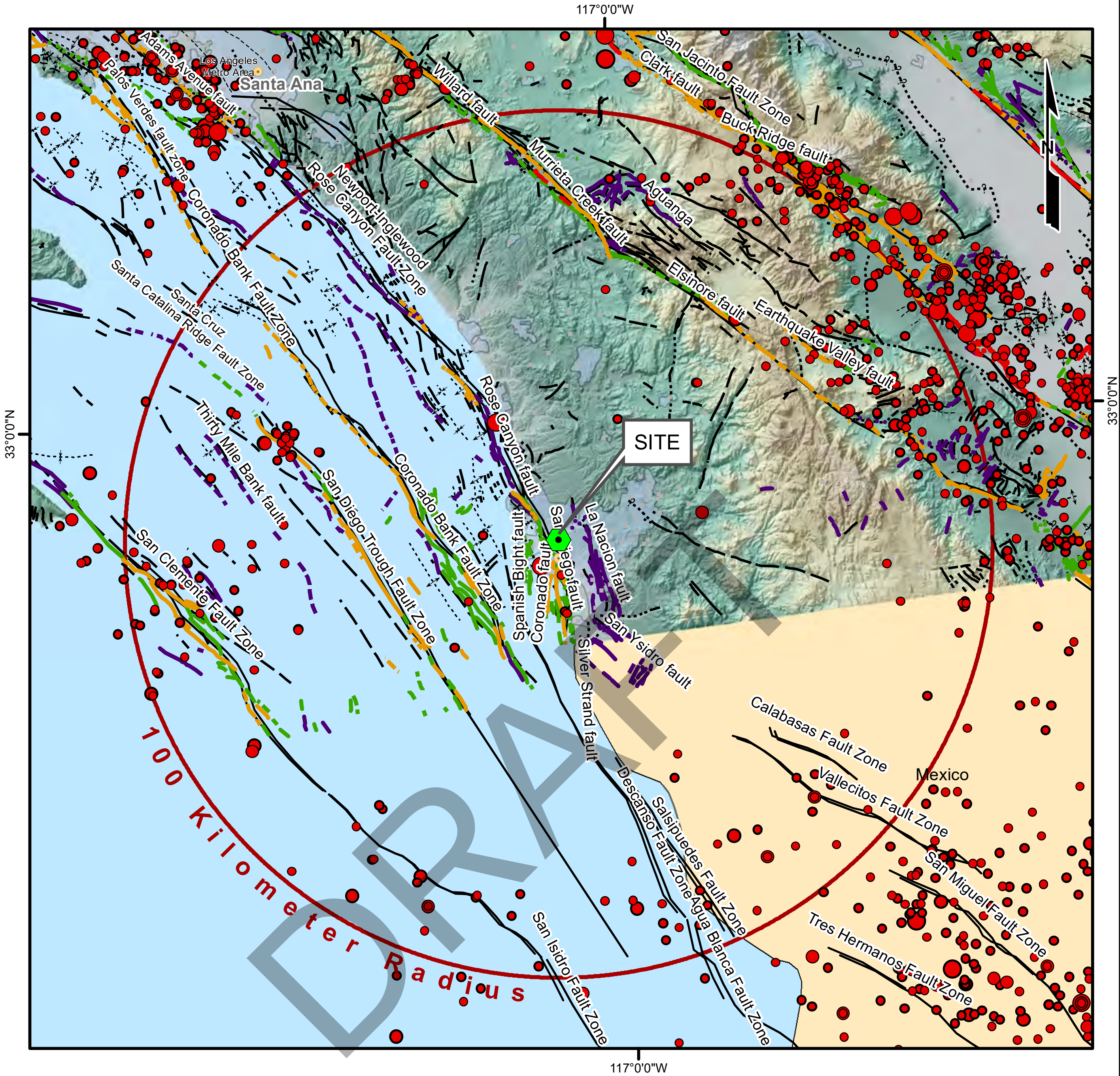

Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2009)

Historic displacement (< 200 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Holocene displacement (< 11,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Quaternary displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

ANSS Earthquakes

Magnitude
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7.0 - 7.9
- 8.0 - 8.9

35 17.5 0 35
Kilometers

\\sandiego\SANDIEGO-DATA\GRAPHICS_clients\Scripps\20194096_Mercy\mxd\20194096_EQ_Merc.mxd

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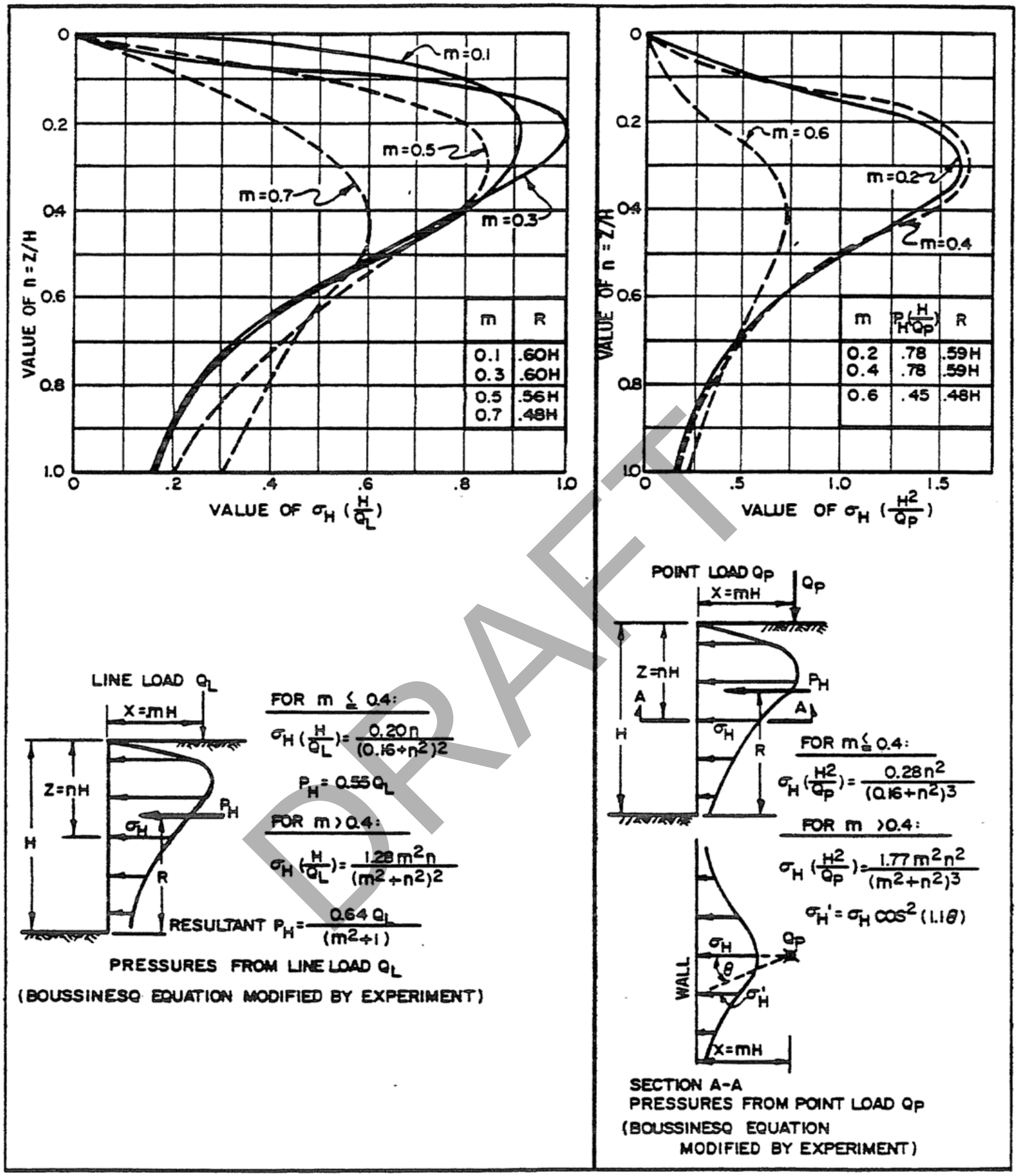
PROJECT NO.	20194096
DRAWN:	6/25/2019
DRAWN BY:	JP
CHECKED BY:	ST
FILE NAME:	20194096_EQ_Merc.MXD

REGIONAL FAULT MAP
AND EARTHQUAKE EPICENTERS
(1800 - MAY 2019)

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
4



KLEINFELDER Bright People. Right Solutions.	PROJECT NO. 20194096 DRAWN BY: T.Cisney CHECKED BY: K. Crennan DATE: 6/25/2019 REVISED: -	**Horizontal Pressures on Rigid Wall from Surface Load (US Navy, 1986)** HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	FIGURE **5**

APPENDIX A

Boring Logs

APPENDIX A
BORING LOGS

The field exploration consisted of a geologic reconnaissance of the site, drilling four borings to evaluate subsurface conditions, drilling two borings for infiltration testing, and collection of soil samples for laboratory testing. The exploration borings were drilled on May 11 and 18, 2019 to depths up to approximately 71 ½ feet and the two borings for infiltration testing were drilled to a depth of 5 feet. Due to access constraints at the site, the drilling contractor, Pacific Drilling of San Diego, used a limited access Fraste PL-G drill rig, with the exception of a truck-mounted Marl-10 which was used in the parking lot at boring B-1. Traffic control with a limited work window on Saturday was required to access the borings off Washington Street. The rig was equipped with 6-inch diameter hollow stem augers. The approximate locations of the borings are shown on the Exploration Map, Figure 2. We have also used information from previous borings near the project site in our analysis and have therefore included logs of these borings as part of this report in Appendix A.1.

The Unified Soil Classification System (USCS) was used for our soil descriptions, the chart and a Boring Log Legend are presented as Figures A-1 and A-2, respectively. The Boring Logs for the proposed hospital support building are presented as Figure A-3 through A-8. The Logs of Borings describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the boring number, drilling date, and the initials of the logger and name of drilling subcontractor. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and intact samples of representative earth materials were obtained from the borings.

In-place soil samples were obtained at the test boring locations using a California penetration sampler or a Standard penetration sampler (SPT) driven a total of 18-inches (or until practical refusal) into the undisturbed soil at the specified sample depth. The sampler was driven using a 140-pound auto-hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count (or N-value) and is recorded on the Boring Logs. Please note that these blow counts have not been adjusted for the effects of overburden pressure, input driving energy, rod length, sampler correction, or boring diameter correction. In addition, to the California and SPT samples, we also obtained bulk samples from the drill cuttings.

An engineer from our office supervised the field operations and logged the borings according to the Unified Soil Classification System (USCS). The boundaries shown between soil types on the logs and cross sections are approximate as the transition between different soil layers may be gradual. Therefore, variations in the subsurface profile should be anticipated throughout the site. The samples retrieved from the borings were sealed, labeled, and transported to our laboratory for further evaluation and testing. Upon completion of the drilling operations, the boreholes were backfilled and the drums were profiled and disposed of, as required by the County of San Diego Department of Environmental Health (DEH).

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu≥4 and 1≤Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu<4 and/ or 1>Cc>3	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu≥4 and 1≤Cc≤3	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu<4 and/ or 1>Cc>3	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (Half or more of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu≥6 and 1≤Cc≤3	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu<6 and/ or 1>Cc>3	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu≥6 and 1≤Cc≤3	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu<6 and/ or 1>Cc>3	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (Half or more of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit 50 or greater)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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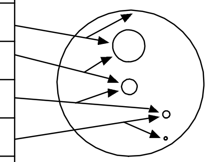
PROJECT NO.: 20194096	GRAPHICS KEY	FIGURE	
DRAWN BY: ST		A-1	
CHECKED BY:	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA		
DATE: 5/13/2019			
REVISED: -			

PLOTTED: 06/25/2019 11:36 AM BY: TCisney
OFFICE FILTER: SAN DIEGO
PROJECT NUMBER: 20194096.001A
gINT FILE: Klf_gint_master_2019
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2019.GLB [LEGEND 1 (GRAPHICS KEY) USCS_STANDARD]

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

	AMOUNT	
Term of Use	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25 ≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5 ≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1 ≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2 ≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4 ≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

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PROJECT NO.: 20194096	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: ST		
CHECKED BY:	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	A-2
DATE: 5/13/2019		
REVISED: -		

Date Begin - End:	5/11/2019	**Drilling Company:**	Pacific Drilling		**BORING LOG B-1**
Logged By:	S.Tena	**Drill Crew:**	Ryan & Toby		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	MARL-10	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	6 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 290.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0	**ASPHALT**: (4 INCHES), No base observed												**pH=** 8.7
		ARTIFICIAL FILL (af) **Clayey SAND (SC)**: medium-grained, subangular gravel, low plasticity, brown (7.5YR 4/3), moist	S1			48"		7.6		79	22			**Resistivity=** 910 ohm-cm **Sulfates=** 170 ppm **Chlorides=** 53 ppm Hand auger to 2 feet and stopped due to a 4-inch cobble.
285	5	very dense	S2		BC=9 50/6"	5"								
		VERY OLD PARALIC DEPOSITS($Qvop_9$) **Clayey SAND with Gravel (SC)**: medium-grained, subangular gravel, low plasticity, brown (7.5YR 4/3), moist, weakly to moderately cemented												Rocky from 7 to 9 feet.
280	10	**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: fine-grained, non-plastic, pale brown (2.5Y 7/4), moist, very dense -rock fragements inside sampler	S3		BC=50/4"	4"								
275	15	-sampler bouncing on a rock			BC=50/0"	NR								
270	20	**SAN DIEGO FORMATION (Tsd)** **Sandy SILT (ML)**: non-plastic, light gray (5Y 7/1), moist, very stiff, weakly cemented, iron oxide staining	S4		BC=19 28 39	18"		12.2	97.0	100	74			
265	25	-mottled with dark grayish brown (10YR 4/2)	S5		BC=18 36 50	18"								
260	30		S6		BC=22 41 50	18"		12.6	94.3	100	65			**DIRECT SHEAR TEST** Cohesion= 650 psf Friction Angle= 33°
255	35		S7		BC=17 32 46	18"								

DRAFT

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PROJECT NO.: 20194096	BORING LOG B-1		FIGURE
DRAWN BY: ST			
CHECKED BY: KC	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA		A-3
DATE: 5/13/2019			
REVISED: -			

PLOTTED: 06/26/2019 09:06 PM BY: TCisney

Date Begin - End:	5/11/2019	Drilling Company:	Pacific Drilling	BORING LOG B-1
Logged By:	S.Tena	Drill Crew:	Ryan & Toby	
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	MARL-10	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger	
Weather:	Cloudy	Auger Diameter:	6 in. O.D.	

Approximate Ground Surface Elevation (ft.): 290.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			SAN DIEGO FORMATION (Tsd) **Sandy SILT (ML)**: non-plastic, light gray (5Y 7/1), moist, very stiff, weakly cemented, iron oxide staining	S8		BC=18 27 36	18"				100	59			
245	45			S9		BC=18 35 47	18"								
240	50		subrounded gravel, pale yellow (5Y 7/3)	S10		BC=21 34 45	18"								
			-lense of gravel layer from 52 to 54 feet												Effort of drilling increased from 52 to 54 feet.
235	55		light gray (5Y 7/1)	S11		BC=25 42 50	18"		7.4	100.8					
230	60		-gravel layer from 60 to 61 feet												
			-sampler bouncing during sampling			BC=50/0"	NR								
225	65		gray (5Y 6/1), rock fragments	S13		BC=37 30 50	14"								
220	70		pale olive (5Y 6/4)	S14		BC=15 27 35	14"		27.0	95.0	100	66			**DIRECT SHEAR TEST** Cohesion= 700 psf Friction Angle= 32°
215	75														

DRAFT

The boring was terminated at approximately 71.5 ft. below ground surface. The boring was backfilled with bentonite chips and patched with concrete on May 11, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.: 20194096	BORING LOG B-1	FIGURE
DRAWN BY: ST		
CHECKED BY: KC	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	A-3
DATE: 5/13/2019		
REVISED: -		PAGE: 2 of 2

BORING LOG B-2

Date Begin - End:	5/11/2019	**Drilling Company:**	Pacific Drilling
Logged By:	S.Tena	**Drill Crew:**	Ryan & Toby
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	MARL-10
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Cloudy	**Auger Diameter:**	6 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

Approximate Ground Surface Elevation (ft.): 291.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		ASPHALT: (4 INCHES), No base observed												Hand auger to 2.5 feet and stopped due to hard ground conditions. **Expansion Index=** 0
290		**ARTIFICIAL FILL (af)** **Clayey SAND (SC)**: medium-grained, low plasticity, reddish brown (5YR 4/4), moist, trace of gravel	S1			24"		13.3						
		VERY OLD PARALIC DEPOSITS(Qvop₉) **Silty SAND with Gravel (SM)**: non-plastic, yellow (10YR 7/6), moist, gravel and cobbles very pale brown (10YR 7/4), very dense, with cobbles	S2			24"								
285	5		S3		BC=19 50/6"	8"								Hard drilling from 6 to 10 feet.
280	10	**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: fine-grained, non-plastic, light gray (5Y 7/2), moist, dense, weak cemented	S4		BC=7 15 17	18"								
275	15	-rock at tip of sampler			BC=50/4.5"	NR								
		light gray (5Y 7/1), very dense, iron oxide staining	S5		BC=16 34 50	18"		10.1	94.5					
270	20													
265	25													
260	30													

The boring was terminated at approximately 20 ft. below ground surface. The boring was backfilled with auger cuttings and patched with concrete on May 11, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

DRAFT

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PROJECT NO.: 20194096	BORING LOG B-2	FIGURE
DRAWN BY: ST		A-4
CHECKED BY: KC	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	
DATE: 5/13/2019		
REVISED: -		PAGE: 1 of 1

BORING LOG B-3

Date Begin - End:	5/18/2019	**Drilling Company:**	Pacific Drilling
Logged By:	S.Tena	**Drill Crew:**	Gerardo & Ryan
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Cloudy	**Auger Diameter:**	6 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

Approximate Ground Surface Elevation (ft.): 291.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		TOPSOIL: (6 INCHES)												
290		**ARTIFICIAL FILL (af)** **Silty SAND (SM)**: medium-grained, non-plastic, reddish brown (5YR 4/4), moist	S1			42"								
		Clayey SAND (SC): fine to medium-grained, low plasticity, reddish brown (5YR 4/3), moist												
	5	**VERY OLD PARALIC DEPOSITS($Qvop_9$)** **Silty SAND with Gravel (SM)**: subangular gravel, non-plastic, reddish yellow (7.5YR 6/6), moist, very dense, with cobbles	S2		BC=10 29 50/3"	15"								Hard drilling at 4 feet due to gravel and rock.
285		-rock fragments in soil cuttings medium-grained, strong brown (7.5YR 5/6), rock fragments												

The boring was terminated because of refusal (↑) at approximately 8.5 ft. below ground surface on on two attempts. The boring was backfilled with auger cuttings on May 18, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Elevation scale: 290, 285, 280, 275, 270, 265, 260; Depth scale: 5, 10, 15, 20, 25, 30

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PROJECT NO.: 20194096	BORING LOG B-3		FIGURE
DRAWN BY: ST			A-5
CHECKED BY: KC	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA		
DATE: 5/13/2019			
REVISED: -			PAGE: 1 of 1

BORING LOG B-4

Date Begin - End:	5/18/2019	**Drilling Company:**	Pacific Drilling		
Logged By:	S.Tena	**Drill Crew:**	Gerardo & Ryan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	6 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 289.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0	**TOPSOIL**: (6 INCHES)												Hand auger to 2.5 feet and stop due to a cobble.
		ARTIFICIAL FILL (af) **Silty SAND (SM)**: non-plastic, strong brown (7.5YR 5/6), moist	S1			54"								
285	5	**Clayey SAND (SC)**: low plasticity, brown (7.5YR 4/3) with reddish yellow (7.5YR 6/6), moist, medium dense -rock fragments inside sampler medium-grained, subrounded boulders, brown (7.5YR 4/2), some clay content	S2		BC=3 3 16	8"		12.4				22	4	
280		**VERY OLD PARALIC DEPOSITS(Qvop$_9$)** **Clayey SAND with Gravel (SC)**: fine-grained, low plasticity, light yellowish brown (10YR 6/4), moist, very dense												Hard drilling at 8 feet due to rocks and cobbles.
	10		S3		BC=17 50/3"	9"								
275		**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: fine-grained, non-plastic, light yellowish brown (2.5Y 6/3), moist, medium dense, micaceous												
	15		S4		BC=50 8 15	18"		19.9						
270	20	**SAN DIEGO FORMATION (Tsd)** **Sandy SILT (ML)**: fine-grained, light yellowish brown, moist, very stiff -No sample recovery			BC=50/2"	NR								
265	25	fine to medium-grained, light brownish gray (2.5Y 6/2), weak cemented, iron oxide staining	S5		BC=23 37 50/4"	16"	ML			100	58			**DIRECT SHEAR TEST** Cohesion= 750 psf Friction Angle= 31°
260	30	fine-grained, gray (2.5Y 6/1)	S6		BC=29 50	12"								
255														

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PROJECT NO.: 20194096	BORING LOG B-4		FIGURE
DRAWN BY: ST			A-6
CHECKED BY: KC	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA		
DATE: 5/13/2019			
REVISED: -			

Date Begin - End:	5/18/2019	**Drilling Company:**	Pacific Drilling		**BORING LOG B-4**
Logged By:	S.Tena	**Drill Crew:**	Gerardo & Ryan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	6 in. O.D.		

Approximate Ground Surface Elevation (ft.): 289.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			SAN DIEGO FORMATION (Tsd) **Sandy SILT (ML)**: fine-grained, light yellowish brown, moist, very stiff	S7		BC=23 40 50/5"	16"								
250	40		light gray (5Y 7/1)	S8		BC=24 50	12"		12.7	90.1					
245	45			S9 S10		BC=23 34 50/3"	15"								
240	50		fine to medium-grained, light yellowish brown (2.5Y 6/4), trace of subrounded gravel	S11 S12		BC=19 37 50/4"	16"								
			-gravel lens at 52 feet												Hard drilling at 52 feet.
235	55														
230	60														
225	65														
220															

The boring was terminated at approximately 53 ft. below ground surface. The boring was backfilled with bentonite chips on May 18, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	5/18/2019	**Drilling Company:**	Pacific Drilling		**BORING LOG PERC-1**
Logged By:	S.Tena	**Drill Crew:**	Gerardo & Ryan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

FIELD EXPLORATION								LABORATORY RESULTS							
Approximate Elevation (feet)	Depth (feet)	Graphical Log	Approximate Ground Surface Elevation (ft.): 291.00 Surface Condition: Grass Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			TOPSOIL: (6 INCHES)												
290			**ARTIFICIAL FILL (af)** **Clayey SAND (SC)**: medium-grained, low plasticity, dark brown (7.5YR 3/2), moist												
			light brown (7.5YR 6/3)	S1			24"								
	5		**VERY OLD PARALIC DEPOSITS($Qvop_9$)** **Silty SAND with Gravel (SM)**: medium-grained, non-plastic, light brown (7.5YR 6/3), moist, rock fragments												
285															
	10														
280															
	15														
275															
	20														
270															
	25														
265															
	30														
260															

The boring was terminated at approximately 5 ft. below ground surface. The boring was backfilled with auger cuttings on May 18, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

PROJECT NO.: 20194096	BORING LOG PERC-1	FIGURE
DRAWN BY: ST		A-7
CHECKED BY: KC	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	
DATE: 5/13/2019		
REVISED: -		PAGE: 1 of 1

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Date Begin - End:	5/18/2019	**Drilling Company:**	Pacific Drilling		**BORING LOG PERC-2**
Logged By:	S.Tena	**Drill Crew:**	Gerardo & Ryan		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 289.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			TOPSOIL: (6 INCHES)												
			ARTIFICIAL FILL (af) **Silty SAND (SM)**: medium-grained, non-plastic, strong brown (7.5YR 5/6), moist subrounded gravel, brown (7.5YR 4/2)												
285			**Clayey SAND (SC)**: medium-grained, low plasticity, brown (7.5YR 4/2), moist	S1			24"								
	5		The boring was terminated at approximately 5 ft. below ground surface. The boring was backfilled with auger cuttings on May 18, 2019.												
280	10														
275	15														
270	20														
265	25														
260	30														
255															

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

DRAFT

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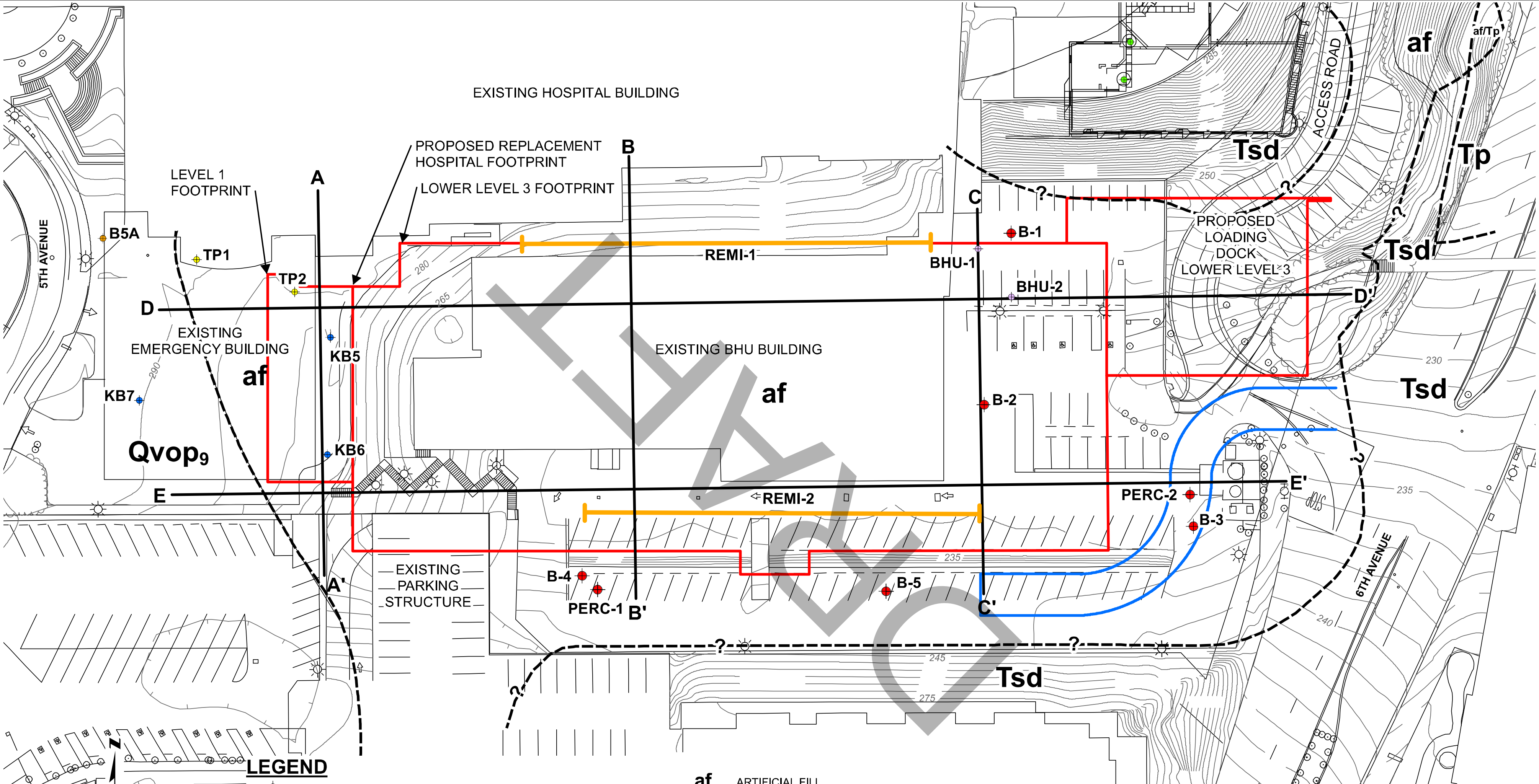
PROJECT NO.: 20194096	BORING LOG PERC-2	FIGURE
DRAWN BY: ST		
CHECKED BY: KC	HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	A-8
DATE: 5/13/2019		
REVISED: -		PAGE: 1 of 1

APPENDIX A.1

Previous Boring Logs

\\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20183768_001_Mercy\mxd\20183768_Site2_Merc.mxd



LEGEND

- APPROXIMATE BORING LOCATION, KLEINFELDER 2018
- APPROXIMATE BORING LOCATION, KLEINFELDER 2006
- APPROXIMATE BORING LOCATION, KLEINFELDER 2005
- APPROXIMATE BORING LOCATION, BENTON 1994
- APPROXIMATE BORING LOCATION, WOODWARD CLYDE 1986
- APPROXIMATE BORING LOCATION, BENTON 1977
- APPROXIMATE LOCATION OF CROSS SECTION, KLEINFELDER 2018
- APPROXIMATE LOCATION OF ReMi LINE
- PROPOSED REPLACEMENT HOSPITAL FOOTPRINT
- PROPOSED FIRE LANE

af	ARTIFICIAL FILL
$Qvop_9$	VERY OLD PARALIC DEPOSITS
Tsd	SAN DIEGO FORMATION
Tp	POMERADO CONGLOMERATE
- - -	GEOLOGIC CONTACT

0 25 50 100 Feet

N

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PROJECT NO.	20183768
DRAWN:	4/22/2019
DRAWN BY:	JP
CHECKED BY:	KC
FILE NAME:	20183768_Site2_Merc.mxd

EXPLORATION MAP

REPLACEMENT HOSPITAL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
2

Date Begin - End:	4/04/2018	Drilling Company:	ABC LIOVIN	**BORING LOG B-4**
Logged By:	S.Tena	**Drill Crew:**	Raul & Juan	
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-85	**Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.	

