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

Bella Mar Apartments

[Insert Permit Application Number]

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

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

Engineer of Work:



Bryan D Smith, RCE 75822 Exp. 06/30/20

Provide Wet Signature and Stamp Above Line

Prepared For:

Red Tail Acquisitions, LLC

2082 Michelson Drive, 4th Floor

Irvine, CA 92612

(949) 433-5610

Prepared By:

FUSCOE ENGINEERING

Fuscoe Engineering, Inc.

6390 Greenwich Dr., Suite 170

San Diego, CA 92122

858-554-1500

Date:

November 25, 2020

Approved by: City of San Diego Date

The City of SAN DIEGO

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Project Name: Bella Mar Apartments

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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:
Permit Application

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

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



Engineer of Work's Signature

75822 — PE#

06/30/20 — Expiration Date

Bryan Smith

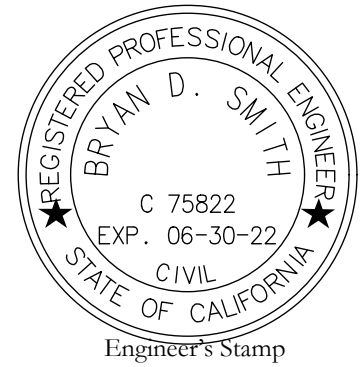
Print Name

Fuscoe Engineering

Company

11/25/20

Date



Engineer's Stamp

Project Name: Bella Mar Apartments

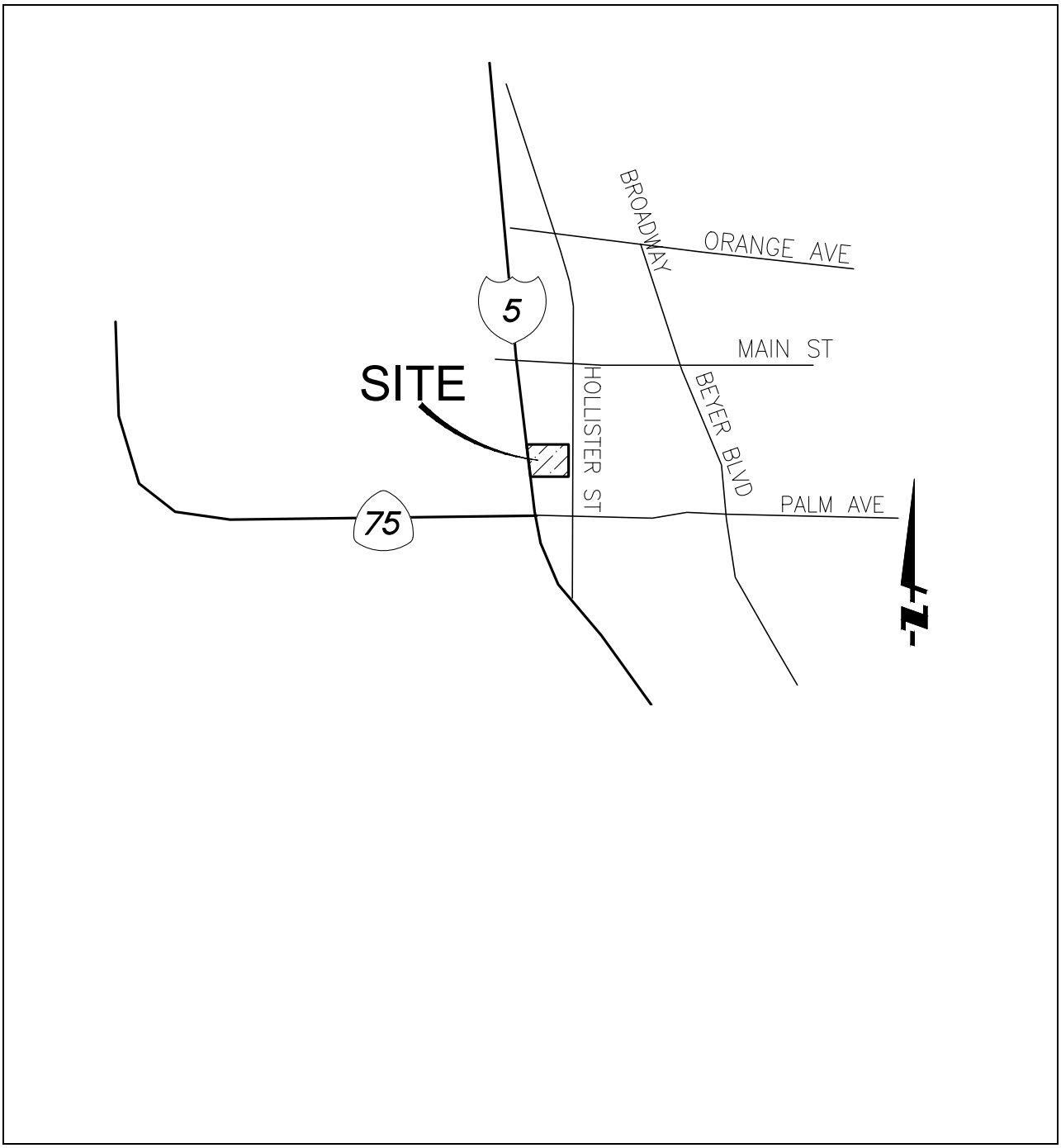
Submittal Record

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	02/15/19	☑ Preliminary Design/Planning/CEQA ☐ Final Design	Initial Submittal
2	10/15/19	☑ Preliminary Design/Planning/CEQA ☐ Final Design	Appendix B Worksheetsadded
3	11/25/20	☑ Preliminary Design/Planning/CEQA ☐ Final Design	Modified Green Streets
4		☐ Preliminary Design/Planning/CEQA ☐ Final Design	

Project Vicinity Map

Project Name: Bella Mar Apartments
Permit Application



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

Attach DS-560 form.