Approximate Ground Surface Elevation (ft.): 238.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		(3 inches)												
		(3 inches)	S1			42"		10.3		82	34			**R-Value=** 66
235		**ARIFICIAL FILL (af)** **Silty SAND with Gravel (SM)**: fine-grained, non-plastic, light olive gray (5Y 6/2), moist light yellowish brown (2.5Y 6/4)												
	5	**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: fine-grained, non-plastic, light gray (5Y 7/2), moist, very dense, weakly cemented, iron oxide staining	S2		BC=17 36 50/5"	17"		6.9						
230														
	10		S3		BC=7 30 40	18"		9.3	94.8					
225														
	15	light olive gray (5Y 6/2), dense, increase in moisture	S4		BC=10 16 13	18"		21.5						**pH=** 9.1 **Resistivity=** 1800 ohm-cm **Sulfates=** 84 ppm **Chlorides=** 53 ppm
220														
	20	**POMERADO CONGLOMERATE (Tp)** **Silty SAND with Gravel (SM)**: fine to medium-grained, light gray (5Y 7/2) with brownish yellow (10YR 6/8), subrounded gravel (2"), some Clay content	S5		BC=7 50/5"	6"		10.9	122.9					Hard drilling at 20 feet.
215														
	25													
210														

The boring was terminated because of practical auger refusal (↑) at approximately 21 ft. below ground surface. The boring was backfilled with bentonite on April 04, 2018.

GROUNDWATER LEVEL INFORMATION:
Perched groundwater was observed at approximately 21 ft. below ground surface at the end of drilling.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by ALTA SURVEY.

DRAFT

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PROJECT NO.:	20183768	BORING LOG B-4	FIGURE
DRAWN BY:	ST		A-6
CHECKED BY:	SHR	REPLACEMENT HOSPITAL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	
DATE:	4/6/2018		
REVISED:	6/6/2018		PAGE: 1 of 1

DATE DRILLED:	**10/27/2005**	WATER DEPTH:	**None**
DRILLING COMPANY:	**Baja Exploration**	DATE OBSERVED:	**10/27/2005**
DRILLING METHOD:	**CME 75, 140 lb hammer, 30" drop**	GROUND ELEVATION:	**±288 feet MSL**
		LOGGED BY:	**SHR**
HOLE DIAMETER:	**8" Hollow Stem Auger (HSA)**	REVIEWED BY:	**BRB**

ELEVATION (ft.)	DEPTH (feet)	SAMPLES: BULK	SAMPLES: DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	NOTES, LAB, PID READINGS (ppm)
							4 inches of asphalt concrete			
		X			1		**FILL:**			EI=0
285			■	50/5"	2		SILTY SAND (SM), greenish gray, moist to wet, medium dense, fine grained, driving on cobble, poor sample recovery			'R'=65
	5		■	27	3					
280							**SAN DIEGO FORMATION:**			
	10		■	50/4"	4		SILTY SAND (SM), yellow brown with reddish brown mottling, moist, dense, very fine grained, weakly to moderately cemented	87.8	10.9	
275										
	15		■	45	5		Becomes light greenish gray			
270										
	20		■	50/6"	6		becomes very dense, fine grained			
265										
	25		■	50/5"	7					
260							Bottom of boring 26.5 feet No groundwater observed Boring backfilled with bentonite grout			
	30									
255										

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

PROJECT NO. 57557

SCRIPPS MERCY HOSPITAL
EMERGENCY DEPARTMENT
SAN DIEGO, CALIFORNIA

LOG OF BORING KB6

FIGURE
A4

DATE DRILLED:	10/27/2005	WATER DEPTH:	None
DRILLING COMPANY:	Baja Exploration	DATE OBSERVED:	10/27/2005
DRILLING METHOD:	CME 75, 140 lb hammer, 30" drop	GROUND ELEVATION:	±290 feet MSL
		LOGGED BY:	SHR
HOLE DIAMETER:	8" Hollow Stem Auger (HSA)	REVIEWED BY:	BRB

ELEVATION (ft.)	DEPTH (feet)	SAMPLES BULK	SAMPLES DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	NOTES, LAB, PID READINGS (ppm)
							4 inches of asphalt concrete pavement over 4 inches gravel base			
							LINDAVISTA FORMATION:			
			■	50/4"	1		SILTY SAND with trace GRAVEL (SM), light reddish brown, dry, very dense, coarse grained, very angular gravel 0 to 10%			M&D, SA (12%)
285	5		■	50/4"	2		1.5" total sample recovery, probable slough and not representative of in place material	112.8	7.3	
							Gravel zone 6 to 8 feet			
							SAN DIEGO FORMATION:			
280	10		■	85	3		SILTY SAND (SM), light greenish gray, dry to moist, very dense, fine grained, slightly micaceous, massive, weakly to moderately cemented	97.2	10.5	M&D
275	15		■	35	4		Medium greenish gray silt bed at 15 to 16 feet Light greenish gray, dry to moist, dense to very dense, slightly micaceous, massive			
270	20		■	50/5"	5		Becomes moist, very dense, slightly micaceous, massive, vertical fracture in sample			
							Bottom of boring 20.9 feet No groundwater observed Boring backfilled with bentonite			
265	25									
260	30									

KLEINFELDER
5015 SHOREHAM PLACE
SAN DIEGO, CALIFORNIA 92122

PROJECT NO. 57557

SCRIPPS MERCY HOSPITAL
EMERGENCY DEPARTMENT
SAN DIEGO, CALIFORNIA

LOG OF BORING KB7

FIGURE **A5**

APPENDIX B

Laboratory Test Results

APPENDIX B
LABORATORY TEST RESULTS

GENERAL

Laboratory tests were performed on selected, representative samples as an aid in classifying the soils and to evaluate physical properties of the soils, which may affect foundation design and construction procedures. A description of our laboratory testing program is presented below:

CLASSIFICATION

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory excavations in Appendix A.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Natural moisture content and dry density tests were performed on the intact samples collected. Moisture content was evaluated in general accordance with ASTM Test Method D 2216; dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. The values are shown on the boring logs in Appendix A.

SIEVE ANALYSIS

Sieve analyses were performed on six samples from the site to evaluate the gradation characteristics of the soil and to aid in its classification. The tests were performed in general accordance with ASTM Test Method D 422. The test results are presented on Figures B-1 through B-6.

ATTEBERG LIMITS TEST

Liquid and plastic limits tests were conducted on one selected sample. The test was performed in general accordance with ASTM D 4318. In addition, the values are shown on the boring logs in Appendix A and Figure B-7.

DIRECT SHEAR TEST

Three direct shear tests were performed on three representative soil samples to evaluate the shear strength of the site soils. The soil samples were tested in a saturated state and subjected to three different normal pressures in general accordance with ASTM Test Method D3080. The test results are presented on Figures B-8 through B-10.

EXPANSION INDEX TEST

One expansion index (EI) test was performed on a representative sample of the near-surface materials obtained during our investigation. The test was performed in accordance with ASTM D4829. The corrected expansion index value for the sample is presented in Table B-1. The test result indicates a very low expansion potential when compared to Table B-2 to qualitatively evaluate the expansion potential of the site soils.

Table B-1

Expansion Index Test Results

Boring	Depth (ft)	Soil Type	EI
B-1	1-5	Clayey Sand	0

Table B-2

ASTM D 4829 Expansion Index and Potential

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

CORROSION TEST

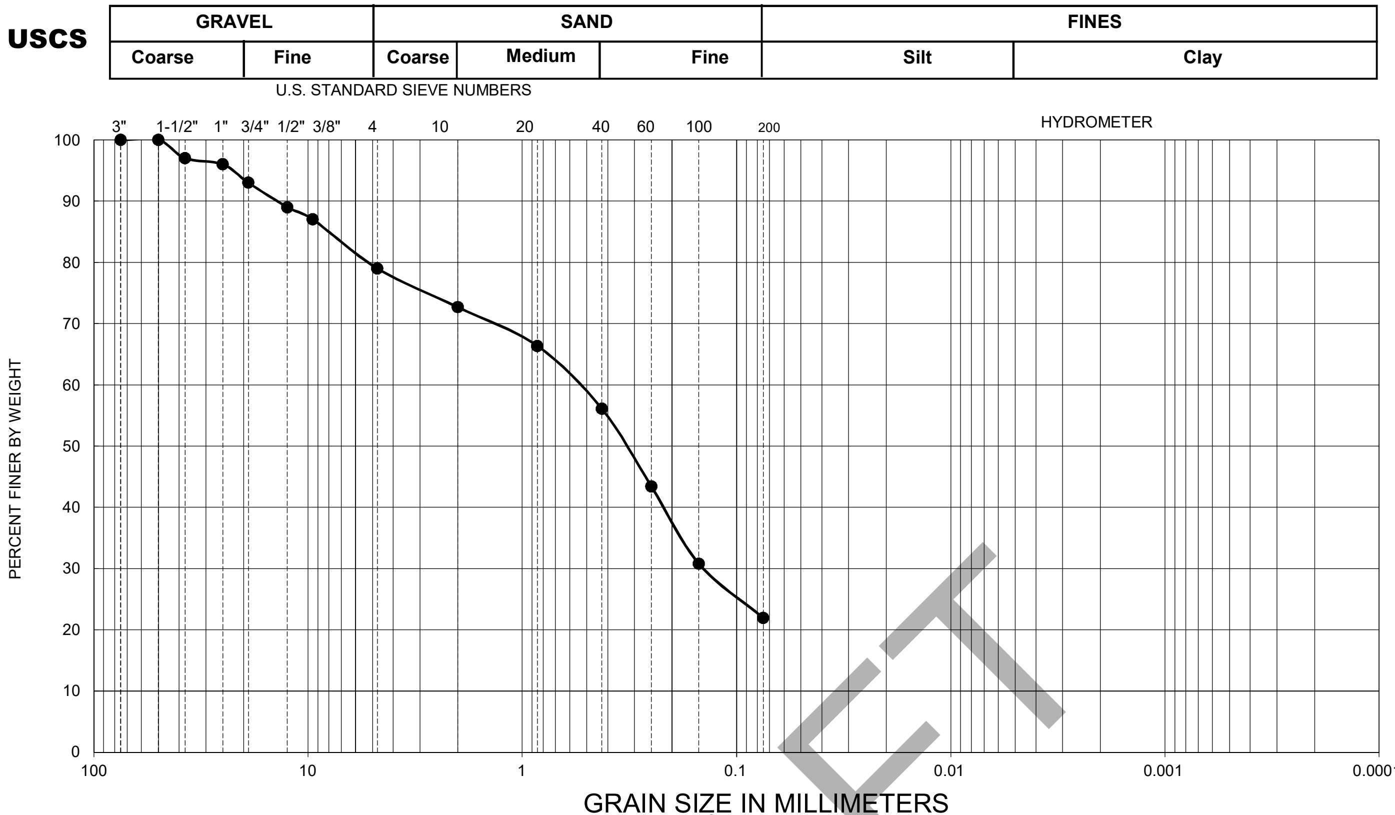
The sulfate and chloride contents and pH of two selected samples were evaluated in general accordance with California Test 417 and California Test 422. Our boring logs and these test results should be reviewed by a qualified corrosion engineer to evaluate the general soil stratigraphy corrosion potential with respect to construction materials and determine whether further testing is warranted. The test results are presented on Table B-3 and Figure B-11.

Table B-3

Corrosion Test Results

Sample	Depth (ft)	pH	Minimum Resistivity (ohm-cm)	Sulfate (%)	Chloride (%)
B-3/S1	0-5	8.7	9100	0.017	0.005

Date Tested: **5/30/2019**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B1	S1	1-5	22.0	SC

Sample Description	Clayey Sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	97
	1"	25 mm	96
	3/4"	19 mm	93
	1/2"	12.5 mm	89
	3/8"	9.5 mm	87
	No. 4	4.75 mm	79
	No. 10	2.0 mm	73
	No. 20	0.85 mm	66
	No. 40	0.425 mm	56
	No. 60	0.25 mm	43
	No 100	0.15 mm	31
	No 200	.075 mm	22.0

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	S.Tena	Tech:	Uly P.
Project No.	20194096.001A	Date:	25-Jun-19

GRADATION TEST RESULTS

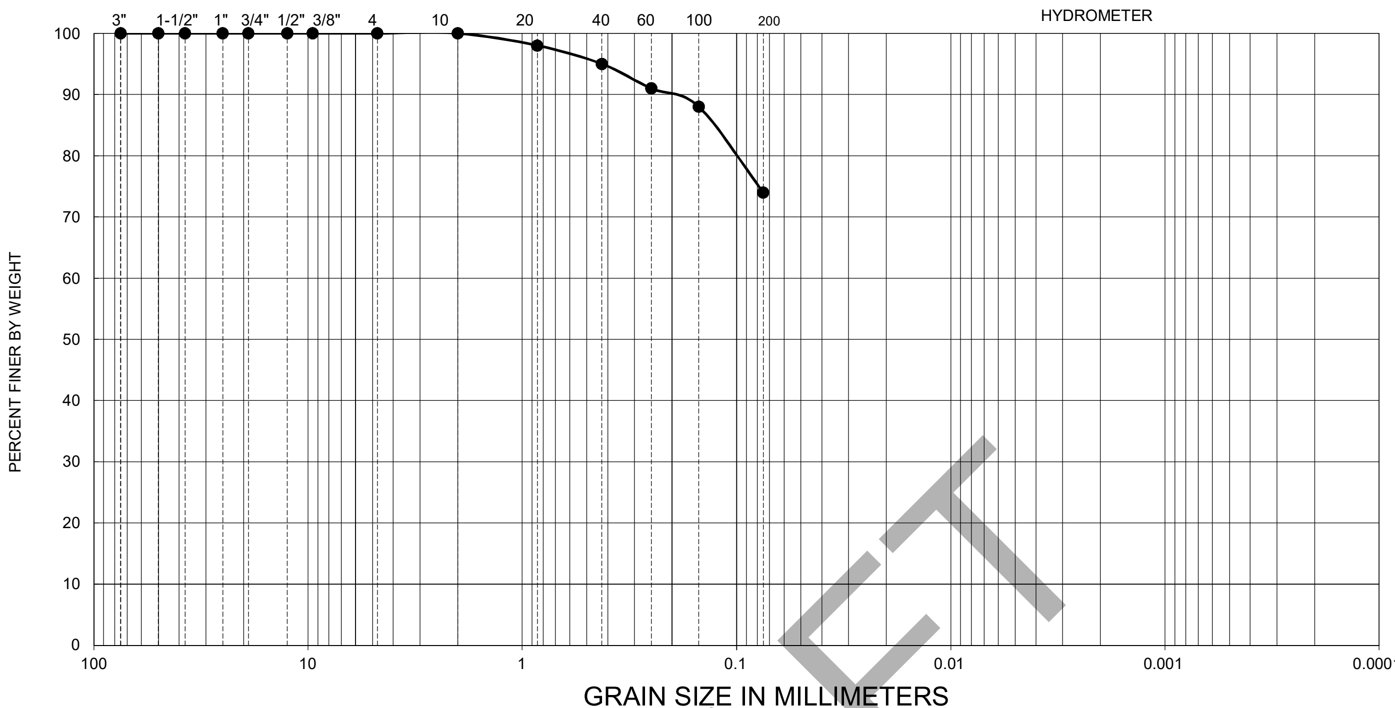
HOSPITAL REPLACEMENT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-1

Date Tested: **6/5/2019**

USCS	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B1	S4	21-21.5	74.0	ML

Sample Description	Silt with sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	98
	No. 40	0.425 mm	95
	No. 60	0.25 mm	91
	No 100	0.15 mm	88
	No 200	.075 mm	74.0

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	S.Tena	Tech:	Uly P.
Project No.	20194096.001A	Date:	25-Jun-19

GRADATION TEST RESULTS

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

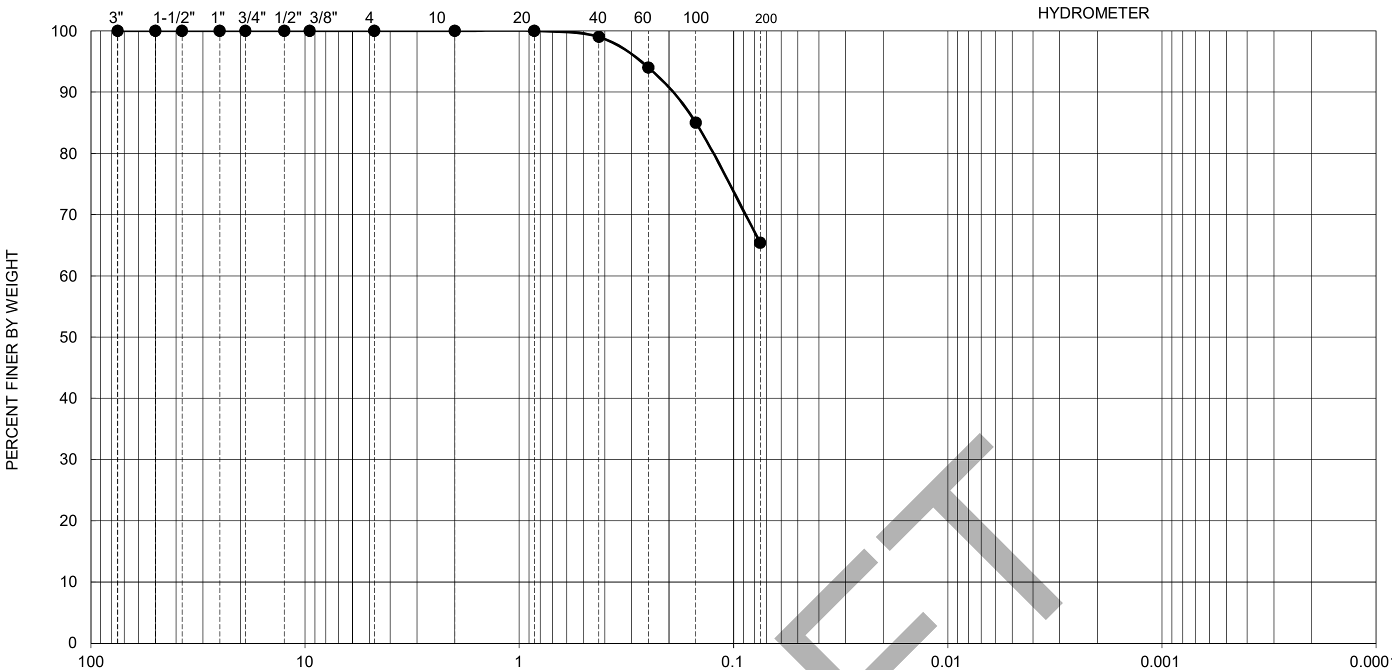
FIGURE

B-2

Date Tested: **5/31/2019**

USCS	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

U.S. STANDARD SIEVE NUMBERS



GRAIN SIZE IN MILLIMETERS

Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B1	S6	31-31.5	65.4	ML

Sample Description	Sandy silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	99
	No. 60	0.25 mm	94
	No 100	0.15 mm	85
	No 200	.075 mm	65.4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	S.Tena	Tech:	Uly P.
Project No.	20194096.001A	Date:	25-Jun-19

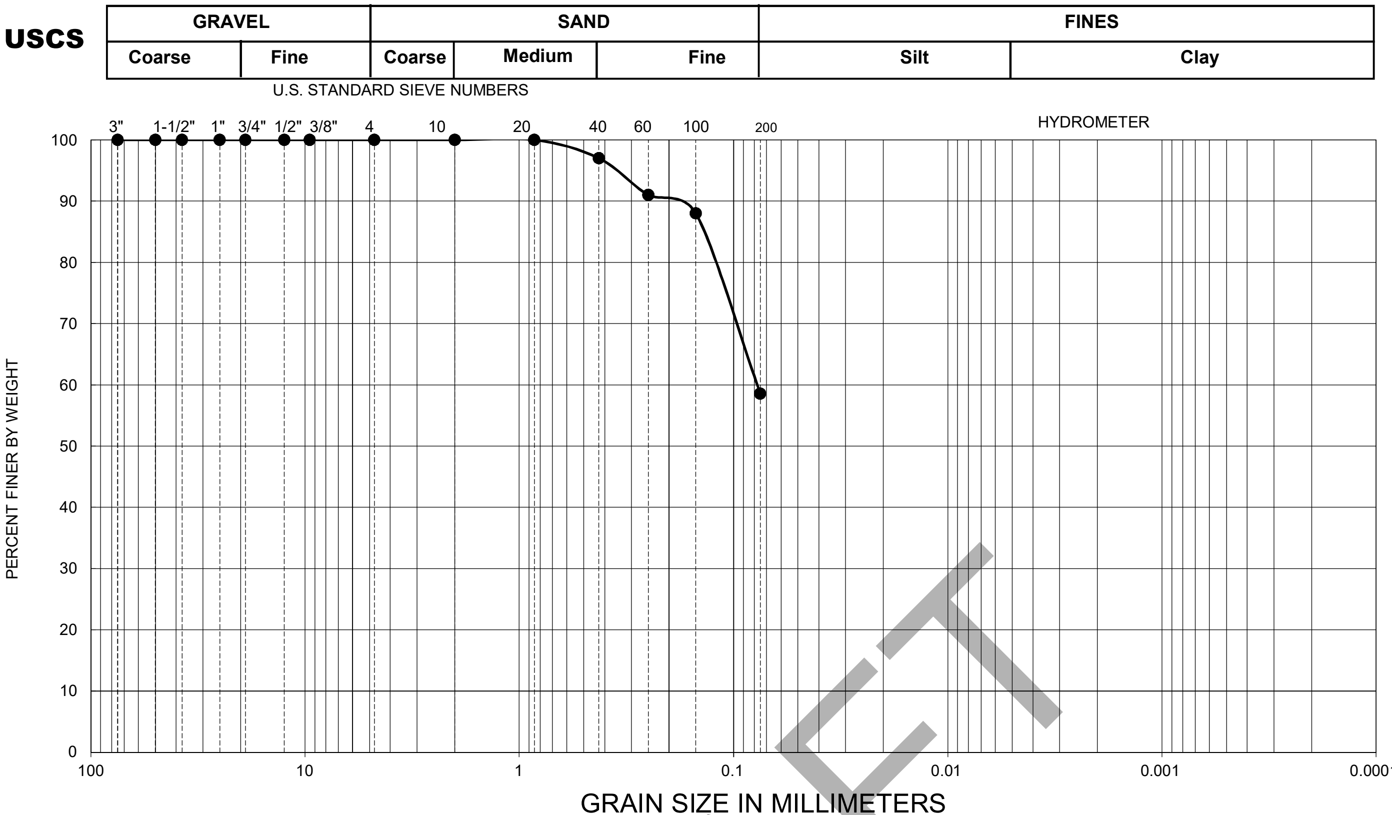
GRADATION TEST RESULTS

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-3

Date Tested: **6/5/2019**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B1	S8	41-41.5	58.6	ML

Sample Description	Sandy silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	97
	No. 60	0.25 mm	91
	No 100	0.15 mm	88
	No 200	.075 mm	58.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
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Checked by:	S.Tena	Tech:	Uly P.
Project No.	20194096.001A	Date:	25-Jun-19

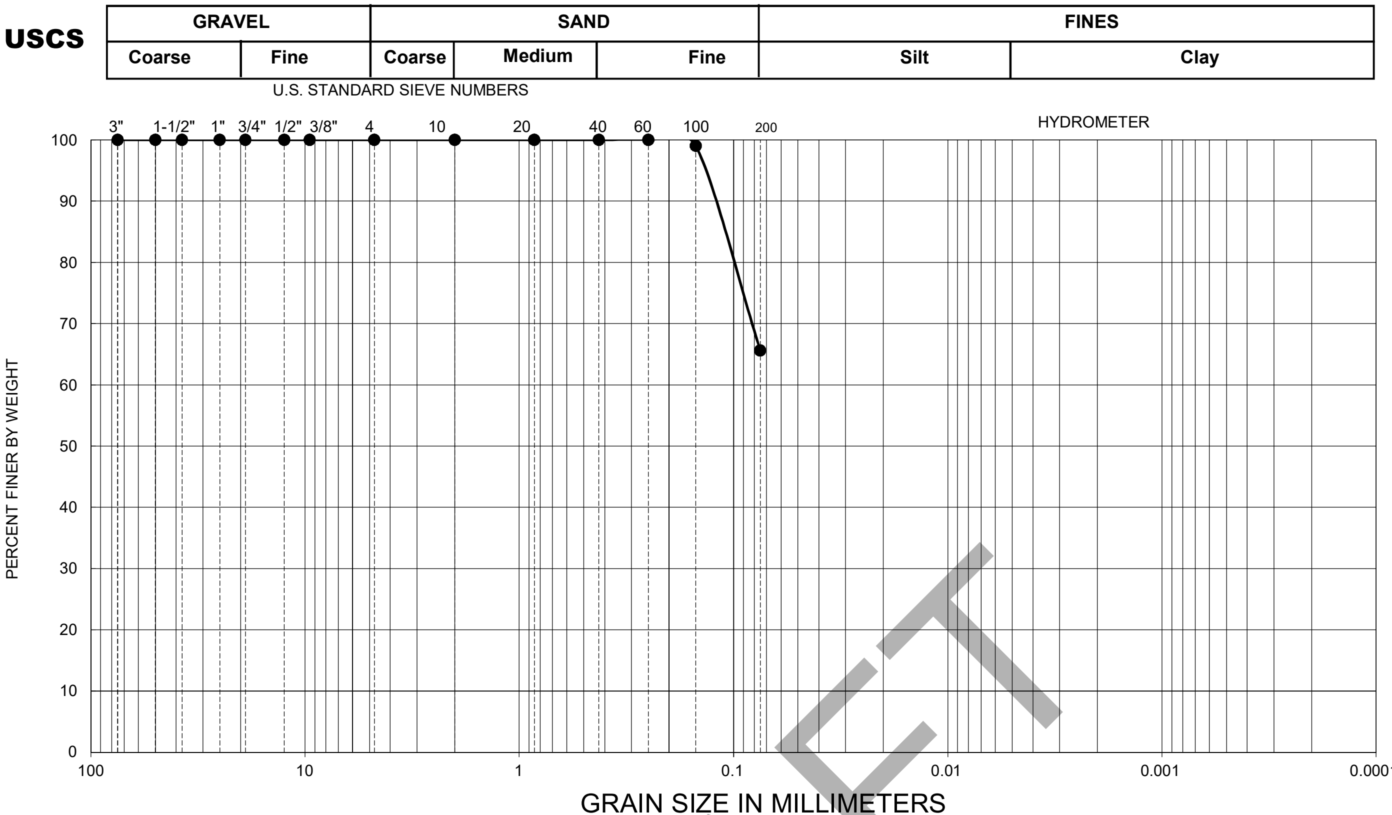
GRADATION TEST RESULTS

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-4

Date Tested: **6/5/2019**



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B1	S13	71-71.5	65.6	ML

Sample Description	Sandy silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	100
	No. 60	0.25 mm	100
	No 100	0.15 mm	99
	No 200	.075 mm	65.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

KLEINFELDER
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Checked by:	S.Tena	Tech:	Uly P.
Project No.	20194096.001A	Date:	25-Jun-19

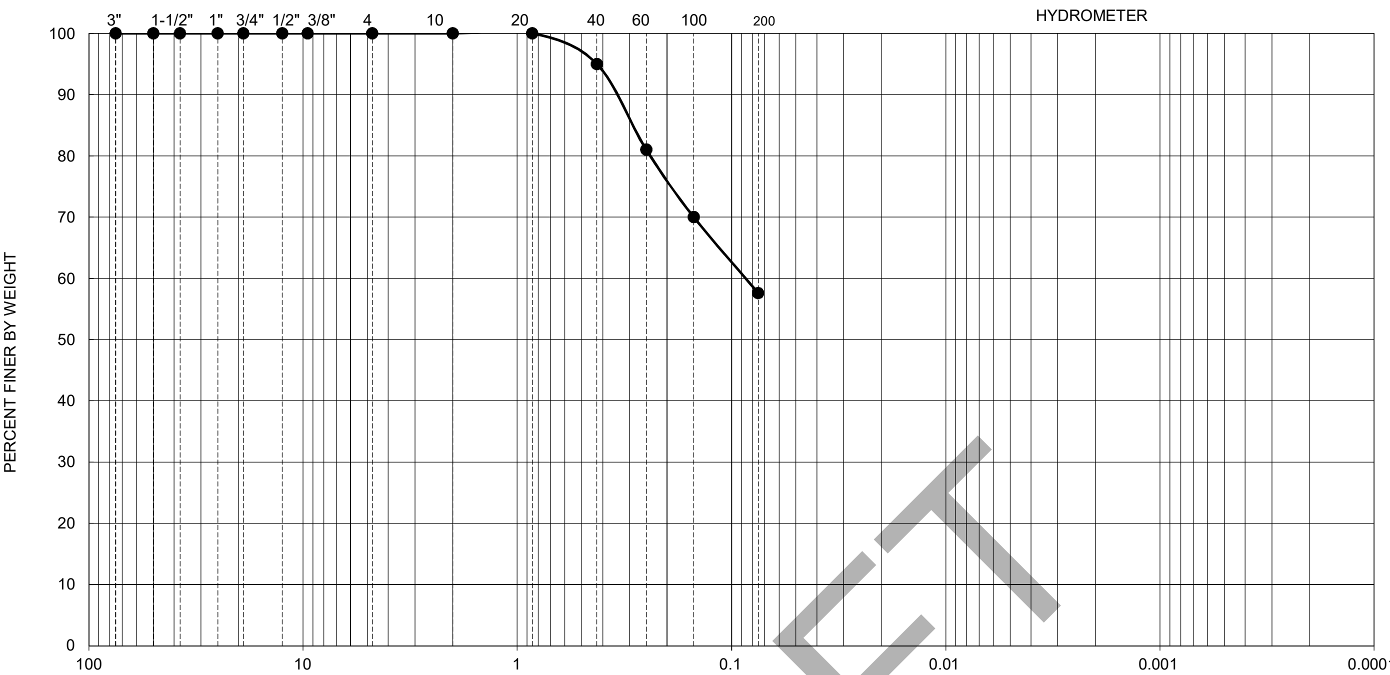
GRADATION TEST RESULTS

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
B-5

Date Tested: **6/5/2019**

USCS	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B4	S5	25-26.5	57.6	ML

Sample Description	Sandy silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	100
	No. 40	0.425 mm	95
	No. 60	0.25 mm	81
	No 100	0.15 mm	70
	No 200	.075 mm	57.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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GRADATION TEST RESULTS

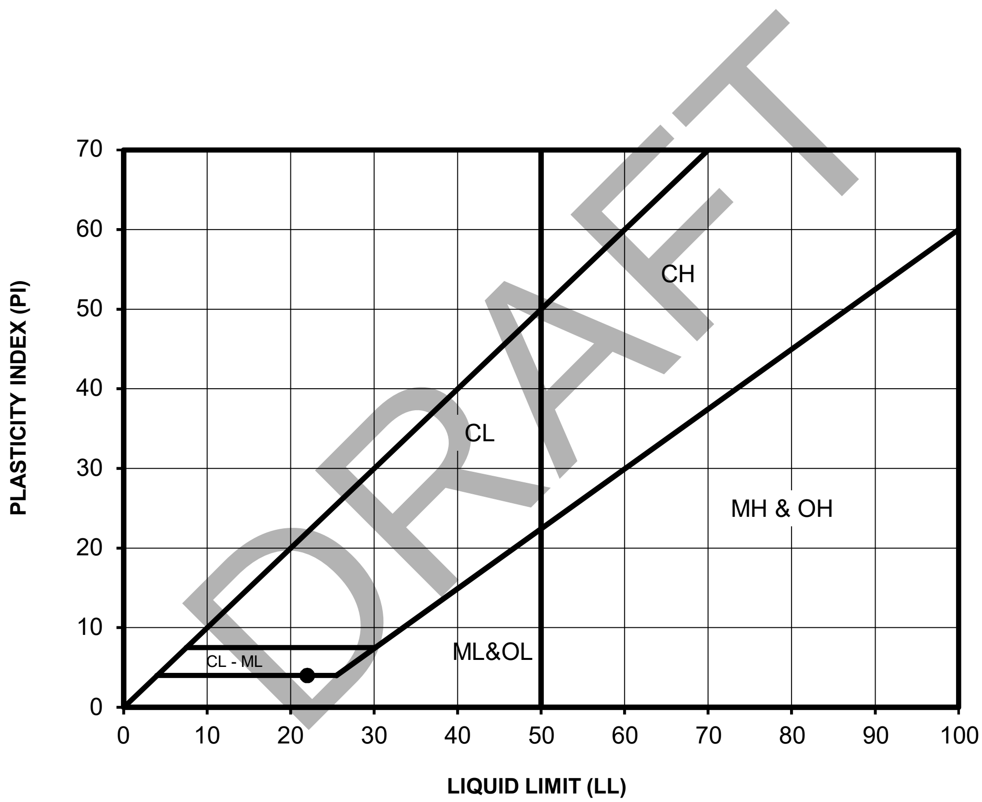
HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE **B-6**

Checked by:	S.Tena	Tech:	Uly P.
Project No.	20194096.001A	Date:	25-Jun-19

Date Tested : 6/5/2019

SYMBOL	SAMPLE NAME	DEPTH (ft)	LL	PL	PI	USCS CLASSIFICATION (Minus No. 40 Sieve Fraction)	USCS (Entire Sample)
●	B4-S2	6-6.5	22	18	4	CL-ML	SC-SM



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Limitations: Pursuant to applicable codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specification were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided. This report may not be reproduced, except in full, without written approval of Kleinfelder.

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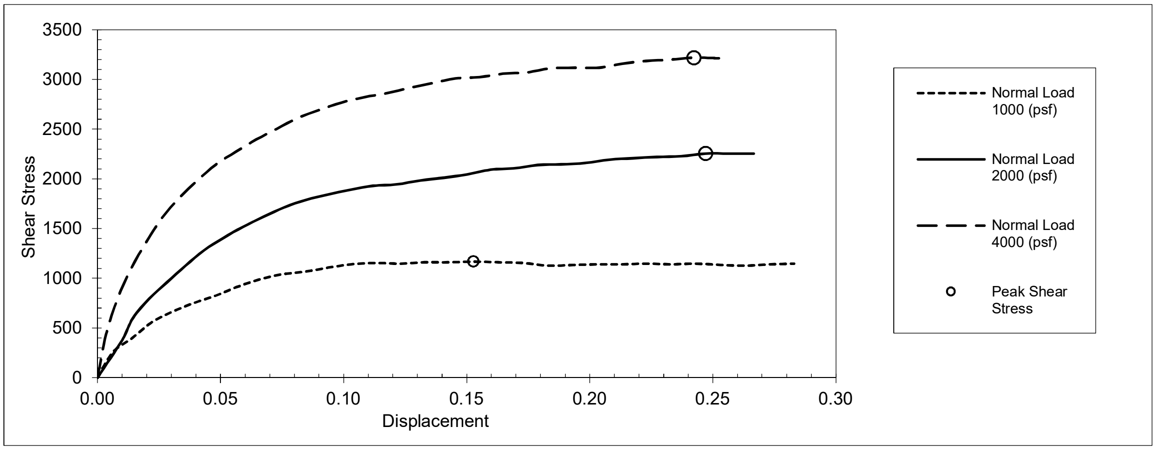
Checked by S.Tena	TECH Uly P.		
PROJECT NO: 20194096.001A	25-Jun-19		

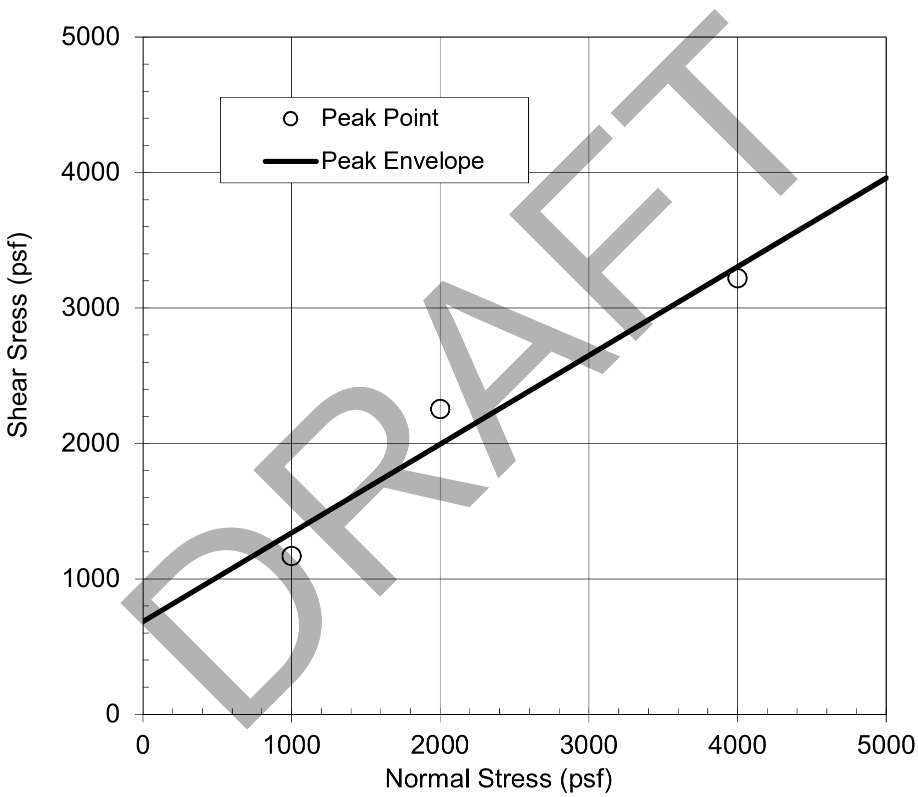
ATTERBERG LIMITS TEST RESULTS

HOSPITAL SUPPORT BUILDING
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-7



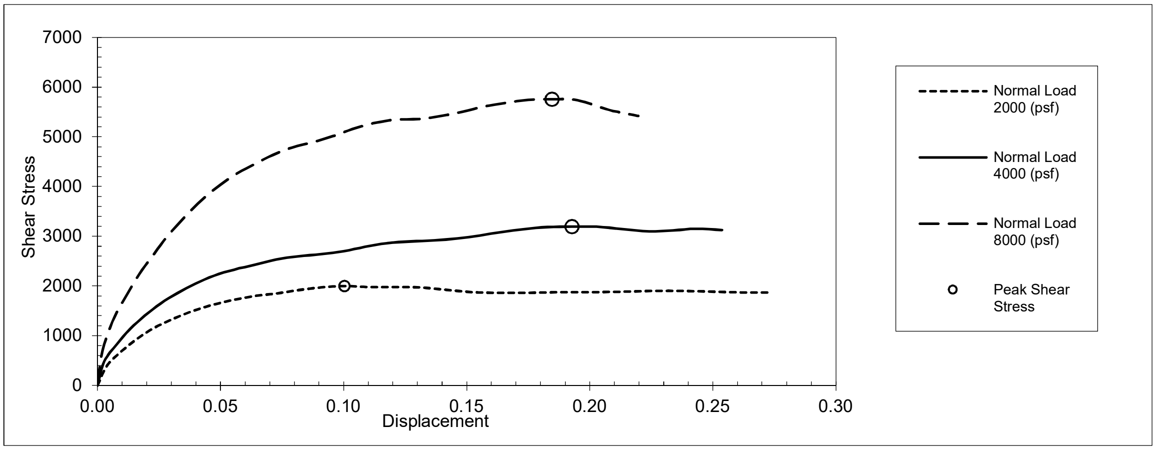


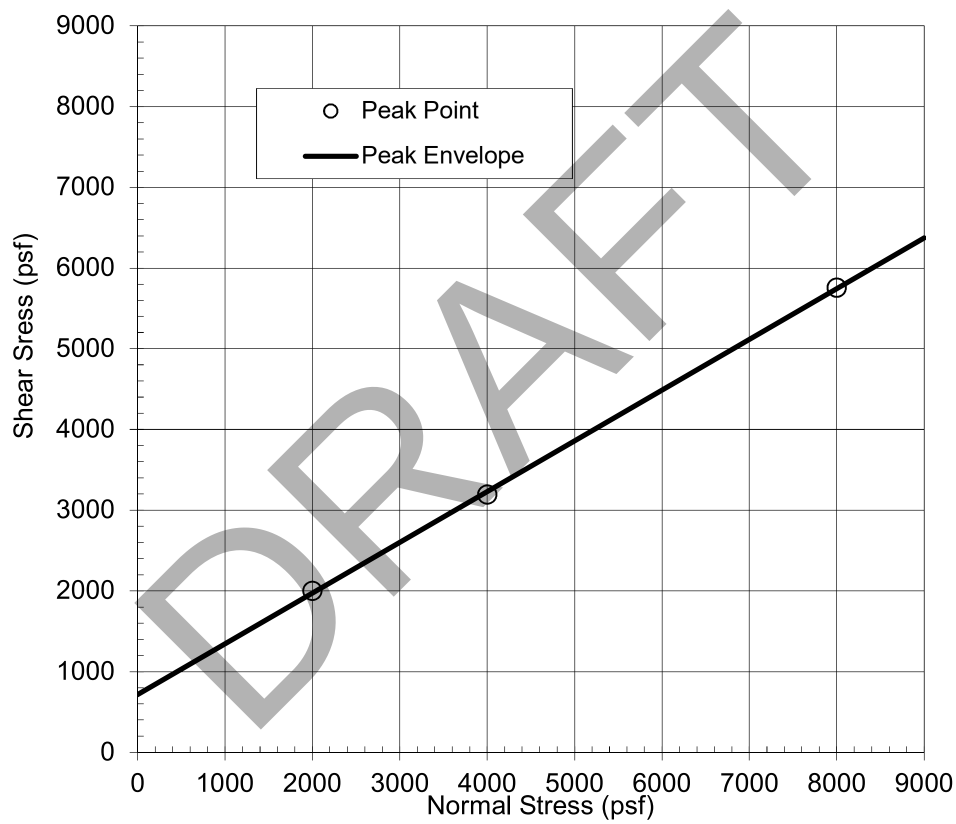
Strain Rate = 0.0118 inch/min
Date Tested: 5/29/2019

				Interpreted Shear Strength			
				Peak			
Boring No.	Sample No.	Depth	UCSC	Cohesion (psf)	Friction Angle (deg)		
B1	S6	31'-31.5'	ML	685	33.2		

Sample description: Sandy silt

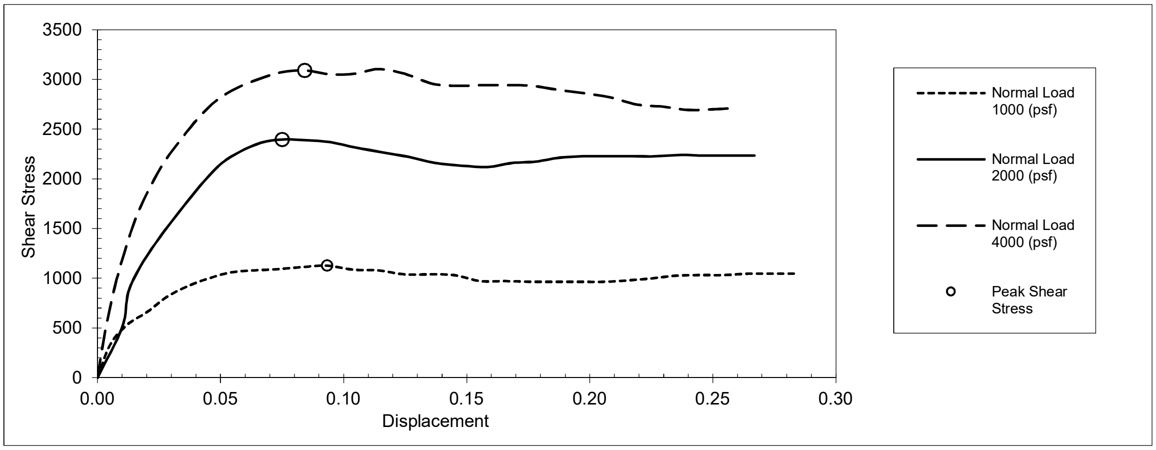
KLEINFELDER Bright People. Right Solutions.	**Direct Shear Test Results (ASTM D 3080)**	Figure
Checked By: / Tech : Uly	**HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA**	**B-8**
Project # 20194096.001A / 25-Jun-19		

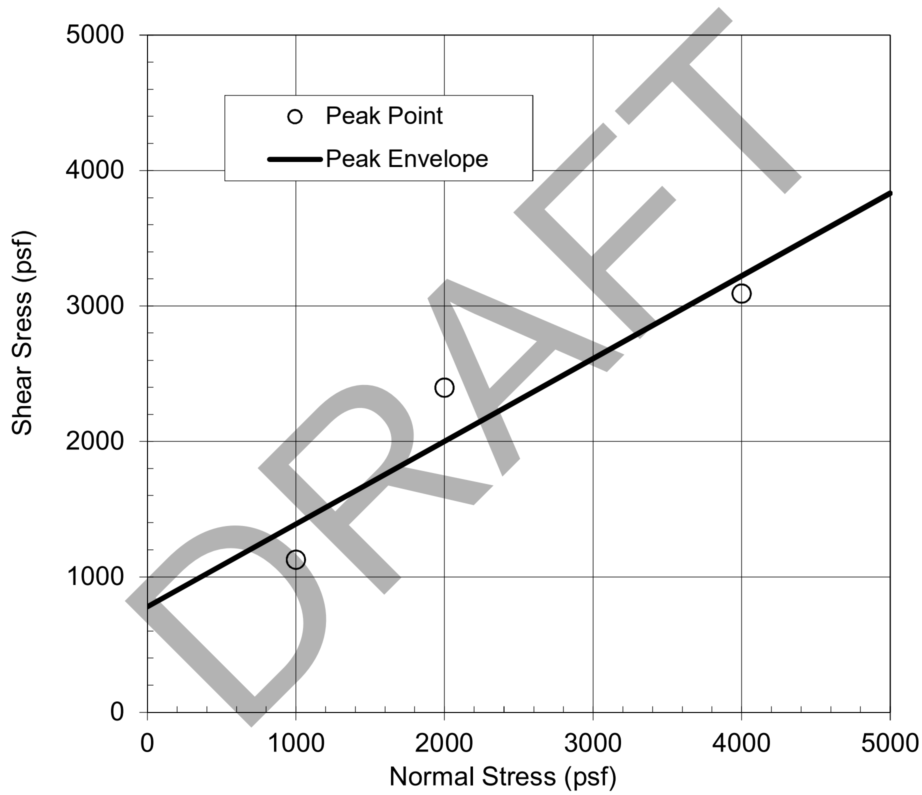




Strain Rate =	0.0118	inch/min		Interpreted Shear Strength			
Date Tested:	6/3/2019			Peak			
Boring No.	Sample No.	Depth	UCSC	Cohesion (psf)	Friction Angle (deg)		
B1	S13	71'-71.5'	ML	715	32.2		
Sample description:		sandy silt					

KLEINFELDER Bright People. Right Solutions.		**Direct Shear Test Results (ASTM D 3080)**	Figure
Checked By:	Tech : Uly	**HOSPITAL SUPPORT BUILIDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA**	**B-9**
Project # 20194096.001A	25-Jun-19		





Strain Rate = 0.0118 inch/min

Date Tested: 5/31/2019

Boring No.	Sample No.	Depth	UCSC	Interpreted Shear Strength: Peak Cohesion (psf)	Interpreted Shear Strength: Peak Friction Angle (deg)		
B4	S5	26'-26.5'	ML	779	31.4		

Sample description: sandy silt

KLEINFELDER Bright People. Right Solutions.		**Direct Shear Test Results (ASTM D 3080)**	Figure
Checked By:	Tech : Uly	**HOSPITAL SUPPORT BUILDING SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA**	**B-10**
Project # 20194096.001A	25-Jun-19		

L A B O R A T O R Y   R E P O R T

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Date: May 30, 2019
Purchase Order Number: PROJECT#20194096.001A
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Laboratory Number: SO7353   Customers Phone: 619-831-4600
Fax: 619-831-4619

Sample Designation:
--
One soil sample received on 05/29/19 at 9:00am,
marked as follows:
Project: Scripps Mercy Replacement Hospital
Project #: 20194096.001A
Boring #: B1
Sample #: S1
Depth: 0'-5'
Date Sampled 05/11/2019

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.7

Water Added (ml)	Resistivity (ohm-cm)
10	7200
5	3600
5	3100
5	2000
5	1500
5	950
5	910
5	960
5	980

29 years to perforation for a 16 gauge metal culvert.
38 years to perforation for a 14 gauge metal culvert.
53 years to perforation for a 12 gauge metal culvert.
68 years to perforation for a 10 gauge metal culvert.
82 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.017% (170 ppm)
Water Soluble Chloride	Calif. Test 422	0.005% (53 ppm)

Rosa Bernal
RMB/dbb

APPENDIX C

Infiltration Study

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
		Preliminary Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ◉ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ○ Yes; continue to Step 1F. ○ No; conduct appropriate number of tests.	
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ○ Yes; continue to Step 1G. ○ No; select appropriate factor of safety.	
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ○ Yes; answer "Yes" to Criteria 1 Result. ○ No; answer "No" to Criteria 1 Result.	
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ○ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ◉ No; full infiltration is not required. Skip to Part 1 Result.	
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.01 inches per hour.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	○ Yes	○ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	○ Yes	○ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Summarize findings and basis; provide references to related reports or exhibits.

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	○ Full infiltration Condition ◉ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:

Criteria 3 : Infiltration Rate Screening	
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⊙ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ⊙ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ⊙ No: Skip to Part 2 Result.
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.01 inches per hour.	

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	◯ Yes	◯ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◯ Yes	◯ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◯ Yes	◯ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◯ Yes	◯ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◯ Yes	◯ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	◯ Yes	◯ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.	

Part 2 – Partial Infiltration Geotechnical Screening Result[5]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	○ Partial Infiltration Condition ◉ No Infiltration Condition

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]	Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:
	Preliminary Phase
Criteria 1: Infiltration Rate Screening	

1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ⦿ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ◯ Yes; continue to Step 1F. ◯ No; conduct appropriate number of tests.	
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ◯ Yes; continue to Step 1G. ◯ No; select appropriate factor of safety.	
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ◯ Yes; answer "Yes" to Criteria 1 Result. ◯ No; answer "No" to Criteria 1 Result.	
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ◯ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ◉ No; full infiltration is not required. Skip to Part 1 Result.	
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.03 inches per hour.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	Yes	No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	Yes	No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	Yes	No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	Yes	No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	Yes	No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	○ Yes	○ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Summarize findings and basis; provide references to related reports or exhibits.

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	○ Full infiltration Condition ⊙ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:
Criteria 3 : Infiltration Rate Screening	

3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⦿ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ⦿ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ⦿ No: Skip to Part 2 Result.

Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).

Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.03 inches per hour.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	○ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○Yes	○No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○Yes	○No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.	

Part 2 – Partial Infiltration Geotechnical Screening Result[5]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	○ Partial Infiltration Condition ⦿ No Infiltration Condition

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

Percolation Test Data Sheet

Project:	Mercy Replacement Hospital	Tested By:	Sean Hanrahan
		Date:	5/19/2019
Project No:	20194096.001A	Checked By:	S.Tena

Borehole ID:	**PERC-1**	
Depth of Borehole:	5	feet
Diameter of Borehole:	8	inches
USCS Soil Classification:	SC	
PVC Pipe Hieght above Surface	0.2	ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	6:43	7:13	30	0.79	0.84	0.05	50.00
2	7:13	7:43	30	0.79	0.80	0.01	250.00
3	7:43	8:13	30	0.79	0.81	0.02	125.00
4	8:13	8:43	30	0.79	0.81	0.02	125.00
5	8:43	9:13	30	0.79	0.81	0.02	125.00
6	9:13	9:43	30	0.79	0.80	0.01	250.00
7	9:43	10:13	30	0.79	0.81	0.02	125.00
8	10:13	10:43	30	0.79	0.81	0.02	125.00
9	10:43	11:13	30	0.79	0.81	0.02	125.00
10	11:13	11:43	30	0.79	0.81	0.02	125.00
11	11:43	12:13	30	0.79	0.82	0.03	83.33
12	12:13	12:43	30	0.79	0.81	0.02	**125.00**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H \ 60 \ r}{\Delta t \ (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	50.50	inches
Hf	50.26	inches
ΔH	0.24	inches
Havg	50.38	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.02**	in/hr

Percolation Test Data Sheet

Project: Mercy Replacement Hospital

Tested By: Sean Hanrahan
Date: 5/19/2019

Project No: 20194096.001A

Checked By: S.Tena

Borehole ID: **PERC-2**
Depth of Borehole: 5 feet
Diameter of Borehole: 8 inches
USCS Soil Classification: SC
PVC Pipe Hieght above Surface 0.7 ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	6:47	7:17	30	0.33	0.42	0.09	27.78
2	7:17	7:47	30	0.33	0.42	0.09	27.78
3	7:47	8:17	30	0.33	0.39	0.06	41.67
4	8:17	8:47	30	0.33	0.39	0.06	41.67
5	8:47	9:17	30	0.33	0.39	0.06	41.67
6	9:17	9:47	30	0.33	0.39	0.06	41.67
7	9:47	10:17	30	0.33	0.38	0.05	50.00
8	10:17	10:47	30	0.33	0.39	0.06	41.67
9	10:47	11:17	30	0.33	0.39	0.06	41.67
10	11:17	11:47	30	0.33	0.39	0.06	41.67
11	11:47	12:17	30	0.33	0.38	0.05	50.00
12	12:17	12:47	30	0.33	0.39	0.06	**41.67**

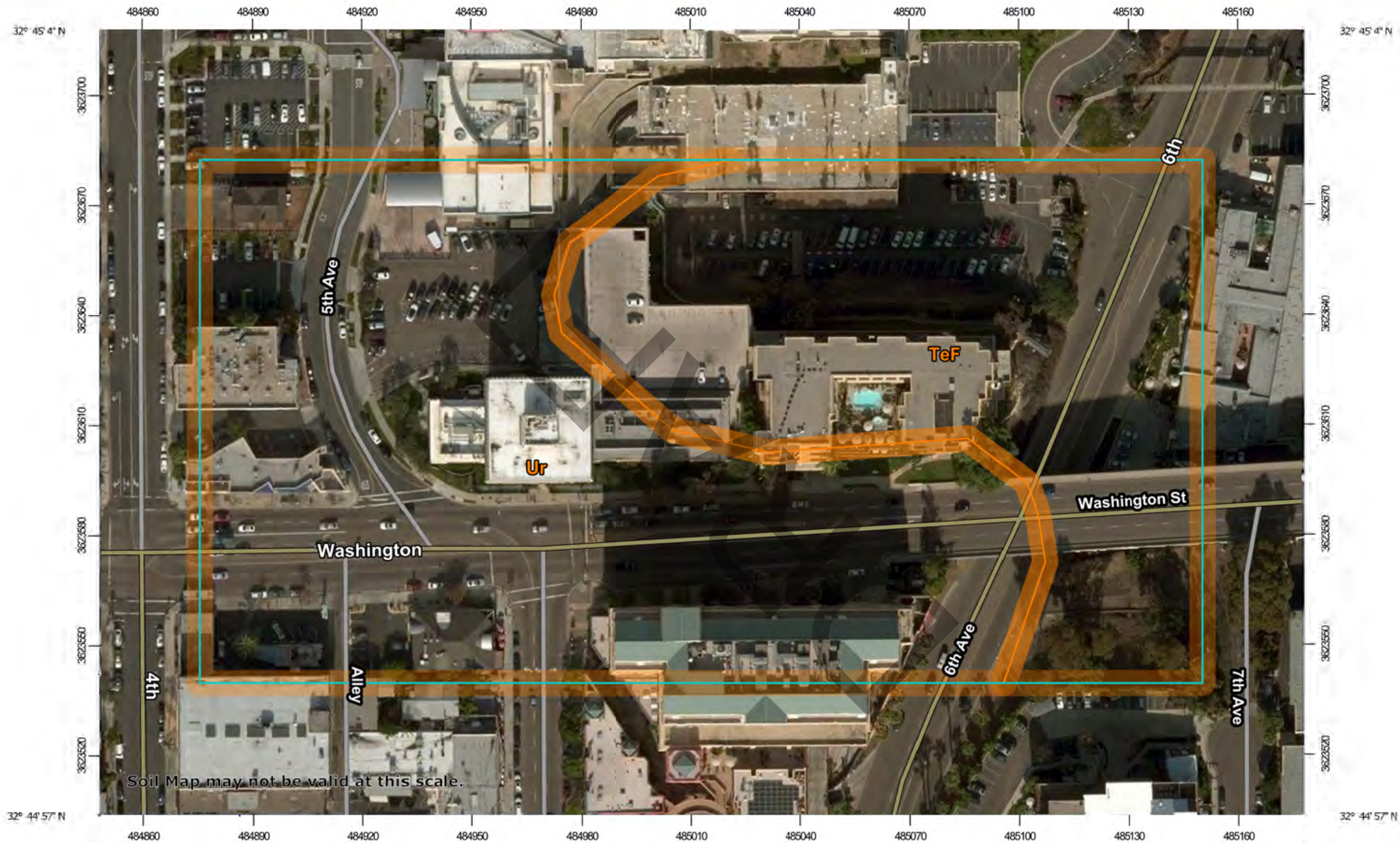
Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

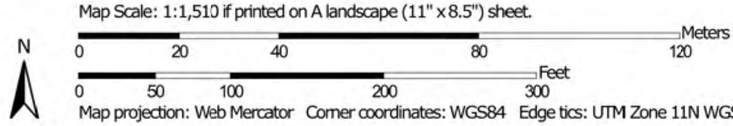
$$I_t = \frac{\Delta H \; 60\, r}{\Delta t \; (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	56.00	inches
Hf	55.28	inches
ΔH	0.72	inches
Havg	55.64	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.05**	in/hr



Map Scale: 1:1,510 if printed on A landscape (11" x 8.5") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 13, Sep 12, 2018

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 7, 2014—Jan 4, 2015

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
TeF	Terrace escarpments	3.9	40.4%
Ur	Urban land	5.8	59.6%
Totals for Area of Interest		**9.7**	**100.0%**

Hydrologic Soil Group and Surface Runoff

This table gives estimates of various soil water features. The estimates are used in land use planning that involves engineering considerations.

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The four hydrologic soil groups are:

Group 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.

Group 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.

Group 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.

Group 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.

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.

Surface runoff refers to the loss of water from an area by flow over the land surface. Surface runoff classes are based on slope, climate, and vegetative cover. The concept indicates relative runoff for very specific conditions. It is assumed that the surface of the soil is bare and that the retention of surface water resulting from irregularities in the ground surface is minimal. The classes are negligible, very low, low, medium, high, and very high.

Report—Hydrologic Soil Group and Surface Runoff

Absence of an entry indicates that the data were not estimated. The dash indicates no documented presence.

Hydrologic Soil Group and Surface Runoff–San Diego County Area, California			
Map symbol and soil name	**Pct. of map unit**	**Surface Runoff**	**Hydrologic Soil Group**
TeF—Terrace escarpments			
Terrace escarpments	100	Very high	—

Hydrologic Soil Group and Surface Runoff–San Diego County Area, California			
Map symbol and soil name	**Pct. of map unit**	**Surface Runoff**	**Hydrologic Soil Group**
Ur—Urban land			
Urban land	100	Very high	—

Data Source Information

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 13, Sep 12, 2018

Engineering Properties

This table gives the engineering classifications and the range of engineering properties for the layers of each soil in the survey area.