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

Storm Water Requirements Applicability Checklist

FORM
DS-560
November 2018

Project Address: 408 HOLLISTER STREET	Project Number:

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".

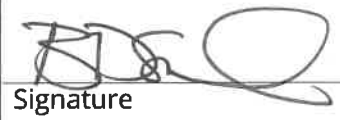
1.	**New Development that creates 10,000 square feet or more of impervious surfaces collectively over the project site.** This includes commercial, industrial, residential, mixed-use, and public development projects on public or private land.	☒ Yes ☐ No
2.	**Redevelopment project that creates and/or 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	☒

Bryan D Smith	Civil Engineer
Name of Owner or Agent *(Please Print)*	Title
	3.4.19
Signature	Date

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 ☒

Bryan D Smith, PE	Civil Engineer
Name of Owner or Agent *(Please Print)*	Title
Signature	Date

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Applicability of Permanent, Post-Construction Storm Water BMP Requirements	Form I-1

Project Identification

Project Name: Bella Mar Apartments	
Permit Application Number:	Date:

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: Bella Mar Apartments

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 (not required if prior lawful approval does not apply):		
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 not apply: The project discharges into the Otay River and ultimately San Diego Bay. Hydromod exempt because discharge point is at or below 10-yr base elevation. See supporting documentation in attachment 2		
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 not apply: There are no CCSYAs on site and the site does not impact any upstream CCSYAs.		

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.