Hydrologic soil group is a group of soils having similar runoff potential under similar storm and cover conditions. The criteria for determining Hydrologic soil group is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Listing HSGs by soil map unit component and not by soil series is a new concept for the engineers. Past engineering references contained lists of HSGs by soil series. Soil series are continually being defined and redefined, and the list of soil series names changes so frequently as to make the task of maintaining a single national list virtually impossible. Therefore, the criteria is now used to calculate the HSG using the component soil properties and no such national series lists will be maintained. All such references are obsolete and their use should be discontinued. Soil properties that influence runoff potential are those that influence the minimum rate of infiltration for a bare soil after prolonged wetting and when not frozen. These properties are depth to a seasonal high water table, saturated hydraulic conductivity after prolonged wetting, and depth to a layer with a very slow water transmission rate. Changes in soil properties caused by land management or climate changes also cause the hydrologic soil group to change. The influence of ground cover is treated independently. There are four hydrologic soil groups, A, B, C, and D, and three dual groups, A/D, B/D, and C/D. In the dual groups, the first letter is for drained areas and the second letter is for undrained areas.

The four hydrologic soil groups are described in the following paragraphs:

Group 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.

Group 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.

Group 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.

Group 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.

Depth to the upper and lower boundaries of each layer is indicated.

Texture is given in the standard terms used by the U.S. Department of Agriculture. These terms are defined according to percentages of sand, silt, and clay in the fraction of the soil that is less than 2 millimeters in diameter. "Loam," for example, is soil that is 7 to 27 percent clay, 28 to 50 percent silt, and less than 52 percent sand. If the content of particles coarser than sand is 15 percent or more, an appropriate modifier is added, for example, "gravelly."

Classification of the soils is determined according to the Unified soil classification system (ASTM, 2005) and the system adopted by the American Association of State Highway and Transportation Officials (AASHTO, 2004).

The Unified system classifies soils according to properties that affect their use as construction material. Soils are classified according to particle-size distribution of the fraction less than 3 inches in diameter and according to plasticity index, liquid limit, and organic matter content. Sandy and gravelly soils are identified as GW, GP, GM, GC, SW, SP, SM, and SC; silty and clayey soils as ML, CL, OL, MH, CH, and OH; and highly organic soils as PT. Soils exhibiting engineering properties of two groups can have a dual classification, for example, CL-ML.

The AASHTO system classifies soils according to those properties that affect roadway construction and maintenance. In this system, the fraction of a mineral soil that is less than 3 inches in diameter is classified in one of seven groups from A-1 through A-7 on the basis of particle-size distribution, liquid limit, and plasticity index. Soils in group A-1 are coarse grained and low in content of fines (silt and clay). At the other extreme, soils in group A-7 are fine grained. Highly organic soils are classified in group A-8 on the basis of visual inspection.

If laboratory data are available, the A-1, A-2, and A-7 groups are further classified as A-1-a, A-1-b, A-2-4, A-2-5, A-2-6, A-2-7, A-7-5, or A-7-6. As an additional refinement, the suitability of a soil as subgrade material can be indicated by a group index number. Group index numbers range from 0 for the best subgrade material to 20 or higher for the poorest.

Percentage of rock fragments larger than 10 inches in diameter and 3 to 10 inches in diameter are indicated as a percentage of the total soil on a dry-weight basis. The percentages are estimates determined mainly by converting volume percentage in the field to weight percentage. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Percentage (of soil particles) passing designated sieves is the percentage of the soil fraction less than 3 inches in diameter based on an ovendry weight. The sieves, numbers 4, 10, 40, and 200 (USA Standard Series), have openings of 4.76, 2.00, 0.420, and 0.074 millimeters, respectively. Estimates are based on laboratory tests of soils sampled in the survey area and in nearby areas and on estimates made in the field. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Liquid limit and *plasticity index* (Atterberg limits) indicate the plasticity characteristics of a soil. The estimates are based on test data from the survey area or from nearby areas and on field examination. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

References:

American Association of State Highway and Transportation Officials (AASHTO). 2004. Standard specifications for transportation materials and methods of sampling and testing. 24th edition.

American Society for Testing and Materials (ASTM). 2005. Standard classification of soils for engineering purposes. ASTM Standard D2487-00.

Report—Engineering Properties

Absence of an entry indicates that the data were not estimated. The asterisk '*' denotes the representative texture; other possible textures follow the dash. The criteria for determining the hydrologic soil group for individual soil components is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Engineering Properties–San Diego County Area, California														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
TeF—Terrace escarpments														
Terrace escarpments	100		0-60	Variable	—	—	—	—	—	—	—	—	—	—
Ur—Urban land														
Urban land	100		0-6	Variable	—	—	—	—	—	—	—	—	—	—

Data Source Information

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 13, Sep 12, 2018

APPENDIX D

Suggested Guidelines for Earthwork Construction

APPENDIX D
SUGGESTED GUIDELINES FOR EARTHWORK CONSTRUCTION

1.0 GENERAL

1.1 **Scope** - The work done under these specifications shall include clearing, stripping, removal of unsuitable material, excavation, preparation of natural soils, placement and compaction of onsite and imported fill material and placement and compaction of pavement materials.

1.2 **Contractor's Responsibility** - The Contractor shall attentively examine the site in such a manner that he can correlate existing surface conditions with those presented in the geotechnical study report. He shall satisfy himself that the quality and quantity of exposed materials and subsurface soil or rock deposits have been satisfactorily represented by the Geotechnical Engineer's report and project drawings. Any discrepancy of prior knowledge to the Contractor to that is revealed through his study shall be made known to the Owner. It is the Contractor's responsibility to review the report prior to construction. The selection of equipment for use on the project and the order of the work shall similarly be the Contractor's responsibility. The Contractor shall be responsible for providing equipment capable of completing the requirements included in the following sections.

1.3 **Geotechnical Engineer** - The work covered by these specifications shall be observed and tested by Kleinfelder, the Geotechnical Engineer, who shall be hired by the Owner. The Geotechnical Engineer will be present during the site preparation and grading to observe the work and to perform the tests necessary to evaluate material quality and compaction. The Geotechnical Engineer shall submit a report to the Owner, including a tabulation of tests performed. The costs of re-testing unsuitable work installed by the Contractors shall be deducted by the Owner from the payments to the Contractor.

1.4 **Standard Specifications** - Where referred to in these specifications, "Standard Specifications" shall mean the State of California Standard Specifications for Public Works Construction, with Regional Supplement Amendments for San Diego County, 2000 Edition.

1.5 **Compaction Test Method** - Where referred to herein, relative compaction shall mean the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D 1557 Compaction Test Procedure. Optimum moisture content shall mean the moisture content at the maximum dry density determined above.

2.0 SITE PREPARATION

2.1 **Clearing** - Areas to be graded shall be cleared and grubbed of all vegetation and debris. The Contractor shall remove these materials from the site.

2.2 **Stripping** - Surface soils containing roots and organic matter shall be stripped from areas to be graded and stockpiled or discarded as directed by the Owner. In general, the depth of stripping of the topsoil will be approximately 3 inches. Deeper stripping, where required to remove weak soils or accumulations of organic matter, shall be performed when determined necessary by the Geotechnical Engineer. Stripped material shall be removed from the site or stockpiled at a location designated by the Owner.

2.3 **Removal of Existing Fill** - Existing fill soils, trash and debris in the areas to be graded shall be removed prior to the placing of any compacted fill. Portions of any existing fills that are suitable for use in new compacted fill may be stockpiled for future use. All organic materials, topsoil, expansive soils, oversized rock or other unsuitable material shall be removed from the site by the Contractor or disposed of at a location onsite, if so designated by the Owner.

2.4 **Ground Surface** - The ground surface exposed by stripping shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction and compacted as required for compacted fill. Ground surface preparation shall be approved by the Geotechnical Engineer prior to placing fill.

3.0 EXCAVATION

3.1 **General** - Excavations shall be made to the lines and grades indicated on the plans. The data presented in the Geotechnical Engineer's report is for information only and the Contractor shall make his own interpretation with regard to the methods and equipment necessary to perform the excavation and to obtain material suitable for fill.

3.2 **Materials** - Soils which are removed and are unsuitable for fill shall be placed in nonstructural areas of the project, or in deeper fills at locations designated by the Geotechnical Engineer.

All oversize rocks and boulders that cannot be incorporated in the work by placing in embankments or used as rip-rap or for other purposes shall be removed from the site by the Contractor.

3.3 **Treatment of Exposed Surface** - The ground surface exposed by excavation shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction and compacted as required for compacted fill. Compaction shall be approved by the Geotechnical Engineer prior to placing fill.

3.4 **Rock Excavation** - Where solid rock is encountered in areas to be excavated, it shall be loosened and broken up so that no solid ribs, projections or large fragments will be within 6 inches of the surface of the final subgrade.

4.0 COMPACTED FILL

4.1 **Materials** - Fill material shall consist of suitable onsite or imported soil. All materials used for structural fill shall be reasonably free of organic material, have an Expansion Index of 30 or less, 100% passing the 3 inch sieve and less than 30 percent passing the #200 sieve.

4.2 **Placement** - All fill materials shall be placed in layers of 8 inches or less in loose thickness and uniformly moisture conditioned. Each lift should then be compacted with a sheepsfoot roller or other approved compaction equipment to at least 90 percent relative compaction in areas under structures, utilities, roadways and parking areas. No fill material shall be placed, spread or rolled while it is frozen or thawing, or during unfavorable weather conditions.

4.3 **Compaction Equipment** - The Contractor shall provide and use sufficient equipment of a type and weight suitable for the conditions encountered in the field. The equipment shall be capable of obtaining the required compaction in all areas.

4.4 **Recompaction** - When, in the judgment of the Geotechnical Engineer, sufficient compactive effort has not been used, or where the field density tests indicate that the required compaction or moisture content has not been obtained, or if pumping or other indications of instability are noted, the fill shall be reworked and recompacted as needed to obtain a stable fill at the required density and moisture content before additional fill is placed.

4.5 **Responsibility** - The Contractor shall be responsible for the maintenance and protection of all embankments and fills made during the contract period and shall bear the expense of replacing any portion which has become displaced due to carelessness, negligent work or failure to take proper precautions.

5.0 UTILITY TRENCH BEDDING AND BACKFILL

5.1 **Material** - Pipe bedding shall be defined as all material within 4 inches of the perimeter and 12 inches over the top of the pipe. Material for use as bedding shall be clean sand, gravel, crushed aggregate or native free-draining material, having a Sand Equivalent of not less than 30.

Backfill should be classified as all material within the remainder of the trench. Backfill shall meet the requirements set forth in Section 4.0 for compacted fill.

5.2 **Placement and Compaction** - Pipe bedding shall be placed in layers not exceeding 8 inches in loose thickness, conditioned to the proper moisture content for compaction and compacted to at least 90 percent relative compaction. All other trench backfill shall be placed and compacted in accordance with Section 306-1.3.2 of the Standard Specifications for Mechanically Compacted Backfill. Backfill shall be compacted as required for adjacent fill. If not specified, backfill shall be compacted to at least 90 percent relative compaction in areas under structures, utilities, roadways, parking areas and concrete flatwork.

6.0 SUBSURFACE DRAINAGE

6.1 **General** - Subsurface drainage shall be constructed as shown on the plans. Drainage pipe shall meet the requirements set forth in the Standard Specifications.

6.2 **Materials** - Permeable drain rock used for subdrainage shall meet the following gradation requirements:

Sieve Size	Percentage Passing
3"	100
1-1/2"	90 - 100
3/4"	50 - 80
No. 4	24 - 40
No. 100	0-4
No. 200	0 - 2

6.3 **Geotextile Fabric** - Filter fabric shall be placed between the permeable drain rock and native soils. Filter cloth shall have an equivalent opening size greater than the No. 100 sieve and a grab strength not less than 100 pounds. Samples of filter fabric shall be submitted to the Geotechnical Engineer for approval before the material is brought to the site.

6.3 **Placement and Compaction** - Drain rock shall be placed in layers not exceeding 8 inches in loose thickness and compacted as required for adjacent fill, but in no case, to be less than 85 percent relative compaction. Placement of geotextile fabric shall be in accordance with the manufacturer's specifications and shall be checked by the Geotechnical Engineer.

7.0 AGGREGATE BASE BENEATH CONCRETE SLABS

7.1 **Materials** - Aggregate base beneath concrete slabs shall consist of clean free-draining sand, gravel or crushed rock conforming to the following gradation requirements:

Sieve Size	Percent Passing
1"	100
3/8"	30 – 100
No. 20	0 – 10

7.2 **Placement** - Aggregate base shall be compacted and kept moist until placement of concrete. Compaction shall be by suitable vibrating compactors. Aggregate base shall be placed in layers not exceeding 8 inches in loose thickness. Each layer shall be compacted by at least four passes of the compaction equipment or until 95 percent relative compaction has been obtained.

APPENDIX E

Geotechnical Business Council Insert

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can

construction project.

Geotechnical-Engineering Services Are Performed for

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GBA
GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org   www.geoprofessional.org

June 26, 2019
Kleinfelder Project No. 20194095.001A

Mr. Bruce A. Rainey
Corporate Vice President
Scripps Health
10140 Campus Point Drive, Suite 210
San Diego, California 92121

SUBJECT: Geologic and Geotechnical Engineering Report
MOB West Core and Shell
Scripps Mercy Hospital
4077 5th Avenue
San Diego, California

Dear Mr. Rainey:

This report presents the results and recommendations of our geologic and geotechnical engineering investigation for the design and construction of the proposed medical office building (MOB). The proposed project is to be constructed on the northwestern portion of the existing Scripps Mercy Hospital campus located on 4077 5th Avenue in San Diego, California.

Based on the results of our geotechnical investigation and engineering evaluation, it is our opinion that the project is feasible from a geotechnical standpoint. Our geotechnical investigation and evaluation are based on the provided RFP documents, discussions with Mr. Dylan Williams of Scripps Health and our proposed scope of work.

We appreciate this opportunity to be of service and look forward to working with you in the future. If you have any questions about this report, please contact us.

Respectfully submitted,

KLEINFELDER

Salvador Tena
Staff Engineer, PE 89071

Scott H. Rugg CEG 1651
Engineering Geologist

Kevin M. Crennan GE 2511
Senior Engineer

GEOLOGIC AND GEOTECHNICAL ENGINEERING REPORT
MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA
KLEINFELDER PROJECT NO. 20194095.001A

JUNE 26, 2019

A Report Prepared for:

Mr. Bruce A. Rainey
Corporate Vice President
Scripps Health
10140 Campus Point Drive, Suite 210
San Diego, California 92121

GEOLOGIC AND GEOTECHNICAL ENGINEERING REPORT
MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

Prepared by:

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June 26, 2019
Kleinfelder Project No: 20194095.001A

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EXECUTIVE SUMMARY

The Scripps Mercy Hospital San Diego is located at 4077 5th Avenue in San Diego, California (Figure 1). The proposed medical office building (MOB) is located in the northwestern portion of the hospital campus, east of 4th Avenue and south of Lewis Street. The approximate footprint of the proposed MOB building site is shown in Figure 2.

The proposed structure will be 5-stories in height with one additional subterranean level. Temporary shoring is anticipated along most or all of the building perimeter to protect existing structures, streets and improvements.

Kleinfelder's field exploration for the MOB building consisted of advancing five hollow stem auger borings, infiltration testing and a geologic reconnaissance of the site. Boring data from previous nearby studies on the Scripps Mercy campus were also evaluated in this study. The Logs of Borings for this study are included in Appendix A and the locations are shown in Figure 2.

The results of our field exploration and review indicate that fill up to approximately 5 feet in depth is present within the building area. The very old paralic deposits underly the fill and overlay the San Diego Formation. These formational units are typically very dense and cemented. The very old paralic deposits also contain gravel and cobble and were very difficult to penetrate with 8-inch diameter hollow stem augers. Based on previous boring logs in the area, the Pomerado Conglomerate is present approximately 60 feet below the surface, well below the depths of excavation. No static groundwater was observed in the borings. A more detailed description of each unit is provided in the attached report.

Based on our review of the data collected during the study, it is our opinion that the proposed development is feasible from a geotechnical perspective if the recommendations contained herein are followed. The following key items are conclusions developed from our study:

- Undocumented fill to depths of up to approximately 5 feet were encountered within our borings in the proposed building area. Deeper fill may be associated with subsurface utilities or buried structures such as underground storage tanks.

- Based on the results of our geotechnical evaluation, it is our opinion that the undocumented fill is not suitable for support of structural loads in their current condition. However, it is anticipated that all of this material will be removed for the building excavation which will be on the order of 15 feet in depth.
- The lower level of the building can be supported on spread footings or a mat foundation system bearing entirely on the San Diego Formation.
- Temporary shoring will be required around most or all of the building perimeter to protect existing improvements during construction of the basement level.
- The replacement hospital building site is located in a seismically active area and could be subject to relatively strong ground shaking due to earthquakes on active faults in the region. The structures should be designed to tolerate seismic shaking in accordance with the 2019 California Building Code (CBC).
- The site is not located within a State of California Alquist-Priolo fault zone.

This summary of findings should not be relied upon without consulting the attached report for a more detailed description of the geotechnical study performed by Kleinfelder for Scripps Mercy Hospital San Diego. This report is subject to the limitations included in Section 7.0 and the Geotechnical Business Council insert in Appendix E.

1 INTRODUCTION

1.1 PURPOSE

In accordance with your authorization, we have completed a geologic and geotechnical investigation for the proposed medical office building (MOB) building, Scripps Mercy Hospital project located at 4077 5th Avenue, San Diego, California (see Vicinity Map, Figure 1). The purpose of our study was to evaluate the surface and subsurface conditions at the site, evaluate potential geologic hazards, and provide geotechnical recommendations for design and construction. This report presents the results of our background review, subsurface exploration, laboratory testing, geotechnical analyses, conclusions regarding the geotechnical conditions at the project site, and our recommendations.

1.2 PROJECT DESCRIPTION

We understand the project is in the early stages of design and preliminary building plans provided by CO Architects were provided for review. The proposed MOB building is anticipated to be 5-stories above grade with one additional subterranean level with a floor elevation on the order of 15 feet below existing grade. The building footprint will be up to approximately 24,000 square feet in size; however, the footprint of the building is not defined at this time. Additional improvements will likely include hardscape, landscaping, lighting, underground utilities and stormwater BMPs. Pavement is not anticipated due to the structure occupying the majority of the site

The address and latitude/longitude coordinates for the MOB building are listed below, and the vicinity map is shown on Figure 1. The site and exploration plan is shown on Figure 2.

Address: Scripps Mercy Hospital San Diego
4077 5th Avenue
San Diego, California 92103

Latitude: 32.7515^{O} N

Longitude: 117.1612^{O} W

1.3 GENERAL SITE DESCRIPTION

The Mercy Hospital campus is generally located north of Washington Street, south of Montecito Way, west of 6th Avenue and east of 4th Avenue, as shown on the Vicinity Map, Figure 1. The majority of the developed campus is relatively level with surface elevations on the order of +290 to +295 feet above mean sea level (MSL). The proposed MOB building is located on the northwestern portion of campus east of 4th Avenue, south of Lewis Street and west of the existing hospital, with surface elevations varying from approximately +288 to +298 feet MSL. This area contains landscaped areas with grass, numerous large trees with concrete retaining walls up to about 5 feet in height, concrete walking paths, a concrete fountain feature with benched concrete seating, and a circular paved dropoff area by the existing hospital entrance. An underground water tank is reportedly located in the northeastern landscaped area just south of the dropoff driveway.

1.4 SCOPE OF SERVICES

The scope of services for this study included the following;

- Review of readily available pertinent reports (including previous borings performed at the site), geologic maps and aerial photographs;
- Drilling and sampling five soil borings;
- Performing geotechnical laboratory testing of the soil samples;
- Providing a site plan with boring locations;
- Discussion of site surface and subsurface conditions encountered, groundwater levels and anticipated excavation characteristics of the materials;
- Discussion of regional geologic setting, geologic features, and potential geologic hazard including the potential for ground rupture due to surface faulting, liquefaction and seismically induced settlement;
- Providing 2019 California Building Code seismic parameters;
- Recommendations for temporary shoring;
- Lateral earth pressures for permanent retaining walls;

- Recommendations for foundation design, including foundation type, soil bearing pressures, and anticipated total and differential settlements;
- Performing a preliminary screening of soil corrosivity characteristics;
- Guidelines for earthwork construction including recommendations for site preparation, shoring, fill placement and compaction.
- Recommendations for slab-on-grade floor design and construction;
- Preliminary stormwater infiltration study; and
- Preparation of this report.

2 METHODS OF STUDY

2.1 FIELD EXPLORATION

Our current field exploration consisted of a geologic reconnaissance of the site, drilling three borings to evaluate subsurface conditions, drilling two borings for infiltration testing, and collection of soil samples for laboratory testing. The exploration borings were drilled on May 7, 2019 to depths ranging from approximately 20 to 31 ½ feet and the two borings for infiltration testing were drilled to a depth of 5 feet. Due to access constraints at the site, the drilling contractor, Pacific Drilling of San Diego, used a limited access Fraste PL-G drill rig. The rig was equipped with 6-inch diameter hollow stem augers, and a 140-pound hammer dropped from a height of 30 inches to drive the sampler into the soil.

A Kleinfelder engineer from our office logged the subsurface soil conditions at each boring location and collected soil samples for laboratory testing. Driven samples were obtained from the borings using Standard Penetration Test and California split spoon samplers lined with three 6-inch brass sleeves. Soil samples obtained from the borings were packaged, sealed in the field to reduce moisture loss and disturbance, and returned to our laboratory for testing. Upon completion, all borings were backfilled with soil cuttings or bentonite chips. A more detailed description of the Kleinfelder boring exploration program, the logs of the exploratory borings are presented in Appendix A.

2.2 GEOLOGIC RECONNAISSANCE AND MAPPING

The site was geologically evaluated by our engineering geologist. Part of this work included reviewing historic maps and aerial photography to evaluate the extent of previous earthwork at the site for infilled drainages. Data from our geologic review is incorporated into this report.

2.3 LABORATORY TESTING

Geotechnical laboratory testing was performed on representative bulk and driven samples to substantiate field classifications and to provide engineering parameters for geotechnical design. Testing performed consisted of moisture/density measurement, sieve analyses, direct shear, Atterberg limits, pH, resistivity, soluble sulfates, and chlorides. Laboratory testing procedures and test results are provided in Appendix B.

2.4 BACKGROUND REVIEW

We have reviewed readily available unpublished geotechnical reports relevant to the subject site. The most pertinent reports to the current project include:

- "Geologic and Geotechnical Engineering Evaluation, Scripps Mercy Hospital Expansion, New Emergency Department, 4077 5th Avenue, San Diego California," prepared by Kleinfelder, dated January 20, 2006.
- "Supplemental Geotechnical Investigation, Proposed Emergency Department Expansion, Scripps Mercy Hospital, 4077 5th Avenue, San Diego California," prepared by Kleinfelder, dated March 6, 2007.

We have also reviewed historic aerial photography and maps of the site as part of our work.

3 GEOLOGY AND SOILS

3.1 REGIONAL GEOLOGIC AND GEOTECTONIC SETTING

The site is located within western coastal platform of the Peninsular Range geomorphic province (Norris and Webb, 1990). This province stretches for several hundreds of miles south from the Los Angeles area to the tip of Baja California. It is dominated by basement material of Cretaceous-age igneous rocks of the Southern California Batholith and various Jurassic-age metamorphic rocks that are often situated as isolated blocks within the igneous rocks. This igneous/metamorphic complex occupies the regions of central and eastern San Diego County.

The western coastal zone of San Diego County is underlain by a westward thickening wedge of sedimentary units that were deposited on the basement rocks described above. These sedimentary units can be divided into three series of deposits based on their sequence and age of deposition. The oldest sequence consists of claystone, siltstone, sandstone, and conglomerate deposited during late Cretaceous time as an apparent submarine fan (Abbott, 1999). These units crop out on Mt. Soledad in La Jolla, Point Loma, and Carlsbad. The second sequence of sediments was deposited during the Tertiary (Eocene and Pliocene) period within an embayment that stretched from northern San Diego County into Mexico (Kennedy, 1975). The sediments consist of a variety of claystone, siltstone, sandstone, and conglomerate.

The most recent sedimentary deposits consist of early to late Pleistocene near shore marine, estuarine, and delta deposits, also typically identified as terrace deposits. Most of these sediments were deposited on wave cut surfaces (terraces) developed in response to sea level fluctuations during the Pleistocene. The oldest terrace deposits are typically identified as the Lindavista Formation and consist of conglomerate and sandstone with minor clay and silt strata. The youngest terrace deposits (late Pleistocene) are known as the Bay Point Formation. More recent geologic maps (Kennedy and Tan, 2007) have subdivided both the Lindavista and Bay Point Formations into numerous very old paralic deposits (Qvop1 – Qvop13) and old paralic deposits (Qop1 – Qop8) and dropped the previous formal names. The Regional Geologic Map is presented as Figure 3 and shows the local extent of the geologic units described herein.

3.2 REGIONAL FAULTING AND SEISMICITY

The San Andreas fault delineates the boundary between two global tectonic plates consisting of the North American Plate on the east and the Pacific Plate on the west and dominates the seismicity of the Southern California region (Wallace, 1990; Weldon and Sieh, 1985). It stretches from the Gulf of California in Mexico along a northwest alignment through the desert region of Southern California up to Northern California, where it eventually trends offshore north of San Francisco. Within Southern California, the mostly right-lateral strain associated with the plate boundary movement extends well westward for up to 150 miles (241 kilometers) from the main San Andreas fault in the Imperial Valley to well offshore of San Diego.

The major faults east of San Diego (from east to west) include the San Andreas fault, the San Jacinto fault, and the Elsinore fault (see Regional Fault Map, Figure 4). Major faults west of San Diego include the Palos Verdes-Coronado Bank fault, the San Diego Trough fault, and the San Clemente fault. The most dominant zone of faulting within the San Diego region are several faults associated with the Rose Canyon Fault Zone (RCFZ).

Most of the seismic energy and associated fault displacement occurs along the fault structures closest to the plate boundary on the Elsinore, San Jacinto, and San Andreas Faults. Approximately 49 mm/yr (1.9 inches/yr) of overall lateral displacement has been measured geodetically as fault slip across the plate boundary. The Elsinore, San Jacinto, and San Andreas Faults combined account for up to 41 mm/yr (1.6 inches/yr) of the total plate displacement (84%), meaning that the remaining 8 mm/yr (0.3 inches/yr) is accommodated across the faults to the west and east (Bennett, et al, 1996). Recent GPS measurements from the offshore islands to the peninsular ranges indicate about 5 to 7 mm/yr of plate movement is accommodated by the coastal and offshore system of faults, including the Rose Canyon.

Historically, San Diego County has long been considered as a region of negligible seismic hazard. Except for a probable local event in 1862 (Legg and Agnew, 1979), there has been a lack of significant seismic activity within the recorded human history of San Diego County. More recent studies have recognized that the potential for significant seismic events is much greater than earlier believed. This potential has been recognized by the discovery of many active fault traces associated with structures within the RCFZ. Studies within Rose Canyon (east of Mt.

Soledad) have revealed fault strands that have clearly displaced Holocene soil horizons with slip rates from 1 to 2 mm/yr (Lindvall and Rockwell, 1995).

These results indicated that at least the northern onshore portion of the RCFZ is active. Additional studies (Testing Engineers and other, 1985; Patterson and others, 1986; and Kleinfelder, 1998) within downtown San Diego revealed additional fault structures offsetting Holocene soil horizons, suggesting the possibility that the entire mapped onshore alignment of the RCFZ may be active.

More regionally, data has been presented that indicates that the RCFZ may be structurally connected to the Newport-Inglewood Fault Zone (Grant and Rockwell, 2002; Grant and Shearer, 2004) on the north and the San Miguel-Vallecitos fault or the offshore Descanso fault on the south, all of which are active faults. Sahakein, et. al. (2017) processed previously collected seismic reflection and bathymetric data, which indicated relatively narrow (2 kilometer) step-overs fault segments in offshore strands between the two major fault systems. This not only provides additional support of the structural connectively between the two fault systems but also indicates the possibility that they could erupt together with greater magnitude events of up to 7.5M. This larger fault system is thus over 150 miles in length.

3.3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

The results of our investigation indicate that the site is underlain by artificial fill over very old paralic deposits and the San Diego Formation. As part of our engineering analysis we reviewed historic aerial photography and topographic maps and compared these to the present conditions at the site. Information from this work in conjunction with our current and previous nearby boring data was used to estimate the depth and extent of geologic units below the site. Detailed descriptions of these materials are provided in the boring logs (Appendix A). Generalized descriptions are provided in the subsequent sections.

3.3.1 Artificial Fill (af)

Artificial fill material was encountered in all of the boring locations and extended to depths of approximately 2 to 5 feet below the ground surface within the borings. It is possible that deeper fill is present at the site in areas of utility trenches or buried structures such as underground storage tanks. The fill material consisted of l of lifts of clayey sand and sandy clay. Due to the lack of compaction records from previous site grading operations and location within a landscaped area, the fill material is considered undocumented and therefore unsuitable for supporting structural loads. It is likely that the fill was derived from nearby campus materials during site grading and construction for previous site development.

3.3.2 Very Old Paralic Deposits (Qvop9)

Very old paralic deposits underlie the fill at a depth of 2 to 5 feet or at approximate elevations of +287 to +290 feet MSL in the five borings. This unit is comprised of a very dense, light reddish yellow, silty sand with gravel and cobble. This material is typically moderately to highly cemented and was very difficult to drill, with near refusal of the boring auger at all locations. Excavation within this unit will be difficult, particularly for trenching operations of utilities and foundations.

3.3.3 San Diego Formation (T_{sd})

The San Diego Formation was encountered below the very old paralic deposits at depths of 8 to 10 feet or approximate elevations of +283 to +284 feet MSL in three of our deeper borings. This formation consists primarily of yellow to olive brown fine silty sand in a very dense and weakly to moderately cemented condition. It also contains some beds of sandy silt, and hard lean clay with sand. The sampler blow counts in this material obtained during drilling was typically in excess of 50 blows per foot, with a few isolated blow counts of 25 and 30 blows per foot. Based on our review of nearby borings for other projects, this unit is anticipated to have a thickness up to about 60.

3.3.4 Pomerado Conglomerate (T_p)

The Pomerado Conglomerate was not encountered within our current borings and is present well below the depths of construction. However, this unit was encountered directly below the San Diego Formation at approximate elevation +220 to +227 feet MSL in our previous borings east of the site for the replacement hospital. Observations of the slope exposures along 6th Avenue east of the site indicate the unit consists of a brown to yellowish brown, cemented cobble conglomerate. The cobbles are typically 3 to 6 inches in size, but larger cobbles and boulders greater than 12 inches in size are occasionally present on the slope outcrops to the east of site. The formation exposed on the slope outcrops typically contain between 20 to 50 percent cobbles.

3.3.5 Groundwater

Groundwater was not encountered in any of our borings. In general, most of the soils encountered are in a moist condition below saturation levels necessary for free groundwater conditions. It is possible that perched groundwater or seepage zones may be present at isolated locations. In particular, perched groundwater typically develops at the interface between more permeable fill and less permeable formational materials or between layers of variable permeability. It should be noted that groundwater levels at the site can fluctuate with time due to changes in weather, irrigation, construction, or other influences that were not present at the time the observations were made.

4 SEISMIC AND GEOLOGIC HAZARDS

We have reviewed the site with respect to the potential presence of geologic and/or seismic hazards. These hazards include landslides, expansive soils, liquefaction, seismic compression, fault surface rupture, and flooding. The following sections discuss these hazards and their potential at this site in more detail.

4.1 EXPANSIVE SOILS

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade.

Visual classification of the soils near anticipated subgrade elevations indicates that these soils primarily consist of non- to low-plastic silty sand and clayey sand. Based on the results of our field investigation and previous experience in the site area, it is our opinion that the site soils generally have a very low to low expansion potential. Isolated zones of more expansive soils may also be encountered but are not anticipated. No special requirements for footing and floor slab reinforcement are recommended from a geotechnical perspective.

4.2 SEISMIC GROUND SHAKING

The project site, like all Southern California, is a seismically active area and is likely to experience ground shaking as a result of earthquakes on nearby or more distant faults. The Rose Canyon fault zone and Elsinore fault zones dominate the seismicity of the area. The Rose Canyon fault zone (CDMG, 1998) is located approximately 1.5 miles southwest of the project site.

We understand that the proposed structure will be designed in accordance with the requirements of the latest 2019 edition of the California Building Code (CBC). It should be noted that the seismic provision of the 2019 CBC are based on and refer to (for more requirements) "Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7" (referred to herein as "ASCE 7").

Based on the results of our borings, we classify the site as Site Class C. Site Class C is defined as a very dense soil and soft rock with average shear wave velocities within the upper 30 meters between 1,200 ft/sec. and 2,500 ft/sec., average SPT N>50, or average undrained shear strength Su > 2,000psf. The recommended seismic design parameters are presented in Table 1.

Table 1

Recommended 2019 CBC Seismic Design Parameters

Design Parameter	Symbol	Recommended Value	2016 CBC (ASCE 7) Reference(s)
Site Class	--	C	Section 1613A.5.2
Mapped spectral acceleration for short periods	S_s	1.38g	Section 1613A.5.1
Mapped spectral acceleration for a 1-second period	S_1	0.47g	Section 1613A.5.1
Site Coefficient	F_a	1.2	Table 1613A.5.3(1)
Site Coefficient	F_v	1.5	Table 1613A.5.3(2)
MCE* Peak Ground Acceleration (S_M at T=0)	PGA_M	0.75 g	n/a
MCE* spectral response acceleration for short periods	S_{MS}	1.66 g	Section 1614A.1.1 (Section 21.4)
MCE* spectral response acceleration at 1-second period	S_{M1}	0.71 g	Section 1613A.5.3 (Section 21.4)
Design Peak Ground Acceleration (S_D at T=0)	PGA_D	0.86 g	Section 1802A.2.7
Design spectral response acceleration (5% damped) at short periods	S_{DS}	1.11 g	Section 1613A.5.4 (Section 21.4)
Design spectral response acceleration (5% damped) at 1-second period	S_{D1}	0.47 g	Section 1613A.5.4 (Section 21.4)

*MCE : Maximum Considered Earthquake

4.3 LIQUEFACTION

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or “cyclic mobility”, increased lateral earth pressures on retaining walls, post liquefaction settlement, lateral spreading and “flow failures” in slopes.

Based on the absence of near-surface groundwater, as well as the presence of dense to very dense formational soils, the potential for liquefaction of the subsurface soils at the site is considered low.

4.4 SEISMIC COMPRESSION

Seismic compression results from the accumulation of contractive volumetric strains in unsaturated soil during earthquake shaking. Loose to medium dense granular material with no fines or with low plasticity fines are most susceptible to seismic compression. The site will require excavation to proposed finish grade which we anticipate will be on the very dense San Diego Formation, therefore the hazard posed to the site by seismically induced settlement is considered low.

4.5 FAULT SURFACE RUPTURE

Review of readily available geologic and fault maps does not show any active or potentially active fault features passing through or nearby the site. The closest active fault to the site is the Rose Canyon fault, which is located approximately 1.5 miles to the southwest. The site is not within a State of California Alquist-Priolo Earthquake Fault Zone. In addition, published geologic maps do not show any faults crossing through or nearby the site. Finally, review of predevelopment aerial photographs do not show geomorphic features or lineaments indicative of faulting across the site. Based on this information, the geologic hazard with respect to fault rupture is considered low.

4.6 LANDSLIDES

Landslides are deep-seated ground failures (tens to hundreds of feet deep) in which a large section of a slope slides downhill. Landslides are not to be confused with minor surficial slope failures (slumps), which are usually limited to the topsoil zone and can occur on slopes composed of almost any geologic material. Landslides can cause damage to structures both above and below the slide mass. Undermining of foundations can damage structures above the slide area. Areas below a slide can be damaged by being overridden and crushed by the failed slope material.

The site is located in a relatively level area well away from slopes. Therefore, the potential for landsliding is considered slow.

4.7 TSUNAMIS AND SEICHES

A tsunami is a giant sea wave (which can reach over 50 feet in height) usually generated by catastrophic displacement on a submarine fault. Tsunamis can travel at speeds of hundreds of miles per hour over distances of thousands of miles. In the open ocean, tsunamis have large wavelengths and are difficult to detect. As the sea wave approaches shore, the wave decreases in wavelength and increases in amplitude (height). Large tsunamis can travel well beyond the normal wave break of the shoreline and cause damage to near shore structures.

A seiche is an oscillation (wave) of a body of water in an enclosed or semi-enclosed basin that varies in period, depending on the physical dimensions of the basin, from a few minutes to several hours, and in height from several inches to several feet. A seiche is caused chiefly by local changes in atmospheric pressure, aided by winds, tidal currents, and occasionally earthquakes.

The project site is located about 5 ½ miles from the Pacific Ocean and is located at an elevation of approximately 290 feet or more above mean sea level. Additionally, the site is not located adjacent to any large bodies of water that could adversely affect the site in the event of earthquake-induced failures or seiches. Therefore, the hazard with respect to a tsunami or seiche is considered low.

4.8 FLOODING

Flooding occurs as a result of several factors in developed areas. These factors include rainfall rates that exceed an area's ability to absorb or control the runoff; impounded water retained behind a flood control structure (upstream-inundation), failure of a flood control structure (downstream-indundation), Seiches, or tsunami.

The Federal Emergency and Management Administration (FEMA) maintains a collection of Flood Insurance Rate Maps (FIRM), which cover the entire United States. These maps identify those areas which may be subjected to 100-year and 500-year cycle floods. A set of these maps for the County of San Diego are available for viewing on the FEMA website (https://msc.fema.gov/portal). Based on our review of FEMA map panel 06073C1618G, the site is not within any designated flood zones and therefore the potential for flooding of the proposed development is considered low.

5 DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

Based on the results of our background review, subsurface exploration, laboratory test results, and engineering analyses, it is our opinion that the construction of the proposed MOB building is feasible from a geotechnical standpoint provided the recommendations of this report are incorporated into the design and construction of the project. The following recommendations were prepared based on our understanding of the project as depicted on the progress plans titled “Scripps Mercy MOB West,” prepared by CO Architecture and dated May 17, 2019.

5.1 SITE GRADING

The following recommendations were prepared based on our understanding of the project as previously described in this report. Kleinfelder should be provided with updated plans by the design team if design is modified.

5.1.1 General

All site preparation and earthwork operations should be performed in accordance with applicable codes including the current edition of the California Building Code, Section 1803A. All reference in this report to maximum dry density is established in accordance with American Society for Testing and Materials (ASTM) ASTM D 1557. We recommend that site earthwork and construction be performed in accordance with the following recommendations and guidelines presented in the Suggested Guidelines for Earthwork Construction included in Appendix D. In case of conflict, the following recommendations supersede those outlined in Appendix D.

5.1.2 Site Preparation

Since the excavation for the proposed structure will be on the order of 15 feet in depth, all of the existing pavement, hardscape, site improvements and landscaping will be demolished and removed prior to construction of the proposed building. Man-made structures, including buried pipes, utilities, etc., should be completely removed within the building pad. Subsurface utilities should be rerouted or plugged and capped at the building perimeter. Excavations for removal of any man-made items should be dish-shaped and backfilled with properly compacted engineered fill per Section 5.1.4. All surficial vegetation and deleterious material should be stripped and completely removed from the proposed site area.

5.1.3 Subgrade Preparation

Excavations for the building foundations are anticipated to be on the order of 15 feet and into native soils of the San Diego Formation. The exposed surface should may be left above finish subgrade elevation during foundation construction and disturbed soils excavated to expose undisturbed formational soils. Remaining disturbed soils may be moisture conditioned and compacted. The need for scarification to provide adequate moisture conditioning may be evaluated by the geotechnical engineer at the time of grading.

5.1.4 Engineered Fill

The majority of existing undocumented fill material is considered suitable for re-use as engineered fill outside of the building footprint, however the clay soils may be unsuitable considering the volume of granular soils which will be generated from the building excavation. The onsite San Diego Formation materials may be used as engineered fill, as they are not anticipated to contain oversized rock, organic materials, expansive clay, and deleterious debris. If encountered, oversize material in excess of 6 inches in diameter should not be used in engineered fill and material larger than 3 inches should not be used within the upper 6 inches for foundation subgrade. The onsite soil placed as engineered fill should be moisture conditioned between 0 and 3 percent above optimum moisture content and compacted to a minimum of 95 percent relative compaction in structural areas, and 90 percent in non-structural areas, based on ASTM D 1557.

Import materials used as engineered fill (if any) should consist of clean, granular material that has less than 30 percent passing the #200 sieve, a minimum R-value of 20, and expansion index of 20 or less as evaluated by ASTM 4829. Imported engineered fill should be moisture conditioned between 0 to 3 percent above optimum moisture content and compacted to a minimum of 95 percent relative compaction in structural areas and 90 percent in non-structural areas, based on ASTM D 1557.

Although the optimum lift thickness for fill soils will be dependent on the size and type of compaction equipment utilized, fill should generally be placed in uniform lifts not exceeding approximately 8 inches in loose thickness. Oversized material, rocks, or hard clay lumps greater than 6 inches in dimension should not be used in compacted fills and greater than 3 inches in dimension should not be used in the upper 3 feet in structural areas. In pavement and exterior flatwork areas the upper 12 inches of subgrade soils should be moisture conditioned 0 to 3 percent above optimum content, and compacted to at least 95 percent relative compaction, just prior to placement of aggregate base.

5.1.5 Pipe Bedding and Trench Backfill

Pipe bedding should consist of sand or similar granular material having a sand equivalent value of 30 or more. The sand should be placed in a zone that extends a minimum of 6 inches below and 12 inches above the pipe for the full trench width. The bedding material should be compacted to a minimum of 90 percent of the maximum dry density. Trench backfill above pipe bedding may consist of approved, onsite or import soils placed in lifts no greater than 8 inches loose thickness and compacted to 90 percent of the maximum dry density.

5.1.6 Temporary Slopes for Shallow Excavations

Shallow, temporary utility trench excavations are anticipated for installation of the required utility lines. If very steep or vertical-sided excavations in excess of 5 feet deep are necessary, we recommend the sidewalls be shored in accordance with OSHA standards to provide temporary trench stability during construction. The contractor should be responsible for the structural design and safety of the temporary shoring system and we recommend that this design be submitted to Kleinfelder for review and approval.

For preliminary planning of OSHA sloping and shoring requirements, we recommend that fill materials be considered as Type C soils and that native formational materials be considered as Type B soils. The actual OSHA soil type should be determined by the contractor's "competent person" based on conditions exposed in the field.

Heavy construction loads, such as those resulting from stockpiles and heavy machinery, should be kept a sufficient distance away from the top of the excavation or shoring to prevent unanticipated surcharge loading. All surface water should be diverted away from excavations.

5.1.7 Excavation Considerations

Excavation in the existing fill and San Diego Formation can generally be accomplished with conventional, heavy earthmoving equipment in good operating condition. Excavation into the very old paralic deposits is anticipated to be difficult due to cemented layers of cobbles and gravel. Potential excavations for utility trenches, soldier beams for shoring, through this unit may be particularly difficult due to cementation. cobbles and limited size of excavation.

5.1.8 Dewatering

Static groundwater was not encountered within the exploratory borings performed by Kleinfelder. Elevated moisture is possible from the landscape areas overlying relatively impermeable permeable formation soils. The regional groundwater table is probably in excess of 100 feet below the hospital campus. However, some minor seepage may be encountered in the excavations due to perched groundwater that may be located near the contact of differing geologic units or soil types. Perched groundwater, if encountered, may require collection, control and disposal of minor amounts of water.

5.2 TEMPORARY SHORING

While the details of site excavation and temporary excavation support are not known at this time, we anticipate that the proposed excavation will require temporary shoring around most, if not all, of the perimeter of the site during construction to protect existing improvements such as buildings, roads and hardscape areas, utilities, power poles, etc. Depending on the proximity to the existing hospital on the east, underpinning may also be necessary. Depending on final plans, the shoring height may be on the order of approximately 15 feet. This is near the conventional limit of cantilever shoring and the need for a row of tieback anchors.

Shoring is anticipated to consist of closely-spaced soldier piles and wooden lagging as it is typically the most feasible and economical method for soils in this area. While soil nailing is technically feasible, permanent encroachment into City streets is unlikely and the cemented cobble conglomerate of the old paralic deposits would be problematic. If tiebacks are required to limit lateral deflections due to the shoring height and surcharge loads, the City of San Diego would require removal of the upper portion of the anchors within their right-of way. Drilling holes for soldier beams through the very old paralic deposits should consider the appropriate diameter to remove cobbles and the effort to penetrate cemented zones. Based on our experience with nearby projects, the caving potential of the onsite soils is low to moderate for undocumented fill and low for formational materials. Recommendations for this system are provided in the following sections.

To accommodate installation of the soldier beams, wide-flange beam sections may be installed into pre-drilled holes surrounded by concrete below excavation depth and cement slurry within the lagging depths. In the unanticipated event that caving of the drilled holes occurs, drilling slurry or casing may be required. In addition, caving of drilled holes for tieback anchors is possible in undocumented fill.

5.2.1 Tieback Anchors

If needed, tiebacks derive their load capacity through frictional resistance along the grouted “bonded zone” which is located beyond the active wedge. For design of tieback anchors, we recommend the active wedge may be assumed at an angle of 30 degrees from vertical, passing through a point located at least 5 feet behind the bottom of the excavation. We recommend the portion of the anchor within the “unbonded zone” within the active wedge either have a sleeve so that it is not bonded to the grout or be backfilled using sand/cement slurry. The shoring contractor should determine the suitable drilling method for tieback installation based on the subsurface conditions at the site and on their experience with similar materials. Tiebacks should be installed at angles between 15 and 30 degrees from horizontal.

Since the load-carrying capacity of the tieback anchors will depend on various site-specific equipment and method-related factors, design tieback capacities should be confirmed by performance testing. We recommend performance testing and proof testing of anchors be performed in accordance with the latest edition of the Post-Tensioning Institute’s (PTI)

Recommendations for Prestressed Rock and Soil Anchors. Performance test for the anchors shall be at a minimum of two (2) times the design loads and shall not exceed 80 percent of the specified minimum tensile strength of the anchor rod. A creep test is required for all pre-stressed anchors that are performance tested. All production anchors shall be tested at 150 percent of design loads and shall not be greater than 70 percent of the specified minimum tensile strength of the anchor rod

Based on experience from nearby projects, the allowable unit friction between grout and the soil may be assumed to be on the order of 2,000 psf for design of tiebacks if post-grouting is performed. However, it is important that the unbonded length not be grouted if post-grouting is performed. If post-grouting is not performed, we recommend an allowable unit friction of 1,000 psf be used. If tieback anchors are installed at an angle below the horizontal, tieback resistance should be taken as the horizontal component of the total anchor capacity. Additionally, the shoring designer should be aware that the vertical component of the total anchor capacity may act as a downward load on the shoring system.

5.2.2 Timber Lagging

Timber lagging may be used between the soldier piles to support the exposed soils. Since the lagging is generally left in place, treated lumber should be used. Lagging should be designed for the lateral pressures recommended in Section 5.2.3 of this report. The soil bridging between the stiffer soldier beam elements and the intermediate wooden lagging results in a reduced lateral earth pressure in the lagging area. The CBC allows soil arching effects to be considered in the design of lagging. We recommend 0.6 of the design earth pressure, or a uniform pressure of 400 psf for level ground without surcharge. It is our understanding that conventional design of earth pressures for soldier beam and wooden lagging walls incorporate this pressure into standard design tables for wooden lagging but do not include surcharge pressures. Therefore, additional surcharge pressures should be added if these tables are utilized.

If possible, structural walls should be cast directly against the shoring, eliminating the need for backfilling a narrow space. Voids between the soil and lagging should be grouted to mitigate the potential for the voids to propagate to the surface and to protect the existing improvements. Voids identified during lagging operations may also be immediately filled by compacting soil behind the lagging.

5.2.3 Lateral Pressures

Cantilever shoring can typically be used for retained shoring heights up to about 12 to 15 feet with level backfill and no surcharge loading, however this height could be increased by using larger beams. Tie-back anchors would likely be needed for surcharges, higher walls or sloping backfill. Cantilever shoring supporting undocumented fill should be designed to resist an equivalent fluid weight of 35 pcf for level ground conditions in fill and 30 pcf for level ground conditions in formational material. Thirty percent of any areal surcharge adjacent to the shoring (including soil stockpiles) may be assumed to act as a uniform horizontal pressure against the shoring for cantilever conditions.

Tied-back shoring systems should be designed to resist a trapezoidal horizontal soil pressure with an equivalent fluid weight of 25H (in psf), where H is the wall height in feet. Increased loads due to ascending slopes should also be added to this. As an approximation, thirty percent of any permanent or temporary surcharge loads adjacent to the shoring (including existing structures, temporary soil stockpiles, material staging, construction trailers, etc.) may be assumed to act as a uniform horizontal pressure against the shoring. A better estimate of horizontal pressure resulting from foundations adjacent to the shoring system can be estimated using the method presented on Figure 5 (based on US Navy, 1986). Strip footings can be represented as line loads and spread footings can be represented as point loads. Special cases such as combinations of sloping and shoring or other surcharge loads (not specified above) may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical engineer on a case-by-case basis. The above pressures do not include hydrostatic pressures since groundwater was not encountered within the depths explored at the site.

All soldier piles should extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. An equivalent fluid unit weight of 400 pcf may be used or allowable passive pressure against soldier piles that extend below the level of excavation. To account for three-dimensional effects, the passive pressure may be assumed to act on an area 2 times the width of the embedded portion of the pile, provided adjacent piles are spaced at least 3 pile diameters center-to-center. Additionally, we recommend a factor of safety of 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 4,000 psf.

Lateral movement of a shored excavation will depend on the type and relative stiffness of the system used, the workmanship of the contractor, and other factors beyond the scope of this study. The shoring engineer should design the system to limit lateral movements and settlements so that effects on adjacent structures, existing utilities to remain in place, and other existing site improvements are minimized.

In addition to monitoring of existing structures, horizontal and vertical movements of the shoring system should also be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor's responsibility. At a minimum, we recommend that the tops of the soldier beams be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the foundation is completed. The soldier beams should be surveyed at spacings no greater than 50 feet on-center.

5.3 FOUNDATION RECOMMENDATIONS

Foundations should be supported entirely on either formational material or compacted fill to mitigate the potential for differential settlement. Based on the currently proposed depth of excavation, very dense soils of the San Diego Formation should be present at foundation elevation. The structure may be supported on spread and continuous foundations, or a mat foundation.

5.3.1 Spread Foundations

For foundations entirely in formational soils, an allowable foundation pressure of 5,000 may be used. For foundations entirely in compacted fill soils such as near surface retaining walls or equipment pads, an allowable foundation pressure of 3,000 pounds per square foot (psf) can be used. The allowable design bearing values can be increased by one third for transient loading due to seismic and wind forces.

Anticipated total settlements will be evaluated when foundation loads are provided but are not expected to exceed 1 inch. Differential settlements over a 40-foot span are not expected to exceed 50 percent of the total settlement. Shallow foundations should contain reinforcing steel as determined by the project structural engineer.

Resistance to horizontal loadings can be developed by passive earth pressure on the sides of footings and frictional resistance developed along the footings bottoms. Passive resistance to lateral earth pressures may be calculated using an allowable equivalent fluid unit weight of 450 psf within formational soils or 350 psf within compacted fill. A frictional coefficient of 0.35 may be applied to vertical dead loads supported either formational or fill soils. The passive pressure and frictional resistance can be combined without reduction to resist lateral loads.

Footings may experience a reduction in bearing capacity or an increased potential to settle when located near existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse, and/or a loss of serviceability. To reduce this risk, utility excavations should not extend below a 2H:1V plane projected downward from 12 inches above the bottom of the outside edge of the footing. Also, no parallel utility excavations should be made within a lateral distance of 18 inches outside the footing.

Prior to placing reinforcing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. All footing excavations should be observed by the project geotechnical engineer immediately prior to placement of reinforcing steel and concrete to check that the recommendations contained herein are implemented during construction.

5.3.2 Mat Foundations

If utilized, a mat foundation bearing on formational materials may be designed using an allowable bearing pressure of 8,000 psf. We recommend an uncorrected vertical modulus of subgrade reaction for a one-foot square plate, k_1, of 250 pci be used for design of the mat foundation. The k_1 value is based on a settlement of 1 inch for a one-foot square plate and should be modified to account for the mat foundation width, B. The modified vertical modulus of subgrade reaction may be calculated using the following equation with actual mat dimensions:

$$k_s = k_1\left(\frac{B+1}{2B}\right)^2$$

Where:

k_1 is the subgrade modulus for a one-foot square foundation (= 250 pci).

k_s is the subgrade modulus in pci for a foundation with width B (B is the least dimension of the mat in feet). This value approaches 0.25 k_1 for large dimensions.

The allowable design bearing value can be increased by one third for short-term loading due to seismic and wind forces. The mat foundation should have a minimum embedment of 18 inches and should be designed by a licensed structural engineer in the state of California for the specific loading conditions.

5.4 INTERIOR CONCRETE SLABS-ON-GRADE

Interior concrete slabs-on-grade may be used with structures not supported on mat foundations. Engineered fill or disturbed formational soils supporting concrete slabs should be scarified to a depth of 6 inches, moisture conditioned to within 3 percent above optimum and compacted to at least 95 percent relative compaction per ASTM D1557. A modulus of subgrade reaction, k, of 175 pounds per cubic inch (pci) may be used to design floors, pavements, and walkways on the compacted subgrades. For slabs subjected to pedestrian-type loadings, we recommend a minimum floor slab thickness of 4 inches.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, and/or curling of the slabs. High water-cement ratio and/or improper curing will increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

The floor slab should be underlain by at least 4 inches of clean coarse sand or fine gravel to provide a capillary moisture break and uniform support to the slab. In cases where the floor may have a vapor/moisture sensitive covering (e.g. tile, linoleum, carpet, wood), may be in a humidity controlled environment, or may likely have one or both of these conditions in the future, we recommend a polyolefin vapor barrier membrane be utilized between the prepared subgrade and the bottom of the floor slab.

Subsurface moisture and vapor naturally migrate upward through the soil. Where the soil is covered by a building or pavement, this subsurface moisture will collect and transmit through the concrete slab-on-grade. Traditional Visqueen vapor barriers may be considered marginally effective and have been shown to eventually disintegrate with time. To reduce the impact of this subsurface moisture and the potential impact of future introduced moisture (such as landscape irrigation or precipitation) we recommend utilizing a polyolefin vapor barrier membrane between the subgrade and slab-on-grade. This vapor barrier membrane should consist of a polyolefin sheeting at least 15 mil in thickness, have a water vapor permeance less than 0.01 perms (ASTM F1249), a puncture resistance of at least 2,200 grams (ASTM D1709), and a tensile strength of at least 45 lbf/in (ASTM D882).

The polyolefin vapor barrier membrane described above should be highly resistant to tearing, cracking, flaking, or puncturing during construction and should not disintegrate with time. A granular subbase below the membrane or a sand or gravel layer on top of the membrane is not required. In accordance with recommendations in ACI guidelines and many flooring companies, placement of the concrete slab may be directly on the vapor barrier. This eliminates the potential for water to be trapped in the blotter layer that could later be transmitted through the slab and adversely affect the flooring system. However, a reduced joint spacing, slab reinforcement, a low shrinkage mix design, and/or other measures to reduce the potential for slab curl should be implemented by the concrete slab designer.

We recommend that the vapor barrier be installed in accordance with ASTM E1643, "Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs". Some salient features of ASTM E1643 are discussed below. All joints and seams should have a minimum 6-inch overlap and be taped. The area of tape adhesion should be free from dust, dirt, and moisture. All penetrations must be sealed using a combination of membrane, tape, and mastic. The tape and mastic used should conform to the vapor barrier manufacturer's recommendations. Care should be taken at the lateral terminations so that vapors do not go around the membrane. This may be accomplished by placing the membrane on top of the footing and against the vertical wall so that the membrane will be sandwiched between the footing, vertical wall, and poured concrete floor slab. If damaged, the membrane should be repaired prior to placing concrete.

It is emphasized that we are not floor moisture-proofing experts. We make no guarantee nor provide any assurance that use of the capillary break will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level. The builder and designers should consider all available measures for slab moisture protection. Exterior grading and/or adjacent landscaping have an impact on the potential moisture beneath floor slabs. Exterior grading and/or adjacent landscaping should be designed to address the potential for increased moisture below moisture sensitive slabs and should at least reference the recommendations contained in Section 5.8 (Site Drainage) of this report.

5.5 PERMANENT BASEMENT AND SITE RETAINING WALLS

Permanent restrained retaining walls for the subterranean levels will likely be placed both against the temporary shoring in areas where used and be backfilled in potential areas where temporary excavation slopes are used. After permanent bracing such as floor slabs have been installed, it is important that the tieback anchors used for the shoring (if any) be detentioned and documented by the geotechnical engineer. The upper portion of the tiebacks will also need to be removed from City right-of-way. Cantilever retaining walls are not anticipated but recommendations are provided in the following sections if site walls are included in project design.

The basement walls should be dampproofed and permanent drainage provisions should be made against the walls. Vertically placed composite geosynthetic drainage panels are typically used rather than open graded gravel. The wall drains should be connected to closed conduits at the base of the walls and brought to a storm drain, sump or other suitable discharge location.

5.5.1 Wall Foundations

The perimeter basement walls will be supported on the foundation for the building. Recommendations for foundation design were previously provided in Section 5.3 of this report. Although not anticipated, cantilevered masonry and poured-in-place concrete walls with shallow reinforced concrete footings are considered suitable for potential site retaining walls. Cantilevered concrete retaining walls may be supported on shallow continuous footings founded entirely on either engineered fill or undisturbed native formational soil. Where existing undocumented fill soils are present, the fill should be removed and recompacted so that retaining walls over 3 feet in height are supported by engineered fill. Shallow foundations supported on engineered fill or formation should be designed for an allowable bearing pressure of 3,000 psf. Estimated total settlements for retaining walls constructed in accordance with the recommendations contained herein are anticipated to be less than ¾ inch. Differential settlements are expected to be less than ½ inch within 40 feet.

Retaining wall foundations should be embedded at least two feet below the lowest adjacent grade or to the depth necessary to provide adequate factors of safety against sliding and overturning as determined by the retaining wall designer, whichever is greater. Reinforcement should be provided as required by the Regional Standard Drawings (if used) or as directed by the wall designer for load carrying purposes.

All footings should be extended in depth as necessary so that no existing or proposed utility trenches will extend below a plane having a downward slope of 2H:1V from 12 inches above the bottom edge of the footing. In addition, no parallel trenches should be within 18 inches from the closest edge of the footing. New footings should not be excavated below the bottom of adjacently located existing foundations. For all retaining walls, we recommend a minimum horizontal distance from the outside face of the footing to daylight of 6 feet.

5.5.2 Static Lateral Earth Pressures

Lateral pressures acting against masonry and poured-in-place concrete retaining walls can be calculated using soil equivalent fluid weight (efw). The efw value used for design depends on allowable wall movement. Walls that are free to rotate at least 0.5 percent of the wall height can be designed for the active efw. Retaining walls that are restrained at the top (such as basement walls), or are sensitive to movement and tilting should be designed for the at-rest efw.

Values given in the Table 2 below are in terms of equivalent fluid weight and assume a triangular distribution. These values assume that the wall is cast against the temporary shoring, or onsite or imported, sandy soils (SP, SM, SC) will be used as backfill and that the backfill is well drained and above the static water table. If walls with undrained backfill are to be used Kleinfelder should be consulted for additional evaluation and recommendations.