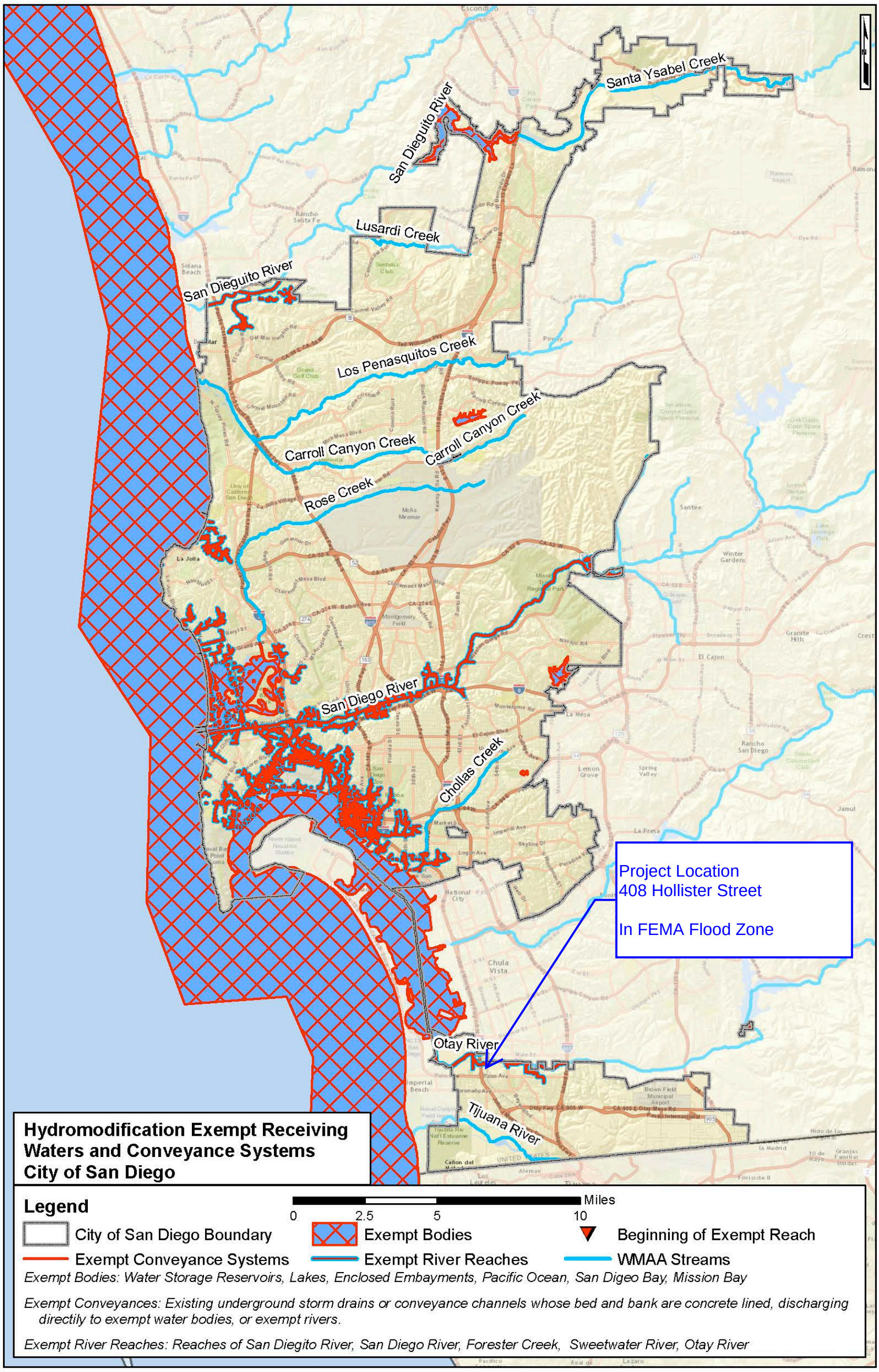

Figure H.9-2 : Hydromodification Exempt Areas

From: Gefrom, Walter <WGefrom@sandiego.gov>
Sent: Thursday, December 13, 2018 5:06 PM
To: Bryan Smith
Cc: Vera, Karen; Martin Jones; Mike Hoe
Subject: RE: Bella Mar HMP - PTS 598995
Attachments: Bella Mar Hydromod Exempt Memo.pdf

Categories: Filed by Newforma

Bryan,

My review of this and comments apply to the project once it comes through my group for Ministerial review/approval. Currently, it appears to be under review/approval through preliminary/discretionary. Also, keep in mind that any revisions to the State storm water permit before you acquire a grading/building permit may require me to void the memo.

Based on the exhibit and supporting documentation provided, the discharge of any storm water treated flows will not require HMP. You will need to revise the memo per my markups based on the updated Storm Water Standards Manual and also update the last page (Manual excerpt) with the newest language. Mike Hoe won't need to sign the memo if he sends the memo directly to me through e-mail instead of a cc. Or, he may acknowledge that he's seen it.

Thanks,

Walter C. Gefrom, PE, QSD, CFM
Deputy City Engineer

Development Services Department - Engineering Division
1222 First Avenue | San Diego | CA | 92101
MS 501

Visit OpenDSD for project info: https://www.sandiego.gov/development-services/opendsd

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From: Bryan Smith [mailto:bsmith@fuscoe.com]
Sent: Tuesday, December 11, 2018 7:59 AM
To: Gefrom, Walter <WGefrom@sandiego.gov>
Cc: Vera, Karen <KVera@sandiego.gov>
Subject: RE: Bella Mar HMP - PTS 598995

Hi Walter,

Just wanted to follow up on this. Can you please review and get back to me when you have the chance?

Thanks,

BRYAN D. SMITH, PE | *Project Manager*

FUSCOE ENGINEERING, INC.
an employee owned company

full circle thinking®

858.554.1500

From: Bryan Smith
Sent: Friday, November 30, 2018 11:05 AM
To: 'Gefrom, Walter' <WGefrom@sandiego.gov>
Cc: 'Vera, Karen' <KVera@sandiego.gov>
Subject: Bella Mar HMP - PTS 598995

Walter,

Hope all is well with you. Over a year ago, we met and discussed this multifamily residential project in Otay Nestor at 408 Hollister. I don't expect that you recall the original meeting, but we discussed a possible HMP exemption for this site. The project went on hold for some time but has since been restarted. Most recently, we went through preliminary review and were assigned the above PTS number and also met with you to discuss stockpiling in the FEMA Floodplain, as you probably recall. The stockpiling idea has been put on hold but we are working toward an entitlement submittal for a Tentative Map.

Our discussion last year was based on the Hydromod exemption. The site discharges through a Caltrans culvert under the I-5 free and discharges to an unlined channel to the West. The unlined channel and culvert outlet is within the 10-year flood plain elevation associated with the Otay River (see attached Memo documenting this). According to our meeting, you believed this would qualify for an exemption from HMP requirements but you asked that we document it in a memo and send to you.

Please find the attached memo which we will include the SWQMP. If you could please take a quick review when you have the chance and confirm our understanding it would be much appreciated.

Best,

25 YEARS

BRYAN D. SMITH, PE | *Project Manager*
bsmith@fuscoe.com
CONNECT WITH ME

FUSCOE ENGINEERING, INC.
an employee owned company

6390 Greenwich Drive, Suite 170, San Diego, California 92122
858.554.1500 | fuscoe.com

IRVINE . **SAN DIEGO** . ONTARIO . LOS ANGELES . EL CENTRO . SAN RAMON . MISSION HILLS

Hydromodification Exemption Memo - Bella Mar

To: Walter Gefrom, P.E., City of San Diego DSD

From: Michael Hoe, P.E., Fuscoe Engineering, Inc.

Date: May 17, 2017, Revision Date: December 14, 2018

The subject property is located at 408 Hollister Street in the City of San Diego, County of San Diego. The project site is bordered by private properties to the North and South, Hollister Street on the East and the Interstate 5 Freeway on the West. See the attached project site exhibit on the following sheet. Stormwater runoff on the subject property flows from east to west and discharges into an existing 24" storm drain culvert which runs below the I-5 Interchange bridge. The runoff eventually discharges into the Otay River and ultimately into the San Diego Bay.

Per City of San Diego Storm Water Standards Section 1.6, the Otay River is classified as a hydromodification exempt body of water.

"*Designated exempt river reaches within City of San Diego jurisdiction include the Otay River downstream of Lower Otay Reservoir Dam (Savage Dam). To qualify as a direct discharge to this exempt river reach, the invert elevation of the direct discharge conveyance system (at the point of discharge to the exempt river reach) should be equal to or below the 10-year floodplain elevation. The City Engineer may require additional analysis of the potential for erosion between the outfall and the 10-year floodplain elevation*."