Table 2

Equivalent Fluid Weights (efw) for Calculating Lateral Earth Pressures

Wall Type	Conditions	Level Backfill (psf)
Restrained - Basement Wall	At-Rest	55
Cantilever - Site Wall	Active	35

Fifty and thirty percent of any uniform areal surcharge placed at the top of the wall may be assumed to act as a uniform horizontal pressure over the entire wall for the at-rest and active cases, respectively. As a minimum, we recommend that a traffic surcharge equivalent to 2 feet of soil backfill be assumed as a surcharge for the at-rest condition. For this condition a pressure of 120 psf may be assumed to act as a uniform horizontal pressure over the entire height of the wall, H.

For passive resistance on retaining wall foundations, we recommend using an allowable equivalent fluid weight of 450 psf for undisturbed formational material or 350 pcf for footings poured neat against properly compacted engineered fill. The upper 12 inches of material in areas not protected by adjacent concrete slabs or footings should not be included in design for passive resistance to lateral loads. The allowable coefficient of friction between the bottom of the footing and formational soil or engineered fill can be assumed as 0.35. The passive pressure and frictional resistance can be combined to resist lateral loads, provided that the larger value is reduced by 50 percent.

5.5.3 Seismic Design of Retaining Walls

Retaining walls should be designed to resist dynamic earth pressures from earthquake loading. For both cantilever and restrained conditions, walls can be designed using an incremental seismic force of $12H^2$ for the Design Earthquake PGA (in pounds per linear foot of wall length, with H as the wall height in feet), which are additive to the static active earth pressure described above. The incremental seismic force acts at 0.5H above the base of the wall.

Allowable bearing pressure values described in previous sections of this report can be increased by one-third when calculating resistance caused by loads of short duration, such as earthquake loads. Restraining passive pressure and friction values should not be increased by this amount, but a lower factor of safety than is normally applied to static loads could be used. This factor of safety for dynamic load conditions should not be less than 1.2.

5.5.4 Wall Drainage

The above-recommended values do not include lateral pressures due to hydrostatic water pressures generated by infiltrating surface water that may accumulate behind the walls. Therefore, wall backfill materials should be free draining and provisions should be made to collect and remove excess water that may accumulate behind earth retaining structures.

Wall drainage may be provided by free-draining gravel surrounded by non-woven synthetic filter fabric or by prefabricated, synthetic drain panels. In either case, drainage should be collected by collector pipes at the base and directed to a sump, storm drain, weep hole(s), or other suitable location for disposal. The drainage discharge should not be permitted to discharge over soil in a manner that would cause erosion. If utilized, we recommend that drainage gravel consist of durable stone having 100 percent passing the 1-inch sieve and zero percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a permeability of at least 0.02 centimeters per second, a minimum flow rate of 50 gallons per minute per square foot of fabric, and a minimum puncture strength of 50 pounds. The geotextile manufacturer's recommendations should be followed for installation of a drainage fabric system.

5.6 EXTERIOR CONCRETE FLATWORK

All flatwork and exterior concrete should be supported on at least 12 inches of compacted, low to very low expansive engineered fill. The concrete slabs for walkways and sidewalks should have a nominal thickness of 4 inches. The exposed subgrade should be scarified to a depth of 6 inches, uniformly moisture conditioned to between optimum and 3 percent above optimum moisture content and compacted to at least 95 percent relative compaction as per ASTM D1557.

5.7 PRELIMINARY CORROSIVE SOIL SCREENING

A preliminary corrosive soil screening for the onsite materials was completed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory testing on two representative soil samples obtained during our current subsurface evaluation and review of previous nearby results. Laboratory testing was performed to evaluate soluble chloride, soluble sulfate content, resistivity and pH of soil. Results of the tests are provided in Appendix B.

Caltrans defines a "corrosive site" as one where one or more representative soil and/or water samples contain concentrations of soluble chloride of 0.05 percent (by weight) or greater, soluble sulfate concentrations of 0.2 percent or greater or the pH is 5.5 or less. Based on the laboratory test results, this site is considered "non-corrosive" by the Caltrans definition. The Portland Cement Association (PCA, 1988) defines concrete exposure to sulfate attack as negligible for soil with a water soluble sulfate content of 0.00 to 0.10 percent (by weight), moderate for a sulfate content of 0.10 to 0.20 percent, severe for a sulfate content of 0.2 to 2.00 percent, and very severe for a sulfate content over 2.00 percent. Test results indicate concrete exposure to sulfate attack as low for this site.

Upon saturation, the minimum resistivity result of the test was 1,900 ohm-cm. A test result from the concurrent investigation for the support building to the south indicated a minimum resistivity of 910 ohm-cm. Resistivity values under 500 are considered very corrosive and those between 1,000 and 2,000 ohm-cm are considered moderately corrosive.

We recommend that the corrosion test results be reviewed and evaluated by the project designers considering the proposed improvements and project lifespan requirements. A corrosion engineer can be contacted for detailed evaluation of corrosion potential and corrosion resistant design.

5.8 SITE DRAINAGE AND EROSION CONTROL

Final elevations at the site should be planned so that positive drainage is established around structures. Positive drainage is defined as a slope of 2 percent or more for a distance of 5 feet or more away from structure foundations. Roof gutters and downspouts should be installed on structures. Downspouts should discharge to controlled drainage systems and drainage gradients should be maintained to carry all surface water off the site. Ponding should not occur on the site.

Planters should be built so that water exiting from them will not seep into the foundation areas or beneath slabs and pavement. In any event, the maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to properly sustain the landscaping plants. Should excessive irrigation, waterline breaks, or unusually high rainfall occur, saturated zones and perched groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the foundation or landscaped areas. Potential sources of water, such as water pipes, drains, garden ponds, and the like, should be frequently examined for signs of leakage or damage. Any such leakage or damage should be repaired promptly.

5.9 SIGN AND LIGHT POLE SUPPORT

Proposed sign structures and light standard foundations as columns embedded in earth or embedded in concrete footings in the earth to resist both axial and lateral loads, can be designed in general accordance with Section 1807 of the 2016 California Building Code (CBC). We have conservatively assumed that foundations will be embedded in fill materials with the foundation properties as Class 4 Material as defined by the CBC, Table 1806A.2. We recommend that a lateral soil-bearing pressure of 150 lbs/ft^2 per foot of depth below natural grade be used. An allowable soil-bearing pressure of 2,000 lbs/ft^2 may be used to support vertical loads. The allowable lateral soil-bearing pressure may be increased by a factor of 2 for short-term lateral loads, as allowed in Section 1806A.3.4 of the 2016 CBC, provided the structures will not be adversely affected by a ½ inch of motion at the ground surface.

5.10 STORMWATER INFILTRATION STUDY

We have evaluated the site in conformance with the City of San Diego 2018 BMP Design Manual. For the purpose of this report, infiltration is defined as the flow of water through the ground surface and percolation is defined as the downward flow of water through the subsurface soil layers. Infiltration may be controlled primarily by factors such as the type and porosity of the surface filtering media, maintenance of these media, surface slope, surface vegetation, and intensity, duration, and type of precipitation. Percolation may be controlled primarily by the soil types and properties such as grain size and density, soil layering, porosity, hydraulic head, and the proximity to the groundwater. Surface drainage and maintenance will largely determine the site's infiltration rate and the amount of water that will infiltrate for any given storm. The percolation rate will depend locally on the soil layering and will be controlled by the finer grained soil layers.

Borehole percolation testing was the selected method for field infiltration testing at the site. Two percolation tests were performed at two different locations of the site. The percolation tests were performed in general accordance with those set forth in California Test 750, "Method for Determining the Percolation Rate of Soils Using a 6-Inch-Diameter-Test Hole". The tests were performed in drilled holes advanced to a depth of 5 feet below existing site grades. The measured percolation rates have been converted to an adjusted short-term infiltration rate based on borehole geometry using the Porchet Method (Ritzema, 1994) and are presented in Table 3. These values are converted to long term design infiltration rates later in this report by using correction factors based on Worksheet D.5-1 of the BMP Design Manual.

Table 3

Summary of Adjusted Infiltration Rates

Boring	Tested Depth from Ground Surface (feet)	Adjusted Short Term Infiltration Rate (inch/hour)	Soil Description
PERC-1	5	0.04	Silty Sand with Gravel
PERC-2	5	0.01	Silty Sand with Gravel

Note that relatively clean water was used to perform the tests above. However, surface runoff water from the site would likely contain silt, clay, oil and/or other materials that would eventually decrease the percolation rates. The provided field percolation rates in Table 3 do not include reduction factors for long term performance. These values are converted to long term design infiltration rates later in this report by using correction factors based on Worksheet D.5-1 of the BMP Design Manual.

Based on visual soil classifications and laboratory testing of the two soil samples collected during our field exploration at the percolation test locations, subsurface materials mostly consist of silty sand within the depths of the test. Laboratory testing consisted of sieve analyses from samples obtained at the bottom of the borings and indicated a fines content of 14 and 34 percent passing the #200 sieve for borings Perc-1 and Perc-2, respectively. The results are presented on the boring logs and lab results in Appendix A and Appendix B, respectively.

5.10.1 Mitigation Measures

The following bullets present typical considerations (geotechnical and other) for implementation of infiltration systems, along with site specific conditions.

- Presence of fill soils below building footprint. The site is underlain by about 2 to 5 feet of fill, however the proposed building foundation depth will be approximately 15 feet from existing grade.
- Presence of shallow formational material. The site has very old paralic deposits approximately 2 to 5 feet in depth from the ground surface and the San Diego Formation approximately 8 to 10 feet in depth from the ground surface. Water from overlying BMPs would likely perch on the less permeable formation materials and move laterally to the more permeable material in utility trenches or wall backfill for the subterranean level.
- Building sites located adjacent to or within landslide hazard areas or hillside grading areas. The site is not located near landslide hazard areas.
- Sites with initial seasonal high groundwater elevation within 10 feet of the invert of a proposed basin. The sites are not within 10 feet of high groundwater table.
- Site soils with a moderate or high potential for liquefaction. The sites have a low potential for liquefaction.
- Site soils with a moderate or high expansion potential. The majority of observed soils within the infiltration test areas appear to have low expansion potential.
- Sloping sites. The proposed BMP basin sites are generally in flat and/or near gently sloping areas.
- Sites with soil and/or groundwater contamination. According to the California State Water Resources Control Board Geo Tracker Database, contamination has not been identified in the near the project site.

5.10.2 Data Evaluation

The results of the field testing program provide a design infiltration rate based on correction factors contained within Tables D.5-1 and D.5-2 of the 2018 BMP Design Manal. For preliminary feasibility of infiltration purposes, a factor of safety of 2.0 is used with the non-factored infiltration rates.

5.10.3 Design Infiltration Rates

Based on our evaluation of the percolation test data discussed in a preceding section of this report, the soils encountered exhibit infiltration rates for *short-term, non-factored infiltration rates between **0.04** and **0.01** inch/hour*. The long-term design infiltration rate was calculated by using the following correction factors based on Worksheet D.5-1 of the BMP Design Manual. The completed worksheets are presented in Appendix C. Design infiltration rates have been estimated for PERC-1 through PERC-2 and the values are presented in Table 4. However, feasibility of infiltration for preliminary purposes a factor of safety of 2.0 must be used with the non-factored infiltration rates. For preliminary purposes the design infiltration rates are **0.02** and **0.01** inch/hour.

Table 4
Design Infiltration Rates

Boring	Safety Factor	Long Term Design Infiltration Rate (Inch/hour)
PERC-1	2.0	0.02
PERC-2	2.0	0.01

5.10.4 Recommendations and Conclusions

Based on the design infiltration rates less than 0.5 in/hr and the completed Geotechnical and Groundwater Investigation Requirement Worksheet C.4-1 contained in the BMP Design Manual, we classify the site as a feasibility screening category of “No Infiltration”. The completed C.4-1 worksheets for each BMP location proposed at the site are included in Appendix C of this report.

Based on the field percolation testing, geotechnical observations, laboratory data, and completion of the BMP Manual Worksheets, it is our opinion that the project site is categorized as not suitable for infiltration. The site is underlain by very dense and cemented very old paralic deposits and the San Diego Formation. In our opinion, the underlying formation material has low void ratio and cementation therefore low permeability characteristics. We recommend that BMPs be lined with an impermeable liner and located as far away from proposed building foundations and utility trenches as feasible in order to minimize the potential effects of lateral migration of water.

6 ADDITIONAL STUDIES

The recommendations within this report were based on progress drawings and should be updated if final design differs from that assumed herein. The review of plans and specifications, and the observation and testing by Kleinfelder of earthwork related construction activities, are an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client will be assuming our responsibility for any potential claims that may arise during or after construction. The required tests, observations, and consultation by Kleinfelder during construction include, but is not limited to:

- A review of plans and specifications;
- Observation of site clearing;
- Construction observation and density testing of fill material placement, trench backfill, subgrade preparation, and aggregate base for pavements; and
- Observation of foundation excavations and foundation construction.

7 LIMITATIONS

This report has been prepared for the exclusive use of Scripps Health and their consultants for specific application to the subject project. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No warranty, express or implied, is made.

The scope of services was limited to the field exploration program described in this report. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions presented herein are based on field explorations, laboratory testing, engineering analyses and professional judgement.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues addressed in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, laboratory tests, and our understanding of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, or locations of the improvements, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid until the

changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, ground improvement, preparation of foundations, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on site and off site), or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

Kleinfelder will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials. Additional important information about this report is presented in the attached Geotechnical Business Council insert in Appendix G.

8 REFERENCES

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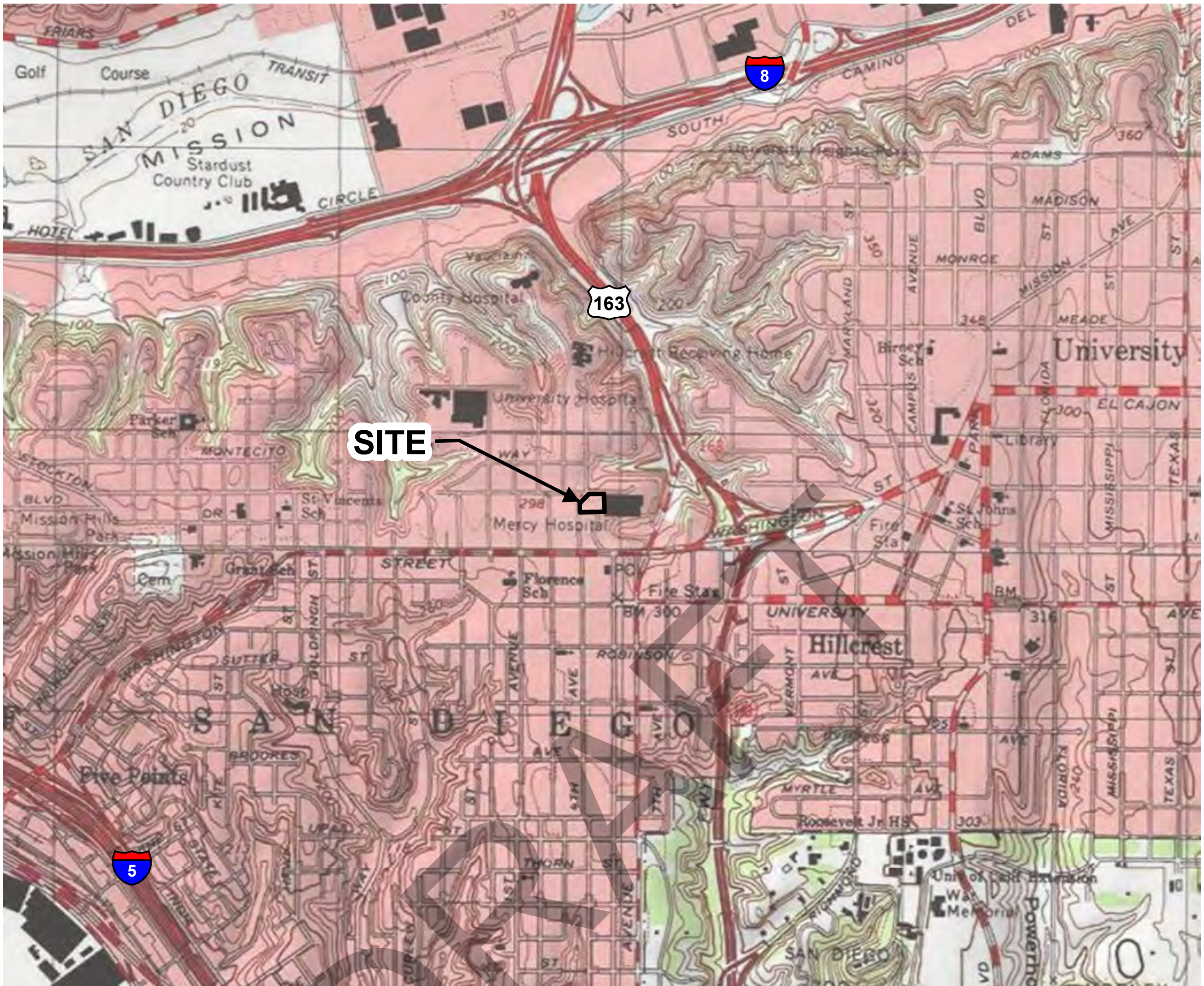
FIGURES



0 1,000 2,000 4,000 Feet

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PROJECT NO.	20194095
DRAWN:	5/21/2019
DRAWN BY:	JP
CHECKED BY:	ST
FILE NAME:	20194095_Vic_Merc.mxd

VICINITY MAP

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

1

\\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20194095_Mercy\20194095_Vic_Merc.mxd

\\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20194095_Mercy\20194095_Site_Merc.mxd

LEGEND

APPROXIMATE LOCATION OF BORING

APPROXIMATE LOCATION OF PERCOLATION TEST

0 25 50 100 Feet

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DRAWN BY:	JP
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FILE NAME:	20194095_Site_Merc.mxd

EXPLORATION MAP

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE 2

LEGEND

- af - ARTIFICIAL FILL
- Qya - YOUNG ALLUVIAL FAN DEPOSITS
- Qvop - VERY OLD PARALIC DEPOSITS, UNITS 6, 8, 9, AND 10
- Tsd - SAN DIEGO FORMATION
- Tmv - MISSION VALLEY FORMATION
- Tp - POMERADO CONGLOMERATE

0 1,000 2,000 Feet

N

SOURCE:
GEOLOGIC MAP OF THE SAN DIEGO 30'X60' QUADRANGLE, CALIFORNIA, KENNEDY AND TAN 2008.

\\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20194095_Mercy\20194095_Geo_Merc.mxd

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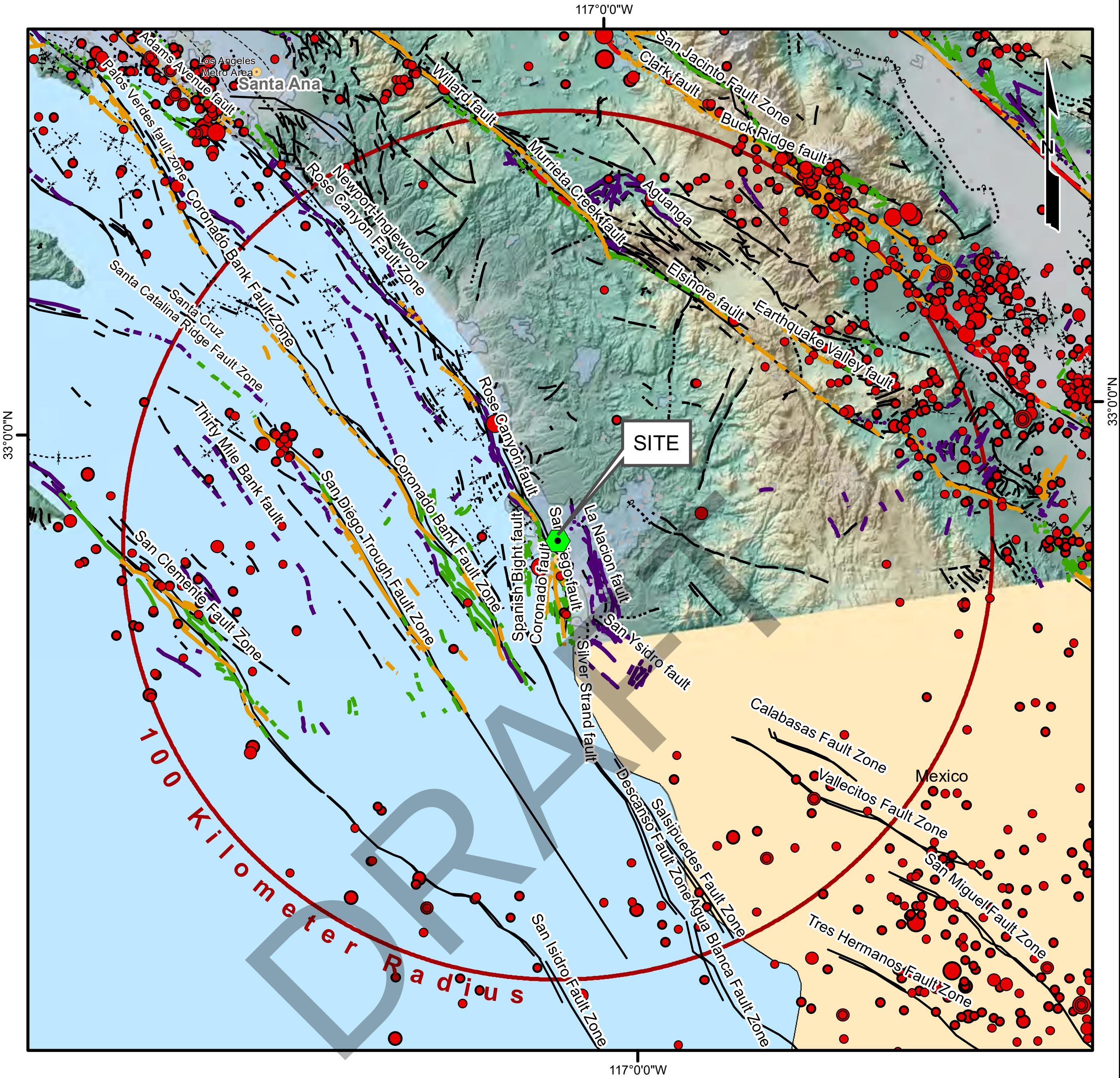
PROJECT NO.	20194095
DRAWN:	5/22/2019
DRAWN BY:	JP
CHECKED BY:	ST
FILE NAME:	20194095_Geo_Merc.mxd

REGIONAL GEOLOGIC MAP

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
3



Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2009)

Historic displacement (< 200 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Holocene displacement (< 11,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Quaternary displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

ANSS Earthquakes

Magnitude
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7.0 - 7.9
- 8.0 - 8.9

35 17.5 0 35
Kilometers

\\sandiego\sandiego-data\GRAPHICS_clients\Scripps\20194095_Mercy\20194095_EQ_Merc.mxd

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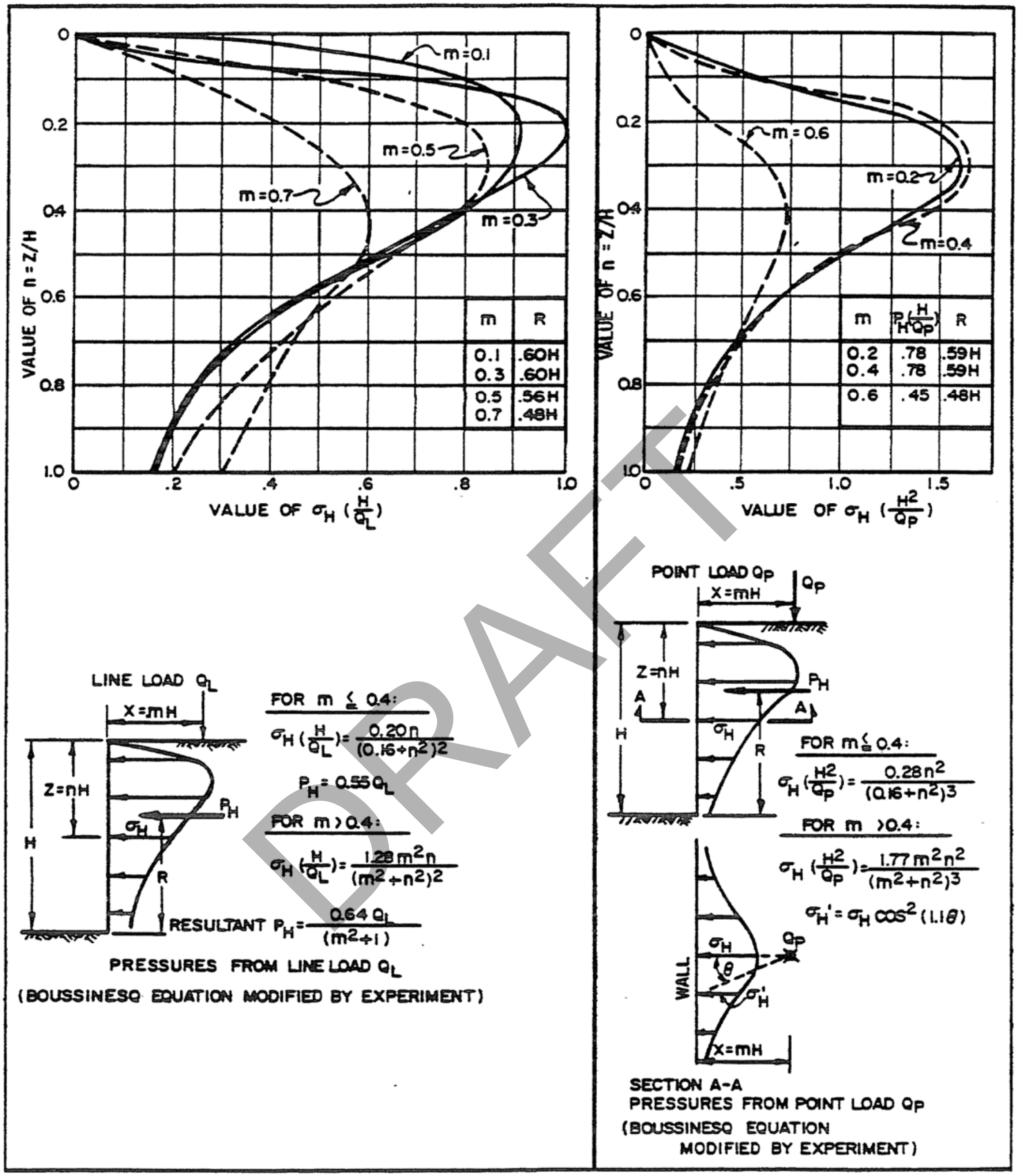
DRAWN:	5/21/2019
DRAWN BY:	JP
CHECKED BY:	ST
FILE NAME:	20194095_EQ_Merc.MXD

REGIONAL FAULT MAP AND EARTHQUAKE EPICENTERS (1800 - MAY 2019)

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE
4



KLEINFELDER Bright People. Right Solutions.	PROJECT NO. 20194095 DRAWN BY: T.Cisney CHECKED BY: K. Crennan DATE: 6/25/2019 REVISED: -	**Horizontal Pressures on Rigid Wall from Surface Load (US Navy, 1986)** MOB WEST CORE AND SHELL SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	FIGURE **5**

APPENDIX A

Boring Logs

APPENDIX A
BORING LOGS

The field exploration consisted of a geologic reconnaissance of the site, drilling three borings to evaluate subsurface conditions, drilling two borings for infiltration testing, and collection of soil samples for laboratory testing. The exploration borings were drilled on May 7, 2019 to depths ranging from approximately 20 to 31 ½ feet and the two borings for infiltration testing were drilled to a depth of 5 feet. Due to access constraints at the site, the drilling contractor, Pacific Drilling of San Diego, used a limited access Fraste PL-G drill rig. The rig was equipped with 6-inch diameter hollow stem augers. The approximate locations of the borings are shown on the Exploration Map, Figure 2.

The Unified Soil Classification System (USCS) was used for our soil descriptions, the chart and a Boring Log Legend are presented as Figures A-1 and A-2, respectively. The Boring Logs for the proposed replacement Hospital Building are presented as Figure A-3 through A-7. The Logs of Borings describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the boring number, drilling date, and the initials of the logger and name of drilling subcontractor. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and intact samples of representative earth materials were obtained from the borings.

In-place soil samples were obtained at the test boring locations using a California penetration sampler or a Standard penetration sampler (SPT) driven a total of 18-inches (or until practical refusal) into the undisturbed soil at the specified sample depth. The sampler was driven using a 140-pound auto-hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count (or N-value) and is recorded on the Boring Logs. Please note that these blow counts have not been adjusted for the effects of overburden pressure, input driving energy, rod length, sampler correction, or boring diameter correction. In addition, to the California and SPT samples, we also obtained bulk samples from the drill cuttings.

An engineer from our office supervised the field operations and logged the borings according to the Unified Soil Classification System (USCS). The boundaries shown between soil types on the logs and cross sections are approximate as the transition between different soil layers may be gradual. Therefore, variations in the subsurface profile should be anticipated throughout the site. The samples retrieved from the borings were sealed, labeled, and transported to our laboratory for further evaluation and testing. Upon completion of the drilling operations, the boreholes were backfilled and the drums were profiled and disposed of, as required by the County of San Diego Department of Environmental Health (DEH).