The flowline elevation at the outlet of the existing 24" storm drain culvert is 12.7' NGVD29 or 14.9' NAVD 88 (see conversion table on the next sheet) per Caltrans As-built Drawing Document Number A-0002600. See the attached as-built drawing for reference.

Based on the most recent Flood Insurance Study (revised May 16, 2012), the 10-year water surface elevation below the Interstate 5 Bridge at the storm drain outlet is 14.9' (NAVD 88). See the flood profile for the Otay River in the following attachments. The storm drain outfall elevation is the same elevation as the 10-year base flood elevation therefore, the project should be considered exempt from Hydromodification Management requirements.

Attachments:

1- Site Plan
2- Caltrans As-Built
3- Flood Insurance Study
4- BMP Manual Excerpt

ATTACHMENT 1
SITE PLAN



ATTACHMENT 2
CALTRANS AS-BUILT

DIST.	COUNTY	ROUTE	POST MILES—TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
11	SD	5	4.4/9.0	42	437

DISTRICT DESIGN ENGINEER Henry R. Obrecht REGISTERED CIVIL ENGINEER No. 15768
DATE APPROVED July 27, 1970

14.9' IE-OUT (NAVD 88)

16.2' IE-IN (NAVD 88)

L0
Δ = 04° 36' 28"
R = 10,000'
T = 402.32'
L = 804.21'

275+59.38 EC L0

Otay Riv Br
Br. No. 57-246R/L

Otay Riv Overflow Br
Br. No. 57-263R/L

R/W

S Dike

Graded "V" Gutter 2:1 SS

N06°54'00"W 285

2693.09'

Profile Grade

Ground Line

-0.38%

NOTE
Profile Grade is 15' Lt & Rt of ℄ L0

U.S.C & G.S. Datum

"AS BUILT"
CONTRACT 122404
YEAR COMPLETED 8-1-73

DRAINAGE DETAILS AND CONTOUR GRADING PLANS

Project Engineer	Date	Design Engineer	Date	Approval Recommended By	Date

AS BUILT PLANS
Contract No. 11-122404
Date Completed 07-70
Document No. 11-000260

FORM WH23A-PF

4

I HEREBY CERTIFY THAT THIS IS A TRUE AND ACCURATE COPY OF THE ABOVE DOCUMENT TAKEN UNDER MY DIRECTION AND CONTROL ON THIS DATE IN SACRAMENTO, CALIFORNIA PURSUANT TO AUTHORIZATION BY THE DIRECTOR OF TRANSPORTATION.

DATE 8-26-74 SIGNATURE [signature] TITLE [illegible]

SCALE 1"=50' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=20' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=10' or 1"=100' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=40' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM

ATTACHMENT 3 :
FLOOD INSURANCE
STUDY

CHARLES AVENUE

LOURET AVENUE

INTERSTATE HIGHWAY 5

INTERSTATE HIGHWAY 5

14.9' 10 YEAR FLOOD ELEVATION @ OUTLET

15.1' 10-YEAR FLOOD ELEVATION @ INLET

200'

ELEVATION IN FEET (NAVD)

30

20

10

0

2600 2800 3000 3200 3400 3600 3800 4000 4200 4400 4600 4800 5000 5200

STREAM DISTANCE IN FEET ABOVE CROSS SECTION A

D E F G

LEGEND

- 0.2% ANNUAL CHANCE FLOOD
- 1% ANNUAL CHANCE FLOOD
- 2% ANNUAL CHANCE FLOOD
- 10% ANNUAL CHANCE FLOOD
- STREAM BED
- CROSS SECTION LOCATION

FLOOD PROFILES

OTAY RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN DIEGO COUNTY, CA
(AND INCORPORATED AREAS)

259P

TABLE 12: FLOODING SOURCE DATUM SHIFT VALUES

Stream Name	Elevation (feet NAVD above NGVD)
Moosa Creek (North Branch)	+2.3
Moosa Creek (South Branch)	+2.3
Murphy Canyon Creek	+2.1
Murray Canyon Creek	+2.1
Nestor Creek	+2.1
North Avenue Tributary	+2.3
North Branch Poway Creek	+2.1
North Tributary to Santa Maria Creek	+2.2
Olive Creek	+2.4
Otay River	+2.2
Pala Mesa Creek	+2.2
Paradise Creek	+2.1
Paradise Creek – Valley Road Branch	+2.1
Pilgrim Creek	+2.3
Poggi Canyon Creek	+2.2
Pomerado Creek	+2.1
Poway Creek	+2.1
Rainbow Creek (Main Branch)	+2.3
Rainbow Creek (West Branch)	+2.3
Rattlesnake Creek	+2.1
Rattlesnake Creek Split Flow at Heritage Hills	+2.1
Rattlesnake Creek Split Flow at Midland Road	+2.1
Reidy Creek	+2.3
Reidy Creek Split Flow	+2.3
Rice Canyon Creek	+2.1
Rincon Avenue Tributary	+2.3
Rose Canyon Creek	+2.1
Samagutuma Creek	+2.4
San Clemente Canyon Creek	+2.1
San Diego Bay	+2.2
San Diego River	+2.1
San Dieguito River	+2.1
San Elijo Creek	+2.2
San Luis Rey River	+2.3
San Marcos Creek	+2.3
San Marcos Creek (Below Lake San Marcos)	+2.3
San Marcos Creek Highway 78 Split Flow	+2.3

- This exemption is subject to the following conditions:
  (a) A properly sized energy dissipation system must be provided in accordance with the City design standards to mitigate outlet discharge velocity from the direct discharge to the water storage reservoir or lake for the ultimate condition peak design flow of the direct discharge,
  (b) The invert elevation of the direct discharge conveyance system (at the point of discharge to the water storage reservoir or lake) should be equal to or below the lowest normal operating water surface elevation at the point of discharge, unless the outfall discharges to quay or other non-erodible shore protection. Normal operating water surface elevation may vary by season; contact the reservoir operator to determine the elevation. For cases in which the direct discharge conveyance system outlet invert elevation is above the lowest normal operating water surface elevation but below the reservoir spillway elevation, additional analysis is required to determine if energy dissipation should be extended between the conveyance system outlet and the elevation associated with the lowest normal operating water surface level.
- No exemption may be granted for conveyance system outlet invert elevations located above the reservoir spillway elevation.

- **Figure 1-2, Node 5** – As allowed by the MS4 Permit, projects discharging directly to an area identified as appropriate for an exemption in the WMAA for the watershed in which the project resides are exempt. Refer to the WMAA for any updates to exempt river reaches. Discharging directly refers to either a) existing underground storm drain systems; or b) conveyance channels whose bed and bank are concrete-lined all the way from the point of discharge to the designated area.
  - Designated exempt river reaches identified in the WMAA and approved by the RWQCB within City of San Diego jurisdiction:
    (a) San Dieguito River downstream of Lake Hodges
    (b) San Diego River downstream of confluence with San Vicente Creek
    (c) Sweetwater River downstream of Sweetwater Reservoir
    (d) Otay River downstream of Lower Otay Reservoir Dam
  - To qualify as a direct discharge to an exempt river reach:
    (a) A properly sized energy dissipation system must be provided to mitigate outlet discharge velocity from the direct discharge to the exempt river reach for the ultimate condition peak design flow of the direct discharge,
    (b) The invert elevation of the direct discharge conveyance system (at the point of discharge to the exempt river reach) should be equal to or below the 10-year floodplain elevation. Exceptions may be made at the discretion of the City Engineer, but shall never exceed the 100-year floodplain elevation. The City Engineer may require additional analysis of the potential for erosion between the outfall and the 10-year floodplain elevation.
  - No exemption may be granted for conveyance system outlet invert elevations located above the 100-year floodplain elevation.

General note regarding HMP: New outfalls shall meet requirements for energy dissipation size in the Drainage Design Manual regardless of the addition of hydromodification controls. Existing outfalls that are insufficient to accommodate additional flows from proposed upstream development projects

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Site Information Checklist For PDPs	Form I-3B
Project Summary Information	
Project Name	Bella Mar Apartments
Project Address	408 Hollister Street San Diego, CA 92154
Assessor's Parcel Number(s) (APN(s))	627-100-09-00
Permit Application Number	
Project Watershed	Select One: ☐ San Dieguito River ☐ Penasquitos ☐ Mission Bay ☐ San Diego River ☑ San Diego Bay ☐ Tijuana River
Hydrologic subarea name with Numeric Identifier up to two decimal places (9XX.XX)	Otay/Otay Valley, 910.20
Project Area (total area of Assessor's Parcel(s) associated with the project or total area of the right-of-way)	14.13 Acres (615,502 Square Feet)
Area to be disturbed by the project (Project Footprint)	12.2 Acres (529,270 Square Feet)
Project Proposed Impervious Area (subset of Project Footprint)	8.6 Acres (373,372 Square Feet)
Project Proposed Pervious Area (subset of Project Footprint)	3.6 Acres (155,898 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	3500 %

Form I-3B Page 2 of 11

Description of Existing Site Condition and Drainage Patterns

Current Status of the Site (select all that apply):

- [x] Existing development
- [ ] Previously graded but not built out
- [ ] Agricultural or other non-impervious use
- [x] Vacant, undeveloped/natural

Description / Additional Information:

The existing site is vacant and mostly undeveloped. There is a small existing parking lot in the northeast corner.

Existing Land Cover Includes (select all that apply):

- [x] Vegetative Cover
- [ ] Non-Vegetated Pervious Areas
- [x] Impervious Areas

Description / Additional Information:

The existing land cover is mostly brush and grass. There is a small area with asphalt parking lot in the northeast corner of the site.

Underlying Soil belongs to Hydrologic Soil Group (select all that apply):

- [x] NRCS Type A
- [ ] NRCS Type B
- [ ] NRCS Type C
- [ ] NRCS Type D

Approximate Depth to Groundwater:

- [ ] Groundwater Depth < 5 feet
- [x] 5 feet < Groundwater Depth < 10 feet
- [ ] 10 feet < Groundwater Depth < 20 feet
- [ ] Groundwater Depth > 20 feet

Existing Natural Hydrologic Features (select all that apply):

- [x] Watercourses
- [ ] Seeps
- [ ] Springs
- [ ] Wetlands
- [ ] None

Description / Additional Information:

The site is located in an existing flood plain. Per FEMA Flood Insurance Rate Map, the site is located in within the floodway boundary

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

Overview:
The site is bounded by Otay River to the north, existing driving range to the south, Interstate 5 Freeway to the west, and Hollister Street to the east. Improvements within ROW are included in the Street Improvement Plans and improvements outside ROW are included in the Grading Plans. The existing site mostly vacant and undeveloped. The site generally drains west toward an existing 24-inch RCP storm drain that runs underneath the Interstate 5 Freeway.