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- HQ CORE SAMPLE (2.500 in. (63.5 mm.) core diameter)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING
- SONIC CONTINUOUS SAMPLER

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu≥4 and 1≤Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu<4 and/ or 1>Cc>3	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu≥4 and 1≤Cc≤3	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu<4 and/ or 1>Cc>3	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (Half or more of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu≥6 and 1≤Cc≤3	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu<6 and/ or 1>Cc>3	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu≥6 and 1≤Cc≤3	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu<6 and/ or 1>Cc>3	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

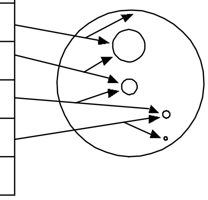
FINE GRAINED SOILS (Half or more of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit 50 or greater)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20194095	GRAPHICS KEY	FIGURE
	DRAWN BY: ST		
	CHECKED BY: SHR	MOB WEST CORE AND SHELL SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	A-1
	DATE: 5/10/2019		
	REVISED: 5/23/2019		

PLOTTED: 05/24/2019 09:06 AM BY: STena
gINT FILE: Klf_gint_master_2019 PROJECT NUMBER: 20194095.001A OFFICE FILTER: SAN DIEGO
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2019.GLB [LEGEND 1 (GRAPHICS KEY) USCS_SOU_CAL]

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

Term of Use	AMOUNT	
	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25 ≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5 ≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1 ≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2 ≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4 ≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

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PROJECT NO.: 20194095	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: ST		
CHECKED BY: SHR	MOB WEST CORE AND SHELL SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	A-2
DATE: 5/10/2019		
REVISED: 5/23/2019		

BORING LOG B-1

Date Begin - End:	5/07/2019	**Drilling Company:**	Pacific Drilling		
Logged By:	S.Tena	**Drill Crew:**	Gerardo, Mike, Toby		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	6 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 292.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			TOPSOIL: (6 INCHES)						7.7						Hand auger to 3.5 feet.
290			**ARTIFICIAL FILL (af)** **Clayey SAND (SC)**: medium-grained, low plasticity, brown (7.5YR 4/2), moist	S1			48"								
285	5		**VERY OLD PARALIC DEPOSITS ($Qvop_9$)** **Silty SAND with Gravel (SM)**: medium-grained, subrounded cobbles, subangular gravel, non-plastic, reddish yellow (7.5YR 6/6), moist			BC=33 33 50/5"	NR								
	10		**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: fine-grained, non-plastic, pale yellow (5Y 7/3), moist			BC=50/6"	NR								Poor recovery likely due to auger pushing down cobble from $Qvop_9$.
280			very dense, iron oxide staining, rock fragments inside sampler	S2		BC=16 18 28	1"								
275	15		**Sandy SILT (ML)**: non-plastic to low plasticity, light gray (5Y 7/1), moist, micaceous	S3		BC=13 22 29	18"				100	52			**DIRECT SHEAR TEST** Cohesion = 500 psf Friction Angle = 40°
270	20		**Silty SAND (SM)**: fine-grained, non-plastic, light gray (5Y 7/1), moist	S4		BC=20 40 50/5"	18"								
265	25														
260	30														

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with bentonite chips on May 07, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20194095	BORING LOG B-1	FIGURE
DRAWN BY: ST		
CHECKED BY: SHR	MOB WEST CORE AND SHELL SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA	A-3
DATE: 5/9/2019		
REVISED: 5/24/2019		PAGE: 1 of 1

BORING LOG B-2

Date Begin - End:	5/07/2019	**Drilling Company:**	Pacific Drilling
Logged By:	S.Tena	**Drill Crew:**	Gerardo, Mike, Toby
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Cloudy	**Auger Diameter:**	6 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

Approximate Ground Surface Elevation (ft.): 293.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
293	0	**TOPSOIL**: (6 inches)											
	0.5	**ARTIFICIAL FILL (af)** **Clayey SAND (SC)**: low plasticity, brown (7.5YR 4/2), moist	S1		54"					38			**Expansion Index=** 36; Rocky at 4 feet.
288	5	**VERY OLD PARALIC DEPOSITS (Qvop$_9$)** **Silty SAND (SM)**: medium-grained, non-plastic, reddish yellow (5YR 6/6), moist, dense, trace of gravel and cobbles	S2	BC=19 22 14	18"		8.5						
283	10	**SAN DIEGO FORMATION (Tsd)** **Silty SAND (SM)**: light brownish gray (2.5Y 6/2), moist -rock fragements		BC=25 50/4"	NR								
	12.5	fine-grained, non-plastic, light gray (5Y 7/1), medium dense, weakly cemented, iron oxide staining	S3	BC=9 12 13	18"								
278	15	very dense	S4	BC=16 25 33	18"		19.7	93.7					
	18.5		S5	BC=16 27 40	18"								

The boring was terminated at approximately 20 ft. below ground surface. The boring was backfilled with auger cuttings on May 07, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

KLEINFELDER
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PROJECT NO.:	20194095
DRAWN BY:	ST
CHECKED BY:	SHR
DATE:	5/9/2019
REVISED:	5/24/2019

BORING LOG B-2

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4077 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE A-4

BORING LOG B-3

Date Begin - End:	5/07/2019	**Drilling Company:**	Pacific Drilling		
Logged By:	S.Tena	**Drill Crew:**	Gerardo, Mike, Toby		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	6 in. O.D.		

Approximate Ground Surface Elevation (ft.): 292.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description (FIELD EXPLORATION)	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol (LABORATORY RESULTS)	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0	**TOPSOIL**: (6 inches)												Hand auger to 3 feet.
290		**ARTIFICIAL FILL (af)** **Sandy Lean CLAY (CL)**: low plasticity, brown (7.5YR 5/4), moist, subrounded cobbles (5"), concrete debris -increase in moisture	S1	Bulk		30"		15.2				30	16	**pH=** 8.1 **Resistivity=** 1900 ohm-cm **Sulfates=** 69 ppm **Chlorides=** 21 ppm
	5	**VERY OLD PARALIC DEPOSITS ($Qvop_9$)** **Silty SAND with Gravel (SM)**: non-plastic, moist -Gravel from 5 to 8 feet.			BC=50/6"	NR								Hard drilling from 5 to 8 feet.
285														
		SAN DIEGO FORMATION (Tsd) **Silty SAND (SM)**: fine-grained, non-plastic to low plasticity, light yellowish brown (2.5Y 6/4), moist												
	10				BC=50/5"	NR								Poor recovery likely due to auger pushing down cobble from $Qvop_9$.
280														
		-rock fragments			BC=50/5"	NR								
		very dense	S2		BC=48 27 27	6"								
	15	non-plastic, light gray (5Y 7/1), dense, iron oxide staining	S3		BC=9 14 16	18"		19.3						
275														
	20	medium-grained, very dense, weakly cemented	S4		BC=19 27 50/5"	18"				100	37			**DIRECT SHEAR TEST** Cohesion = 700 psf Friction Angle = 36°
270														
	25	fine-grained	S5		BC=22 50/3"	9"								
265														
	30		S6		BC=22 32 50/5"	18"								
260														

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with bentonite chips on May 07, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20194095	BORING LOG B-3		FIGURE
DRAWN BY: ST			A-5
CHECKED BY: SHR	MOB WEST CORE AND SHELL SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE SAN DIEGO, CALIFORNIA		
DATE: 5/9/2019			
REVISED: 5/24/2019			PAGE: 1 of 1

Date Begin - End:	5/07/2019	**Drilling Company:**	Pacific Drilling		**BORING LOG PERC-1**
Logged By:	S.Tena	**Drill Crew:**	Gerardo, Mike, Toby		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

Approximate Elevation (feet)	Depth (feet)	Graphical Log	FIELD EXPLORATION: Approximate Ground Surface Elevation (ft.): 292.00 Surface Condition: Grass — Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			TOPSOIL: (6 inches)												Hand auger 4 feet.
290			**ARTIFICIAL FILL (af)** **Sandy Lean CLAY (CL)**: low plasticity, brown (7.5YR 4/3), moist												
			VERY OLD PARALIC DEPOSITS ($Qvop_9$) **Silty SAND with Gravel (SM)**: medium to coarse-grained, subrounded cobbles, subrounded gravel, non-plastic, reddish yellow (7.5YR 6/6), moist	S1				SM			71	14			
	5														
285															
	10														
280															
	15														
275															
	20														
270															
	25														
265															
	30														
260															

The boring was terminated at approximately 5 ft. below ground surface. The boring was not backfilled at time of drilling completion because infiltration testing. The boring was backfilled with auger cuttings on May 08, 2019.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

Date Begin - End:	5/07/2019	**Drilling Company:**	Pacific Drilling		**BORING LOG PERC-2**
Logged By:	S.Tena	**Drill Crew:**	Gerardo, Mike, Toby		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Fraste PL-G	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Auger Diameter:**	8 in. O.D.		

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description (Approximate Ground Surface Elevation (ft.): 292.00; Surface Condition: Grass)	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			TOPSOIL: (6 INCHES)												Hand auger 3 feet.
290			**ARTIFICIAL FILL (af)** **Clayey SAND (SC)**: medium-grained, low plasticity, brown (7.5YR 4/2), moist												
			VERY OLD PARALIC DEPOSITS (Qvop$_9$) **Silty SAND with Gravel (SM)**: medium-grained, subrounded cobbles, subangular gravel, non-plastic, reddish yellow (7.5YR 6/6), moist brown (10YR 5/3)	S1							85	34			
	5														
285			The boring was terminated at approximately 5 ft. below ground surface. The boring was not backfilled at time of drilling completion because infiltration testing. The boring was backfilled with auger cuttings on May 08, 2019.												
	10														
280															
	15														
275															
	20														
270															
	25														
265															
	30														
260															

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

DRAFT

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PROJECT NO.: 20194095	BORING LOG PERC-2	FIGURE
DRAWN BY: ST		
CHECKED BY: SHR	MOB WEST CORE AND SHELL	A-7
DATE: 5/9/2019	SCRIPPS MERCY HOSPITAL 4077 5TH AVENUE	
REVISED: 5/24/2019	SAN DIEGO, CALIFORNIA	PAGE: 1 of 1

APPENDIX B

Laboratory Test Results

APPENDIX B
LABORATORY TEST RESULTS

GENERAL

Laboratory tests were performed on selected, representative samples as an aid in classifying the soils and to evaluate physical properties of the soils, which may affect foundation design and construction procedures. A description of our laboratory testing program is presented below:

CLASSIFICATION

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory excavations in Appendix A.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Natural moisture content and dry density tests were performed on the intact samples collected. Moisture content was evaluated in general accordance with ASTM Test Method D 2216; dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. The values are shown on the boring logs in Appendix A.

SIEVE ANALYSIS

Sieve analyses were performed on four samples and a #200 sieve wash was performed on one sample from the site to evaluate the gradation characteristics of the soil and to aid in its classification. The tests were performed in general accordance with ASTM Test Method D 422. The test results are presented on Figures B-1 through B-4.

ATTEBERG LIMITS TEST

Liquid and plastic limits tests were conducted on one selected sample. The test was performed in general accordance with ASTM D 4318. In addition, the values are shown on the boring logs in Appendix A and Figure B-5.

DIRECT SHEAR TEST

Three direct shear tests were performed on two representative soil samples to evaluate the shear strength of the site soils. The soil samples were tested in a saturated state and subjected to three different normal pressures in general accordance with ASTM Test Method D3080. The test results are presented on Figures B-6 and B-7.

EXPANSION INDEX TEST

One expansion index (EI) test was performed on a representative sample of the near-surface materials obtained during our investigation. The test was performed in accordance with ASTM D4829. The corrected expansion index value for the sample is presented in Table B-1. The test result indicates a low expansion potential when compared to Table B-2 to qualitatively evaluate the expansion potential of the site soils.

Table B-1

Expansion Index Test Results

Boring	Depth (ft)	Soil Type	EI
B-2	0.5-5	Clayey Sand	36
B-5	1-5	Silty Sand	0

Table B-2

ASTM D 4829 Expansion Index and Potential

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

CORROSION TEST

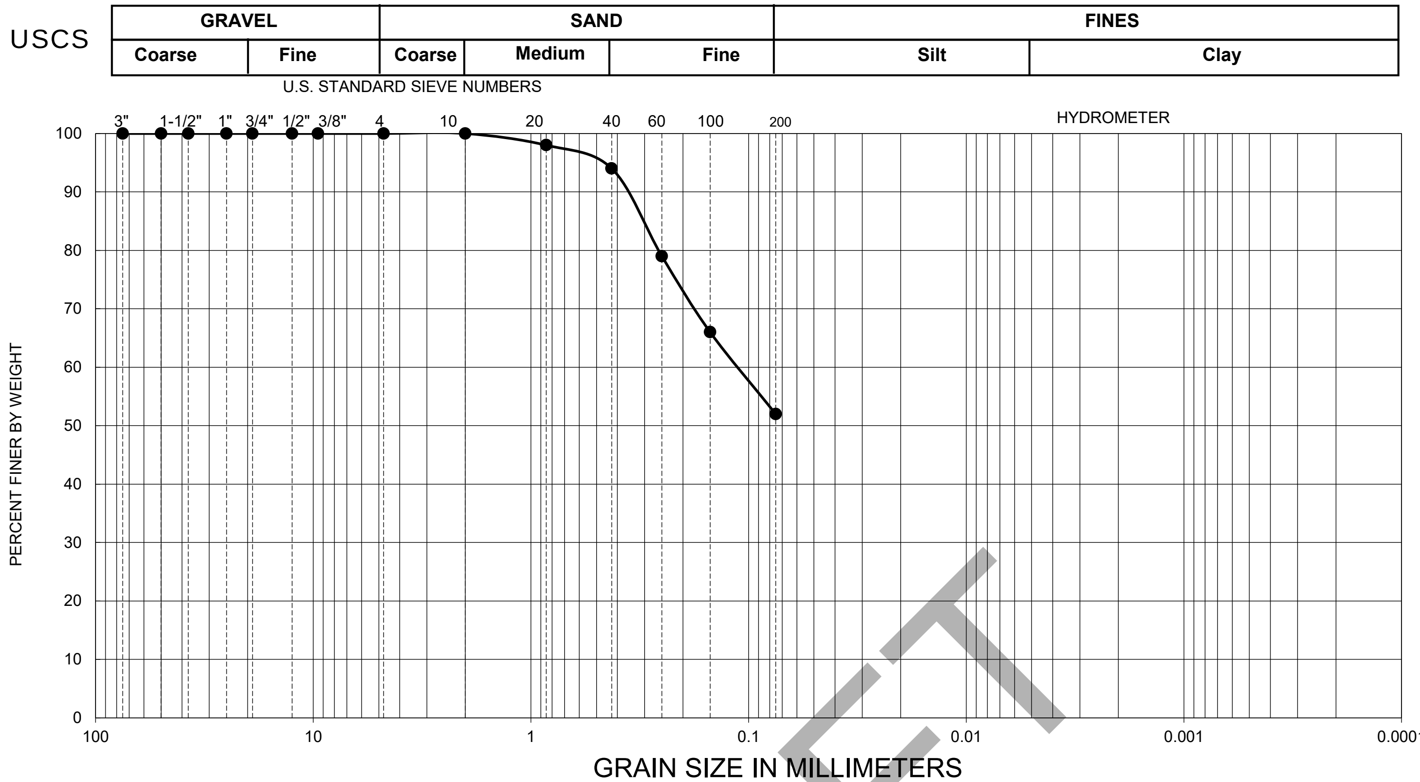
The sulfate and chloride contents and pH of two selected samples were evaluated in general accordance with California Test 417 and California Test 422. Our boring logs and these test results should be reviewed by a qualified corrosion engineer to evaluate the general soil stratigraphy corrosion potential with respect to construction materials and determine whether further testing is warranted. The test results are presented on Table B-3 and Figure B-8.

Table B-3

Corrosion Test Results

Sample	Depth (ft)	pH	Minimum Resistivity (ohm-cm)	Sulfate (%)	Chloride (%)
B-3/S1	0.5-3	8.1	1900	0.007	0.002

Date Tested: 5/16/2019



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B1	S3	16-16.5	52.0	ML

Sample Description	Sandy silt

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	98
	No. 40	0.425 mm	94
	No. 60	0.25 mm	79
	No 100	0.15 mm	66
	No 200	.075 mm	52.0

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

DRAFT

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Checked by:	S.Tena	Tech:	Uly
Project No.	20194095.001A	Date:	22-May-19

GRADATION TEST RESULTS

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

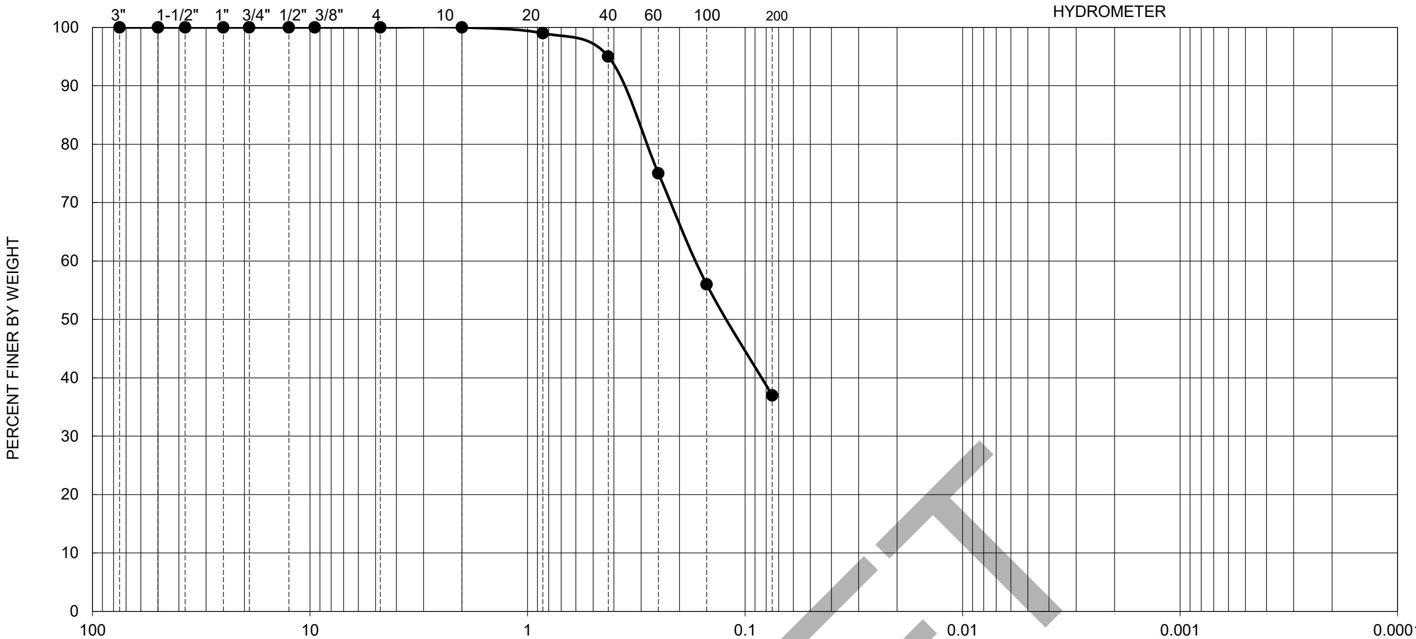
B-1

Date Tested: **5/17/2019**

USCS

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

U.S. STANDARD SIEVE NUMBERS



GRAIN SIZE IN MILLIMETERS

Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
B3	S4	21-21.5	37.0	SM

Sample Description	Silty sand

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	100
	1/2"	12.5 mm	100
	3/8"	9.5 mm	100
	No. 4	4.75 mm	100
	No. 10	2.0 mm	100
	No. 20	0.85 mm	99
	No. 40	0.425 mm	95
	No. 60	0.25 mm	75
	No 100	0.15 mm	56
	No 200	.075 mm	37.0

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	S.Tena	Tech:	Uly
Project No.	20194095.001A	Date:	22-May-19

GRADATION TEST RESULTS

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-2

Date Tested: **5/16/2019**

USCS	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
PERC-1	S1	3-5	13.6	SM

Sample Description	Silty sand with gravel

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	89
	3/4"	19 mm	83
	1/2"	12.5 mm	79
	3/8"	9.5 mm	75
	No. 4	4.75 mm	71
	No. 10	2.0 mm	63
	No. 20	0.85 mm	45
	No. 40	0.425 mm	34
	No. 60	0.25 mm	23
	No 100	0.15 mm	17
	No 200	.075 mm	13.6

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	S.Tena	Tech:	Uly
Project No.	20194095.001A	Date:	22-May-19

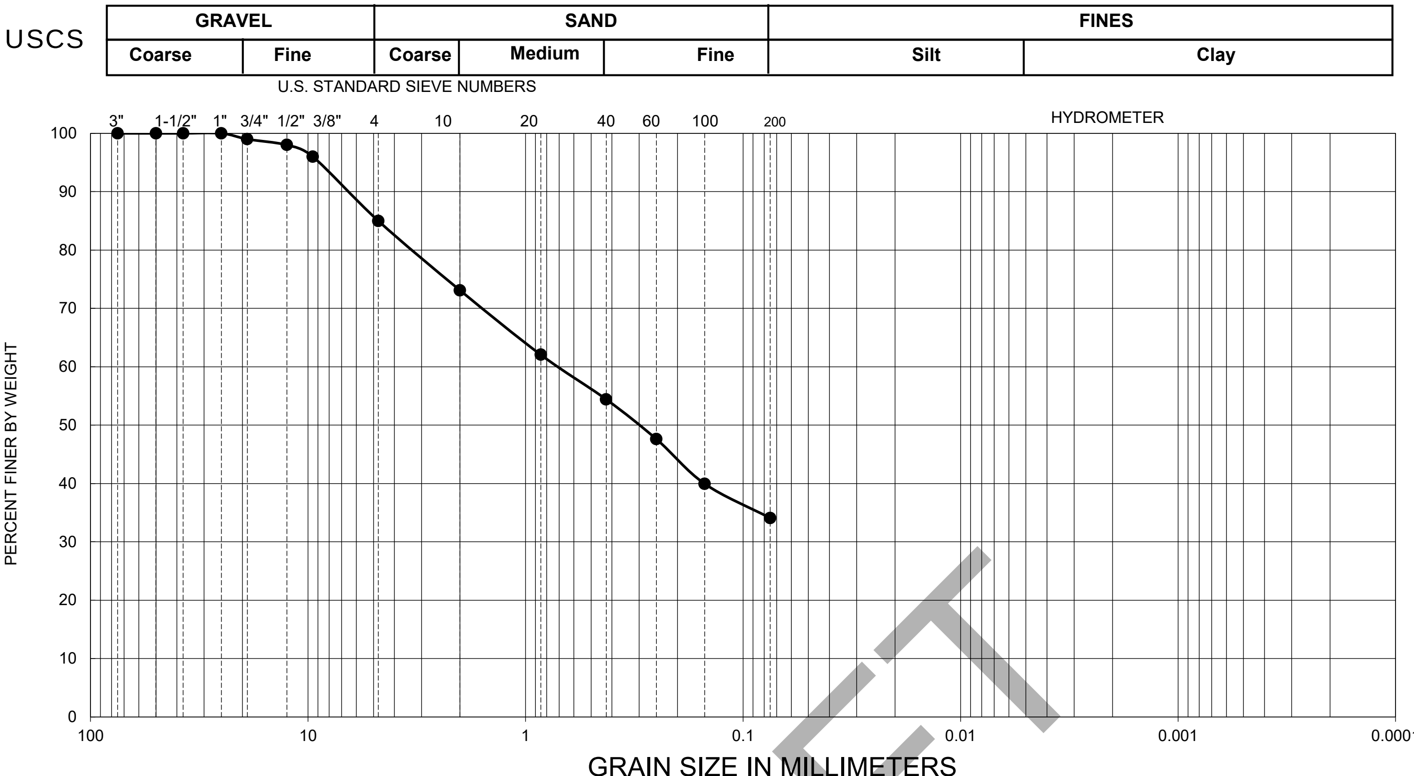
GRADATION TEST RESULTS

MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-3

Date Tested: 5/16/2019



Boring No.	Sample No.	Depth (ft)	Passing 200 (%)	USCS Classification
PERC-2	S1	3-5	34.1	SM

Sample Description	Silty sand with gravel

	Sieve Size		% Passing
Sieve Analysis	3"	75 mm	100
	2"	50 mm	100
	1.5"	37.5 mm	100
	1"	25 mm	100
	3/4"	19 mm	99
	1/2"	12.5 mm	98
	3/8"	9.5 mm	96
	No. 4	4.75 mm	85
	No. 10	2.0 mm	73
	No. 20	0.85 mm	62
	No. 40	0.425 mm	54
	No. 60	0.25 mm	48
	No 100	0.15 mm	40
	No 200	.075 mm	34.1

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

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Checked by:	S.Tena	Tech:	Uly
Project No.	20194095.001A	Date:	22-May-19

GRADATION TEST RESULTS

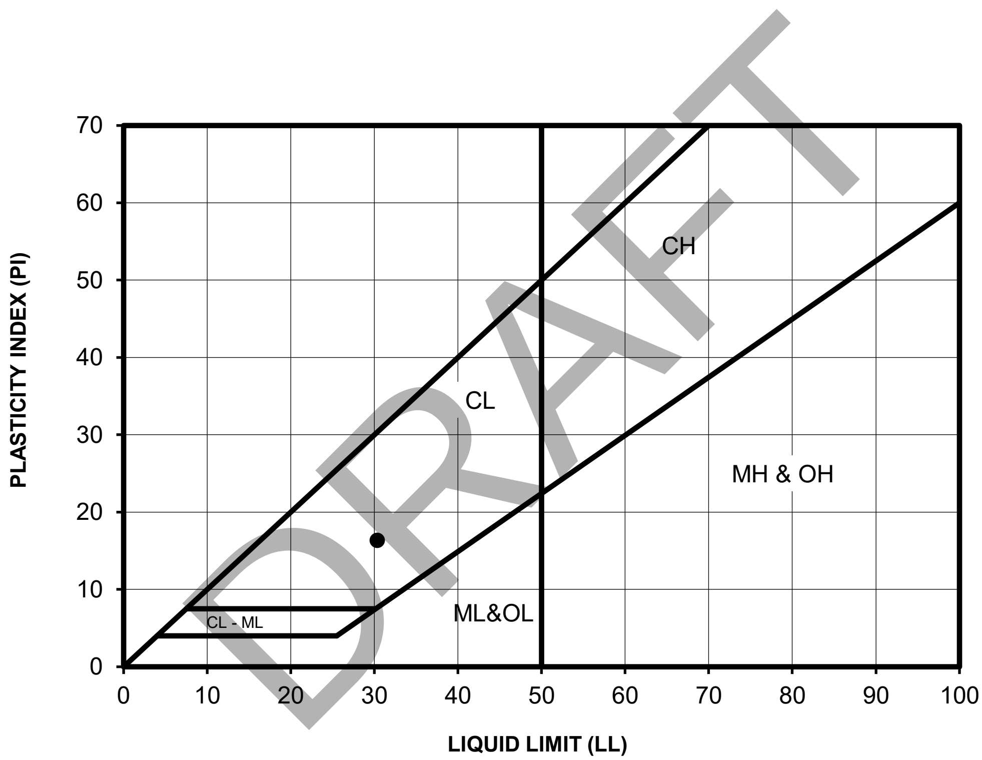
MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-4

Date Tested : 5/17/2019

SYMBOL	SAMPLE NAME	DEPTH (ft)	LL	PL	PI	USCS CLASSIFICATION (Minus No. 40 Sieve Fraction)	USCS (Entire Sample)
●	B3-S1	0.5-3	30	14	16	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Limitations: Pursuant to applicable codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specification were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided. This report may not be reproduced, except in full, without written approval of Kleinfelder.

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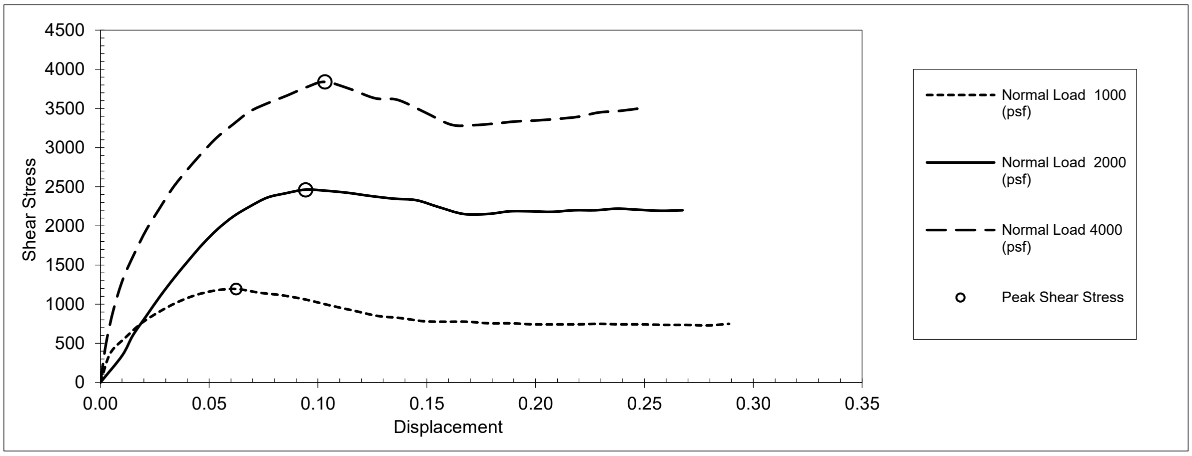
ATTERBERG LIMITS TEST RESULTS

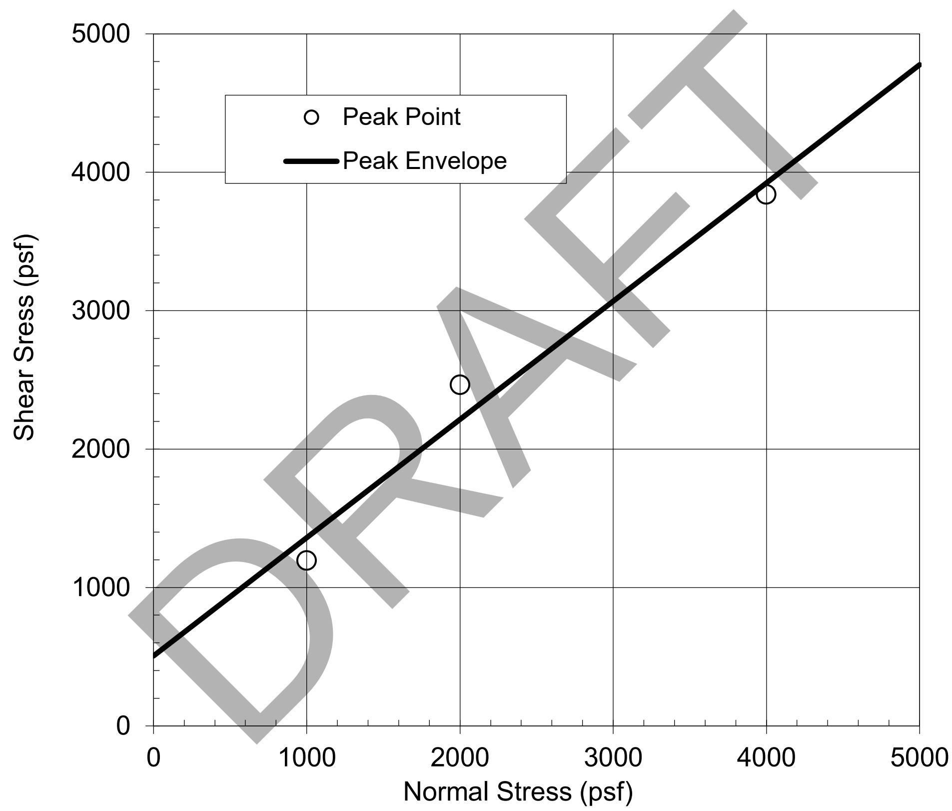
MOB WEST CORE AND SHELL
SCRIPPS MERCY HOSPITAL
4407 5TH AVENUE
SAN DIEGO, CALIFORNIA

FIGURE

B-5

Checked by	S.Tena	TECH Uly P.
PROJECT NO: 20194095.001A		22-May-19





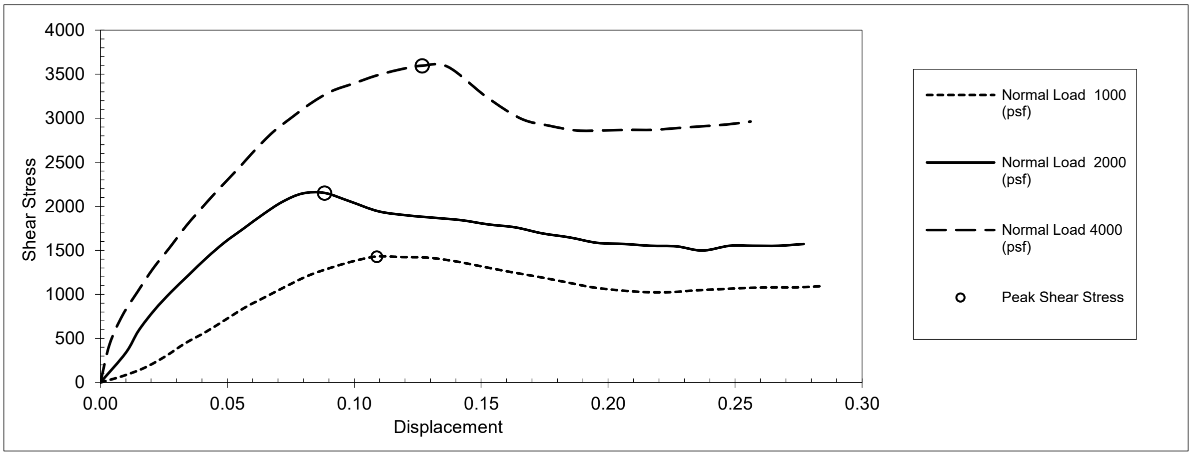
Strain Rate = 0.0118 inch/min

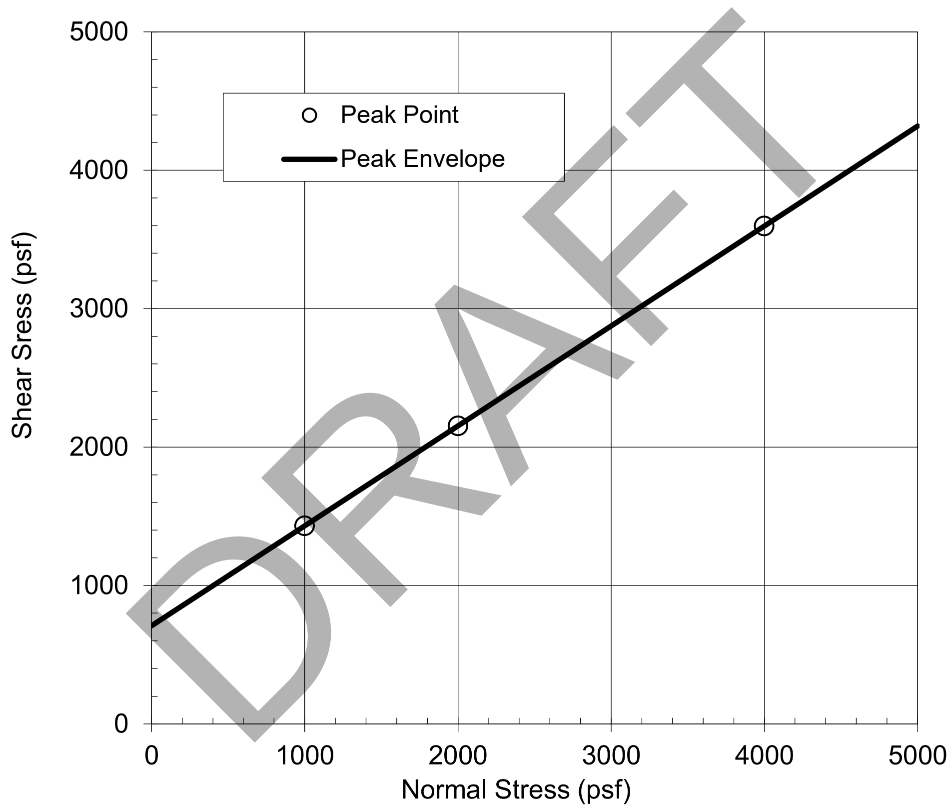
Date Tested: 5/14/2019

Boring No.	Sample No.	Depth	UCSC	Interpreted Shear Strength: Peak Cohesion (psf)	Interpreted Shear Strength: Peak Friction Angle (deg)		
B1	S3	16'-16.5'	ML	506	40.5		

Sample description: sandy silt

KLEINFELDER Bright People. Right Solutions.		Direct Shear Test Results (ASTM D 3080)	Figure
Checked E S.Tena	Tech : Uly P.	MOB WEST CORE AND SHELL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA	B-6
Project # 20194095.001A	22-May-19		





Strain Rate = 0.0118 inch/min
Date Tested: 5/15/2019

				Interpreted Shear Strength			
				Peak			
Boring No.	Sample No.	Depth	UCSC	Cohesion (psf)	Friction Angle (deg)		
B3	S4	21'-21.5'	SM	709	35.8		

Sample description: silty sand

KLEINFELDER Bright People. Right Solutions.		**Direct Shear Test Results (ASTM D 3080)**	Figure
Checked E S.Tena	Tech : Uly P.	**MOB WEST CORE AND SHELL SCRIPPS MERCY HOSPITAL 4407 5TH AVENUE SAN DIEGO, CALIFORNIA**	**B-7**
Project # 20194095.001A	22-May-19		

L A B O R A T O R Y   R E P O R T

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C L A R K S O N L A B O R A T O R Y A N D S U P P L Y I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L A N D C O N S U L T I N G C H E M I S T S

Date: May 20, 2019
Purchase Order Number: 20194095.001A
Sales Order Number: 44360
Account Number: KLE

To:

Kleinfelder Inc.
550 West C Street Ste 1200
San Diego, CA 92101
Attention: Uly Panuncialman

Laboratory Number: SO7331   Customers Phone: 619-831-4600
Fax: 619-831-4619

Sample Designation:

One soil sample received on 05/16/19 at 9:50am,
from MOB Scripps Project# 20194095.001A
marked as B-3,5-1,.5'-3',Bulk.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.1

Water Added (ml)	Resistivity (ohm-cm)
10	4800
5	3000
5	1900
5	1900
5	2000
5	2100

40 years to perforation for a 16 gauge metal culvert.
52 years to perforation for a 14 gauge metal culvert.
72 years to perforation for a 12 gauge metal culvert.
91 years to perforation for a 10 gauge metal culvert.
111 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.007% (69ppm)

Water Soluble Chloride Calif. Test 422   0.002% (21ppm)

Rosa Bernal
RMB/ilv

APPENDIX C

Infiltration Study

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
		Preliminary Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ⦿ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ○ Yes; continue to Step 1F. ○ No; conduct appropriate number of tests.	
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ○ Yes; continue to Step 1G. ○ No; select appropriate factor of safety.	
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ○ Yes; answer "Yes" to Criteria 1 Result. ○ No; answer "No" to Criteria 1 Result.	
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ○ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ⦿ No; full infiltration is not required. Skip to Part 1 Result.	
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report. Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.02 inches per hour.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	◯ Yes	◯ No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	◯ Yes	◯ No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	◯ Yes	◯ No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	◯ Yes	◯ No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	◯ Yes	◯ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○Yes	○ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○Yes	○ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○Yes	○ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	○Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	◯ Yes	◯ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	◯ Yes	◯ No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 1 Result – Full Infiltration Geotechnical Screening [4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	◯ Full infiltration Condition ⦿ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:

Criteria 3 : Infiltration Rate Screening	
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ◉ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ◉ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ◉ No: Skip to Part 2 Result.

Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).

Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.02 inches per hour.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	○ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	○ Yes	○ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	○ Yes	○ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	○ Yes	○ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	○ Yes	○ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.		
Part 2 – Partial Infiltration Geotechnical Screening Result[5]		**Result**
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.		○ Partial Infiltration Condition ◉ No Infiltration Condition

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]	Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
	Preliminary Phase
Criteria 1: Infiltration Rate Screening	

1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ◉ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ○ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ○ No; full infiltration is not required. Answer "No" to Criteria 1 Result.
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.

[1] Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[2] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[3] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ◯ Yes; continue to Step 1F. ◯ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ◯ Yes; continue to Step 1G. ◯ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ◯ Yes; answer "Yes" to Criteria 1 Result. ◯ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ◯ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ◉ No; full infiltration is not required. Skip to Part 1 Result.

Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.

Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.01 inches per hour.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions				Worksheet C.4-1: Form I-8A[2]
Criteria 2: Geologic/Geotechnical Screening				
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.			
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	Yes	No	
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	Yes	No	
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	Yes	No	
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.			
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	Yes	No	
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	Yes	No	

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	◯Yes	◯ No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	◯Yes	◯ No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	◯Yes	◯ No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	◯Yes	◯ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	○ Yes	○ No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	○ Yes	○ No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 1 Result – Full Infiltration Geotechnical Screening[4]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	○ Full infiltration Condition ◉ Complete Part 2

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:
Criteria 3 : Infiltration Rate Screening	

3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ○ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ○ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ⊙ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ○ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ⊙ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ○ Yes; Continue to Criteria 4. ⊙ No: Skip to Part 2 Result.

Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).

Borehole percolation testing was performed to evaluate the infiltration rate at the site. The reliable infiltration rate was 0.01 inches per hour.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	○ Yes	○ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	○ Yes	○ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	○ Yes	○ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	○ Yes	○ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	○ Yes	○ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	○ Yes	○ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	◯Yes	◯No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	◯Yes	◯No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	◯Yes	◯No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	◯Yes	◯No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	◯Yes	◯No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits.	

Part 2 – Partial Infiltration Geotechnical Screening Result[5]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	○ Partial Infiltration Condition ◉ No Infiltration Condition

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

Percolation Test Data Sheet

Project: MOB Scripps Mercy Hospital

Tested By: Salvador Tena

Date: 5/8/2019

Project No: 20194095.001A

Checked By:

Borehole ID: **PERC-1**

Depth of Borehole: 5 feet

Diameter of Borehole: 8 inches

USCS Soil Classification: SM

PVC Pipe Hieght above Surface 0.3 ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	6:43	7:13	30	0.61	0.66	0.05	50.00
2	7:13	7:43	30	0.66	0.71	0.05	50.00
3	7:43	8:13	30	0.71	0.76	0.05	50.00
4	8:13	8:43	30	0.76	0.81	0.05	50.00
5	8:43	9:13	30	0.81	0.85	0.04	62.50
6	9:13	9:43	30	0.85	0.90	0.05	50.00
7	9:43	10:13	30	0.90	0.95	0.05	50.00
8	10:13	10:43	30	0.95	1.00	0.05	50.00
9	10:43	11:13	30	1.00	1.03	0.03	83.33
10	11:13	11:43	30	1.03	1.07	0.04	62.50
11	11:43	12:13	30	1.07	1.10	0.03	83.33
12	12:13	12:43	30	1.10	1.14	0.04	**62.50**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

Reference:

H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H \; 60 \; r}{\Delta t \; (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	46.80	inches
Hf	46.32	inches
ΔH	0.48	inches
Havg	46.56	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.04**	in/hr

Percolation Test Data Sheet

Project: MOB Scripps Mercy Hospital
Tested By: Salvador Tena
Date: 5/8/2019
Project No: 20194095.001A
Checked By:

Borehole ID: **PERC-2**
Depth of Borehole: 5 feet
Diameter of Borehole: 8 inches
USCS Soil Classification: SM
PVC Pipe Hieght above Surface 0.3 ft

Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to water (feet)	Final Depth to Water (feet)	Change in Water Level (feet)	Percolation Rate (min./in.)
1	6:47	7:17	30	0.74	0.75	0.01	250.00
2	7:17	7:47	30	0.75	0.77	0.02	125.00
3	7:47	8:17	30	0.77	0.78	0.01	250.00
4	8:17	8:47	30	0.78	0.78	0	NA
5	8:47	9:17	30	0.78	0.78	0	NA
6	9:17	9:47	30	0.78	0.79	0.01	250.00
7	9:47	10:17	30	0.79	0.80	0.01	250.00
8	10:17	10:47	30	0.80	0.80	0	NA
9	10:47	11:17	30	0.80	0.81	0.01	250.00
10	11:17	11:47	30	0.81	0.82	0.01	250.00
11	11:47	12:17	30	0.82	0.82	0	NA
12	12:17	12:47	30	0.82	0.83	0.01	**250.00**

Porchet Method Conversion - to convert percolation rate to tested infiltration rate

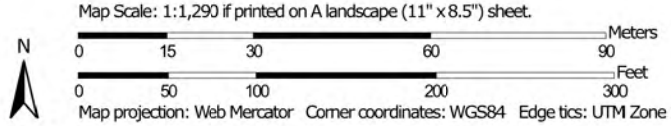
Reference:
H.P. Ritzema, 1994, "Drainage Principles and Applications", International Institute for Land Reclamation and Improvement, Publication 16, 2nd revised edition, Wageningen, The Netherlands

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t\ (r + 2Havg)}$$

Ho = Original height of water column in hole (inches)
Hf = Final height of water column in hole (inches)
ΔH = Change in head over the time interval (inches)
Havg = Average head over the time interval (inches)
Δt = Time interval (minutes)
r = Effective radius of test hole (inches)
It = Tested infiltration rate (inch/hour)

Conversion Parameters (for 8 inch hole)		
Ho	50.16	inches
Hf	50.04	inches
ΔH	0.12	inches
Havg	50.10	inches
Δt	30.00	minutes
r	4.00	inches
It	**0.01**	in/hr

Map Scale: 1:1,290 if printed on A landscape (11" x 8.5") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N WGS84

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features

- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 13, Sep 12, 2018

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 7, 2014—Jan 4, 2015

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
TeF	Terrace escarpments	1.2	13.9%
Ur	Urban land	7.4	86.1%
Totals for Area of Interest		**8.6**	**100.0%**

Hydrologic Soil Group and Surface Runoff

This table gives estimates of various soil water features. The estimates are used in land use planning that involves engineering considerations.

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The four hydrologic soil groups are:

Group 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.

Group 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.

Group 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.

Group 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.

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.

Surface runoff refers to the loss of water from an area by flow over the land surface. Surface runoff classes are based on slope, climate, and vegetative cover. The concept indicates relative runoff for very specific conditions. It is assumed that the surface of the soil is bare and that the retention of surface water resulting from irregularities in the ground surface is minimal. The classes are negligible, very low, low, medium, high, and very high.

Report—Hydrologic Soil Group and Surface Runoff

Absence of an entry indicates that the data were not estimated. The dash indicates no documented presence.

Hydrologic Soil Group and Surface Runoff–San Diego County Area, California			
Map symbol and soil name	**Pct. of map unit**	**Surface Runoff**	**Hydrologic Soil Group**
TeF—Terrace escarpments			
Terrace escarpments	100	Very high	—

Hydrologic Soil Group and Surface Runoff–San Diego County Area, California			
Map symbol and soil name	**Pct. of map unit**	**Surface Runoff**	**Hydrologic Soil Group**
Ur—Urban land			
Urban land	100	Very high	—

Data Source Information

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 13, Sep 12, 2018

Engineering Properties

This table gives the engineering classifications and the range of engineering properties for the layers of each soil in the survey area.

Hydrologic soil group is a group of soils having similar runoff potential under similar storm and cover conditions. The criteria for determining Hydrologic soil group is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Listing HSGs by soil map unit component and not by soil series is a new concept for the engineers. Past engineering references contained lists of HSGs by soil series. Soil series are continually being defined and redefined, and the list of soil series names changes so frequently as to make the task of maintaining a single national list virtually impossible. Therefore, the criteria is now used to calculate the HSG using the component soil properties and no such national series lists will be maintained. All such references are obsolete and their use should be discontinued. Soil properties that influence runoff potential are those that influence the minimum rate of infiltration for a bare soil after prolonged wetting and when not frozen. These properties are depth to a seasonal high water table, saturated hydraulic conductivity after prolonged wetting, and depth to a layer with a very slow water transmission rate. Changes in soil properties caused by land management or climate changes also cause the hydrologic soil group to change. The influence of ground cover is treated independently. There are four hydrologic soil groups, A, B, C, and D, and three dual groups, A/D, B/D, and C/D. In the dual groups, the first letter is for drained areas and the second letter is for undrained areas.

The four hydrologic soil groups are described in the following paragraphs:

Group 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.

Group 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.

Group 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.

Group 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.

Depth to the upper and lower boundaries of each layer is indicated.

Texture is given in the standard terms used by the U.S. Department of Agriculture. These terms are defined according to percentages of sand, silt, and clay in the fraction of the soil that is less than 2 millimeters in diameter. "Loam," for example, is soil that is 7 to 27 percent clay, 28 to 50 percent silt, and less than 52 percent sand. If the content of particles coarser than sand is 15 percent or more, an appropriate modifier is added, for example, "gravelly."

Classification of the soils is determined according to the Unified soil classification system (ASTM, 2005) and the system adopted by the American Association of State Highway and Transportation Officials (AASHTO, 2004).

The Unified system classifies soils according to properties that affect their use as construction material. Soils are classified according to particle-size distribution of the fraction less than 3 inches in diameter and according to plasticity index, liquid limit, and organic matter content. Sandy and gravelly soils are identified as GW, GP, GM, GC, SW, SP, SM, and SC; silty and clayey soils as ML, CL, OL, MH, CH, and OH; and highly organic soils as PT. Soils exhibiting engineering properties of two groups can have a dual classification, for example, CL-ML.

The AASHTO system classifies soils according to those properties that affect roadway construction and maintenance. In this system, the fraction of a mineral soil that is less than 3 inches in diameter is classified in one of seven groups from A-1 through A-7 on the basis of particle-size distribution, liquid limit, and plasticity index. Soils in group A-1 are coarse grained and low in content of fines (silt and clay). At the other extreme, soils in group A-7 are fine grained. Highly organic soils are classified in group A-8 on the basis of visual inspection.

If laboratory data are available, the A-1, A-2, and A-7 groups are further classified as A-1-a, A-1-b, A-2-4, A-2-5, A-2-6, A-2-7, A-7-5, or A-7-6. As an additional refinement, the suitability of a soil as subgrade material can be indicated by a group index number. Group index numbers range from 0 for the best subgrade material to 20 or higher for the poorest.

Percentage of rock fragments larger than 10 inches in diameter and 3 to 10 inches in diameter are indicated as a percentage of the total soil on a dry-weight basis. The percentages are estimates determined mainly by converting volume percentage in the field to weight percentage. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Percentage (of soil particles) passing designated sieves is the percentage of the soil fraction less than 3 inches in diameter based on an ovendry weight. The sieves, numbers 4, 10, 40, and 200 (USA Standard Series), have openings of 4.76, 2.00, 0.420, and 0.074 millimeters, respectively. Estimates are based on laboratory tests of soils sampled in the survey area and in nearby areas and on estimates made in the field. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Liquid limit and *plasticity index* (Atterberg limits) indicate the plasticity characteristics of a soil. The estimates are based on test data from the survey area or from nearby areas and on field examination. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

References:

American Association of State Highway and Transportation Officials (AASHTO). 2004. Standard specifications for transportation materials and methods of sampling and testing. 24th edition.

American Society for Testing and Materials (ASTM). 2005. Standard classification of soils for engineering purposes. ASTM Standard D2487-00.

Report—Engineering Properties

Absence of an entry indicates that the data were not estimated. The asterisk '*' denotes the representative texture; other possible textures follow the dash. The criteria for determining the hydrologic soil group for individual soil components is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Engineering Properties–San Diego County Area, California														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
TeF—Terrace escarpments														
Terrace escarpments	100		0-60	Variable	—	—	—	—	—	—	—	—	—	—
Ur—Urban land														
Urban land	100		0-6	Variable	—	—	—	—	—	—	—	—	—	—

Data Source Information

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 13, Sep 12, 2018

APPENDIX D

Suggested Guidelines for Earthwork Construction

APPENDIX D
SUGGESTED GUIDELINES FOR EARTHWORK CONSTRUCTION

1.0 GENERAL

1.1 **Scope** - The work done under these specifications shall include clearing, stripping, removal of unsuitable material, excavation, preparation of natural soils, placement and compaction of onsite and imported fill material and placement and compaction of pavement materials.

1.2 **Contractor's Responsibility** - The Contractor shall attentively examine the site in such a manner that he can correlate existing surface conditions with those presented in the geotechnical study report. He shall satisfy himself that the quality and quantity of exposed materials and subsurface soil or rock deposits have been satisfactorily represented by the Geotechnical Engineer's report and project drawings. Any discrepancy of prior knowledge to the Contractor to that is revealed through his study shall be made known to the Owner. It is the Contractor's responsibility to review the report prior to construction. The selection of equipment for use on the project and the order of the work shall similarly be the Contractor's responsibility. The Contractor shall be responsible for providing equipment capable of completing the requirements included in the following sections.

1.3 **Geotechnical Engineer** - The work covered by these specifications shall be observed and tested by Kleinfelder, the Geotechnical Engineer, who shall be hired by the Owner. The Geotechnical Engineer will be present during the site preparation and grading to observe the work and to perform the tests necessary to evaluate material quality and compaction. The Geotechnical Engineer shall submit a report to the Owner, including a tabulation of tests performed. The costs of re-testing unsuitable work installed by the Contractors shall be deducted by the Owner from the payments to the Contractor.

1.4 **Standard Specifications** - Where referred to in these specifications, "Standard Specifications" shall mean the State of California Standard Specifications for Public Works Construction, with Regional Supplement Amendments for San Diego County, 2000 Edition.

1.5 **Compaction Test Method** - Where referred to herein, relative compaction shall mean the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D 1557 Compaction Test Procedure. Optimum moisture content shall mean the moisture content at the maximum dry density determined above.

2.0 SITE PREPARATION

2.1 **Clearing** - Areas to be graded shall be cleared and grubbed of all vegetation and debris. The Contractor shall remove these materials from the site.

2.2 **Stripping** - Surface soils containing roots and organic matter shall be stripped from areas to be graded and stockpiled or discarded as directed by the Owner. In general, the depth of stripping of the topsoil will be approximately 3 inches. Deeper stripping, where required to remove weak soils or accumulations of organic matter, shall be performed when determined necessary by the Geotechnical Engineer. Stripped material shall be removed from the site or stockpiled at a location designated by the Owner.

2.3 **Removal of Existing Fill** - Existing fill soils, trash and debris in the areas to be graded shall be removed prior to the placing of any compacted fill. Portions of any existing fills that are suitable for use in new compacted fill may be stockpiled for future use. All organic materials, topsoil, expansive soils, oversized rock or other unsuitable material shall be removed from the site by the Contractor or disposed of at a location onsite, if so designated by the Owner.

2.4 **Ground Surface** - The ground surface exposed by stripping shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction and compacted as required for compacted fill. Ground surface preparation shall be approved by the Geotechnical Engineer prior to placing fill.

3.0 EXCAVATION

3.1 **General** - Excavations shall be made to the lines and grades indicated on the plans. The data presented in the Geotechnical Engineer's report is for information only and the Contractor shall make his own interpretation with regard to the methods and equipment necessary to perform the excavation and to obtain material suitable for fill.

3.2 **Materials** - Soils which are removed and are unsuitable for fill shall be placed in nonstructural areas of the project, or in deeper fills at locations designated by the Geotechnical Engineer.

All oversize rocks and boulders that cannot be incorporated in the work by placing in embankments or used as rip-rap or for other purposes shall be removed from the site by the Contractor.

3.3 **Treatment of Exposed Surface** - The ground surface exposed by excavation shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction and compacted as required for compacted fill. Compaction shall be approved by the Geotechnical Engineer prior to placing fill.

3.4 **Rock Excavation** - Where solid rock is encountered in areas to be excavated, it shall be loosened and broken up so that no solid ribs, projections or large fragments will be within 6 inches of the surface of the final subgrade.

4.0 COMPACTED FILL

4.1 **Materials** - Fill material shall consist of suitable onsite or imported soil. All materials used for structural fill shall be reasonably free of organic material, have an Expansion Index of 30 or less, 100% passing the 3 inch sieve and less than 30 percent passing the #200 sieve.

4.2 **Placement** - All fill materials shall be placed in layers of 8 inches or less in loose thickness and uniformly moisture conditioned. Each lift should then be compacted with a sheepsfoot roller or other approved compaction equipment to at least 90 percent relative compaction in areas under structures, utilities, roadways and parking areas. No fill material shall be placed, spread or rolled while it is frozen or thawing, or during unfavorable weather conditions.

4.3 **Compaction Equipment** - The Contractor shall provide and use sufficient equipment of a type and weight suitable for the conditions encountered in the field. The equipment shall be capable of obtaining the required compaction in all areas.

4.4 **Recompaction** - When, in the judgment of the Geotechnical Engineer, sufficient compactive effort has not been used, or where the field density tests indicate that the required compaction or moisture content has not been obtained, or if pumping or other indications of instability are noted, the fill shall be reworked and recompacted as needed to obtain a stable fill at the required density and moisture content before additional fill is placed.

4.5 **Responsibility** - The Contractor shall be responsible for the maintenance and protection of all embankments and fills made during the contract period and shall bear the expense of replacing any portion which has become displaced due to carelessness, negligent work or failure to take proper precautions.

5.0 UTILITY TRENCH BEDDING AND BACKFILL

5.1 **Material** - Pipe bedding shall be defined as all material within 4 inches of the perimeter and 12 inches over the top of the pipe. Material for use as bedding shall be clean sand, gravel, crushed aggregate or native free-draining material, having a Sand Equivalent of not less than 30.

Backfill should be classified as all material within the remainder of the trench. Backfill shall meet the requirements set forth in Section 4.0 for compacted fill.

5.2 **Placement and Compaction** - Pipe bedding shall be placed in layers not exceeding 8 inches in loose thickness, conditioned to the proper moisture content for compaction and compacted to at least 90 percent relative compaction. All other trench backfill shall be placed and compacted in accordance with Section 306-1.3.2 of the Standard Specifications for Mechanically Compacted Backfill. Backfill shall be compacted as required for adjacent fill. If not specified, backfill shall be compacted to at least 90 percent relative compaction in areas under structures, utilities, roadways, parking areas and concrete flatwork.

6.0 SUBSURFACE DRAINAGE

6.1 **General** - Subsurface drainage shall be constructed as shown on the plans. Drainage pipe shall meet the requirements set forth in the Standard Specifications.

6.2 **Materials** - Permeable drain rock used for subdrainage shall meet the following gradation requirements:

Sieve Size	Percentage Passing
3"	100
1-1/2"	90 - 100
3/4"	50 - 80
No. 4	24 - 40
No. 100	0-4
No. 200	0 - 2

6.3 **Geotextile Fabric** - Filter fabric shall be placed between the permeable drain rock and native soils. Filter cloth shall have an equivalent opening size greater than the No. 100 sieve and a grab strength not less than 100 pounds. Samples of filter fabric shall be submitted to the Geotechnical Engineer for approval before the material is brought to the site.

6.3 **Placement and Compaction** - Drain rock shall be placed in layers not exceeding 8 inches in loose thickness and compacted as required for adjacent fill, but in no case, to be less than 85 percent relative compaction. Placement of geotextile fabric shall be in accordance with the manufacturer's specifications and shall be checked by the Geotechnical Engineer.

7.0 AGGREGATE BASE BENEATH CONCRETE SLABS

7.1 **Materials** - Aggregate base beneath concrete slabs shall consist of clean free-draining sand, gravel or crushed rock conforming to the following gradation requirements:

Sieve Size	Percent Passing
1"	100
3/8"	30 – 100
No. 20	0 – 10

7.2 **Placement** - Aggregate base shall be compacted and kept moist until placement of concrete. Compaction shall be by suitable vibrating compactors. Aggregate base shall be placed in layers not exceeding 8 inches in loose thickness. Each layer shall be compacted by at least four passes of the compaction equipment or until 95 percent relative compaction has been obtained.

APPENDIX E

Geotechnical Business Council Insert

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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