1. Existing drainage conveyance is natural.
2. In existing conditions, the offsite runon from Hollister Street and the adjacent site on the south runs through the site. A portion of the undeveloped area drains towards Otay River.
3. There is an existing 24-inch RCP storm drain located on the near the west boundary of the site.
4. In existing conditions, there are 3 discharge points. One discharge point is in the northwest of the site and drains towards Otay River, the second discharge point is to the existing 24-inch RCP storm drain located in towards the middle of the west boundary, and the third location is located at the southeast corner of the site.

See the Existing Hydrology Map in the Drainage Study included in Attachment 5.

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 development consists of the construction of residential apartments and affordable housing, associated drive aisles, recreation facilities, hardscape, and landscaping.

List/describe proposed impervious features of the project (e.g., buildings, roadways, parking lots, courtyards, athletic courts, other impervious features):

The proposed impervious features include buildings, associated drive aisles, parking spaces, and hardscape.

List/describe proposed pervious features of the project (e.g., landscape areas):

The proposed pervious features include landscaped areas and planter areas.

Does the project include grading and changes to site topography?

☑ Yes

☐ No

Description / Additional Information:

The project proposes to raise the site above the flood plain. The site will still generally drain west to match existing conditions.

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:

Runon from the offsite areas will be diverted around the site using curb and gutter and a vegetated swale. Runoff will be collected and conveyed by a series of area drains, drain inlets, and storm drain throughout the site and will connect to an existing 24-inch RCP storm drain system that runs west underneath Interstate 5 Freeway. The site eventually discharges into Nestor Creek and ultimately into San Diego Bay.

See the Proposed Hydrology Map in the Drainage Study included in Attachment 5.

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):

- [x] Onsite storm drain inlets
- [ ] Interior floor drains and elevator shaft sump pumps
- [ ] Interior parking garages
- [x] Need for future indoor & structural pest control
- [x] Landscape/outdoor pesticide use
- [x] Pools, spas, ponds, decorative fountains, and other water features
- [ ] Food service
- [x] Refuse areas
- [ ] Industrial processes
- [x] Outdoor storage of equipment or materials
- [ ] Vehicle and equipment cleaning
- [ ] Vehicle/equipment repair and maintenance
- [ ] Fuel dispensing areas
- [ ] Loading docks
- [ ] Fire sprinkler test water
- [x] Miscellaneous drain or wash water
- [x] Plazas, sidewalks, and parking lots

Description/Additional Information:

Form I-3B Page 7 of 11
Identification and Narrative of Receiving Water
Narrative describing flow path from discharge location(s), through urban storm conveyance system, to receiving creeks, rivers, and lagoons and ultimate discharge location to Pacific Ocean (or bay, lagoon, lake or reservoir, as applicable) The project site discharges into an existing 24-inch RCP storm drain on near the west boundary. The existing storm drain conveys flow to the west into the Otay River and ultimately into San Diego Bay and Pacific Ocean.
Provide a summary of all beneficial uses of receiving waters downstream of the project discharge locations San Diego Bay - BIOL, COMM, EST, IND, MAR, MIGR, NAV, RARE, REC1, REC2, SHELL, WILD
Identify all ASBS (areas of special biological significance) receiving waters downstream of the project discharge locations N/A - No ASBS receiving waters downstream of project discharge.
Provide distance from project outfall location to impaired or sensitive receiving waters The project site drains to an existing storm drain system that ultimately discharges into the Otay River less than a mile away, which is ultimately discharges into San Diego Bay, which is approximately 3 miles from the site.
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 Approximately 40% of the northern portion of the site is within the City's current MHPA limits. A proposed infiltration basin will encroach into the current MHPA limit.

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)
Poggi Canyon Creek	Nitrogen, Toxicity	TMDL Required
San Diego Bay	Mercury, PAHs, PCBs	TMDL Required
Lower Otay Reservoir	Ammonia, Color, Iron, Manganese, Nitrogen, high pH	TMDL Required

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):

Project discharges into an existing culvert that discharges into the Otay River, which ultimately discharges into San Diego Bay and Pacific Ocean.

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:

Project will not impact any areas determined to be CCSYAs.

Form I-3B Page 10 of 11

Flow Control for Post-Project Runoff*

***This Section only required if hydromodification management requirements apply**

List and describe point(s) of compliance (POCs) for flow control for hydromodification management (see Section 6.3.1). For each POC, provide a POC identification name or number correlating to the project's HMP Exhibit and a receiving channel identification name or number correlating to the project's HMP Exhibit.

N/A - Hydromodification exempt

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:

Hydromodification not required.

Discussion / Additional Information: (optional)

Hydromodification not required.

Form I-3B Page 11 of 11
Other Site Requirements and Constraints
When applicable, list other site requirements or constraints that will influence storm water management design, such as zoning requirements including setbacks and open space, or local codes governing minimum street width, sidewalk construction, allowable pavement types, and drainage requirements. The site is located in a flood plain. It is constrained by the existing freeway, Otay River, existing utilities, and adjacent private property.
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:			

Project Name: Bella Mar Apartments

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:			
4.3.5 Impervious Area Dispersion	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.3.5 not implemented:			
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

Project Name: Bella Mar Apartments

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 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: Harvest and reuse was considered infeasible for the site per Worksheet B.3-1.			
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:

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.

Step 1:
(A) DMA 2 will be self-treating. There are two small areas at the driveway entrances that will be considered "de minimus" and will be treated using permeable pavers.
(B) DCV is estimated for each delineated DMA; see calculations in Attachment 1.

Step 2:
Harvest and use is infeasible per Worksheet B3.1

Step 3:
(A) Form I-8A has been completed per the preliminary Geotechnical Report. Infiltration is infeasible.
(B) No Infiltration Condition.
(C) No Infiltration Condition per preliminary Geotechnical Report.

(continued on next page)

(Continue on page 2 as necessary.)

Form I-6 Page 2 of 8

(Continued from page 1)

Step 4:
DMA 1: MHPA area will be considered self-treating.

DMA 2: Biofiltration Basin (BF-1) will be used. Stormwater will be conveyed to BMP via sheet flow and storm drain.

DMA 3: Slopes will be considered self-treating

DMA 4: Permeable pavers (INF-3) will be used and considered self-retaining

DMA 5: Slope will be considered self-treating

DMA 6: Landscape area at base of wall considered self-treating

DMA 7: Bioclean MWS unit will be used. Stormwater will be conveyed to BMP via sheet flow and storm drain.

DMA 8: Permeable pavers (INF-3) will be used and considered self-retaining

DMA 9: Landscape area at base of wall considered de minimus.

Hollister St: Green street BMPs will be implemented to treat runoff from Hollister St.

Project is hydromodification exempt. All BMPs will provide pollutant control only.

Form I-6 Page 1 of 8 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 2	
Construction Plan Sheet No. 3	
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	Bryan Smith, PE Fuscoe Engineering 858-554-1500
Who will be the final owner of this BMP?	
Who will maintain this BMP into perpetuity?	
What is the funding mechanism for maintenance?	Private

Form I-6 Page 2 of 8 (Copy as many as needed)
Structural BMP ID No. BMP 2
Construction Plan Sheet No. 3
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): See Sizing calculation worksheets

Form I-6 Page 3 of 8 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 4	
Construction Plan Sheet No. 3	
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	
Who will be the final owner of this BMP?	
Who will maintain this BMP into perpetuity?	
What is the funding mechanism for maintenance?	Private

Form I-6 Page 4 of 8 (Copy as many as needed)
Structural BMP ID No. BMP 4
Construction Plan Sheet No. 3
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): See Sizing calculation worksheets

Form I-6 Page 5 of 8 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 7	
Construction Plan Sheet No. 3	
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	
Who will be the final owner of this BMP?	
Who will maintain this BMP into perpetuity?	
What is the funding mechanism for maintenance?	Private

Form I-6 Page 6 of 8 (Copy as many as needed)
Structural BMP ID No. BMP 7
Construction Plan Sheet No. 3
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): Bioclean MWS unit See Sizing calculation worksheets

Form I-6 Page 7 of 8 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BMP 8	
Construction Plan Sheet No. 3	
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	
Who will be the final owner of this BMP?	
Who will maintain this BMP into perpetuity?	
What is the funding mechanism for maintenance?	Private

Form I-6 Page 8 of 8 (Copy as many as needed)
Structural BMP ID No. BMP 8
Construction Plan Sheet No. 3
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): See Sizing calculation worksheets

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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: Bella Mar Apartments

Indicate which Items are Included:

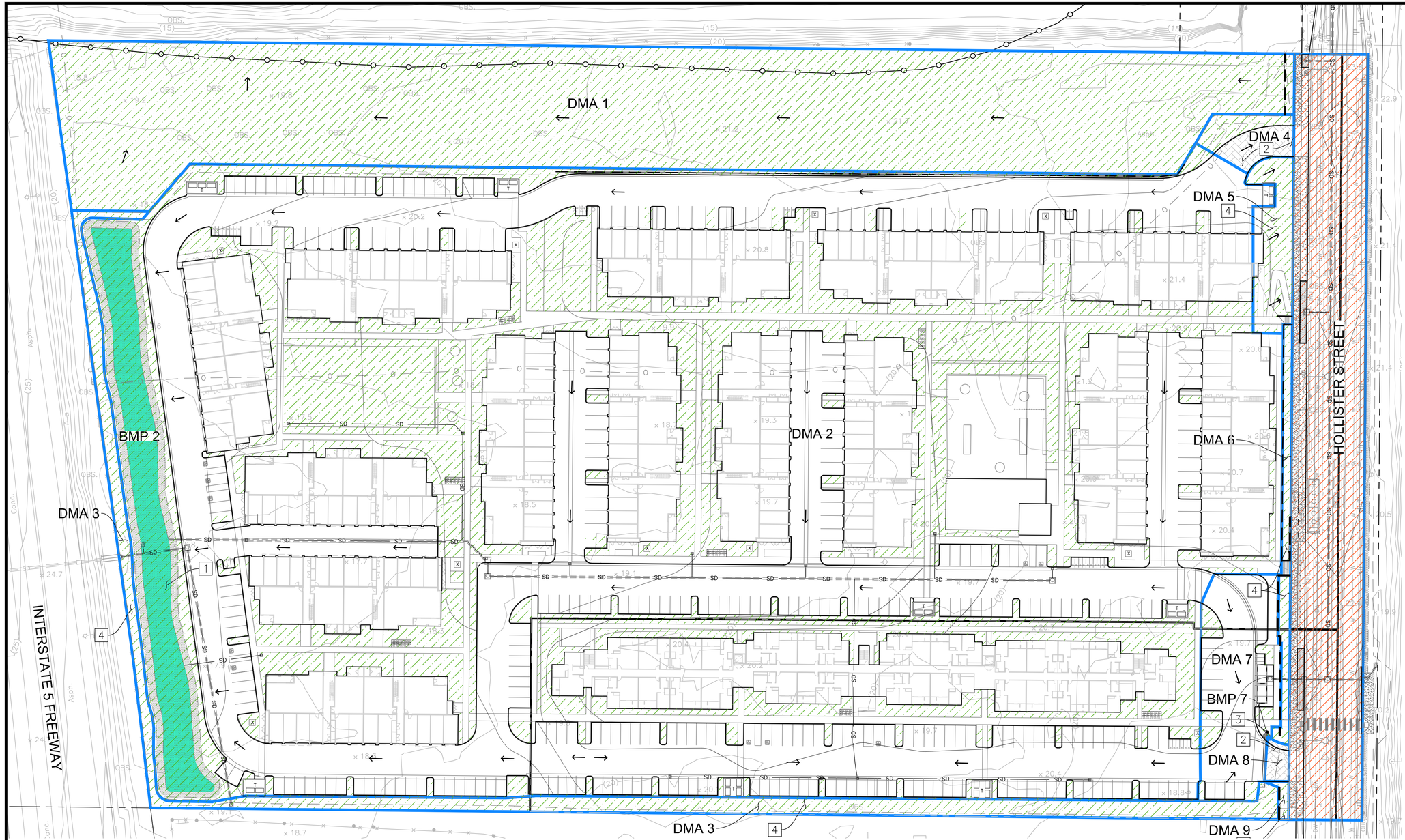
Attachment Sequence	Contents	Checklist
Attachment 1a	DMA Exhibit (Required) See DMA Exhibit Checklist.	☑ Included
Attachment 1b	Tabular Summary of DMAs Showing DMA ID matching DMA Exhibit, DMA Area, and DMA Type (Required)* *Provide table in this Attachment OR on DMA Exhibit in Attachment 1a	☑ Included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

Project Name: Bella Mar Apartments

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

The DMA Exhibit must identify:

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



LEGEND

- PROPERTY LINE
- EXISTING EASEMENT
- RIGHT-OF-WAY
- STREET CENTERLINE
- EXISTING STORM DRAIN
- DMA LIMITS
- DIRECTION OF FLOW
- PROPOSED BIOFILTRATION — BMP 1
- DMA DESIGNATION — DMA 1
- EXISTING CONTOUR — 260
- PROPOSED CONTOUR — 280
- PROPOSED STORM DRAIN
- PROPOSED PERVIOUS AREA
- PROPOSED IMPERVIOUS AREA
- GREEN STREET EXEMPTION

PROJECT SITE INFO

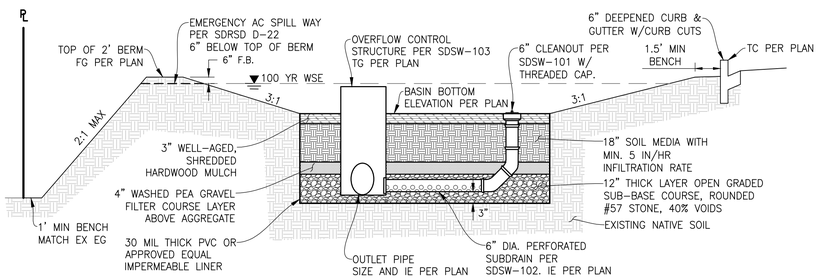
UNDERLYING HYDROLOGIC SOIL: A
APPROXIMATE DEPTH TO GROUNDWATER: 5 FT
EXISTING NATURAL HYDROLOGIC FEATURES (WATERCOURSES, SEEPS, SPRINGS, WETLANDS): NONE
CRITICAL COARSE SEDIMENT YIELD AREAS TO BE PROTECTED: NONE
EXISTING IMPERVIOUS AREA: 10,655 SF
DISTURBED AREA: 529,270 SF
PROPOSED/REPLACED IMPERVIOUS AREA: 373,372 SF
PROPOSED PERVIOUS AREA (INCLUDES LANDSCAPING & PERMEABLE PAVERS): 155,898 SF

PERMANENT STORM WATER BMP NOTES

1. POLLUTANT CONTROL BMP (BF-1, BIOFILTRATION W/PARTIAL INFILTRATION)
2. POLLUTANT CONTROL BMP (INF-3, PERMEABLE PAVEMENT)
3. POLLUTANT CONTROL BMP (FT-5, PROPRIETARY FLOW-THRU TREATMENT)
4. POLLUTANT CONTROL BMP (SD-B, IMPERVIOUS AREA DISPERSION)

SOURCE CONTROL BMPs

SC-1 PREVENTION OF ILLICIT DISCHARGES INTO THE MS4
SC-2 STORM DRAIN STENCILING OR SIGNAGE
SC-5 PROTECT TRASH STORAGE AREAS FROM RAINFALL, RUN-ON, RUNOFF, AND WIND DISPERSAL



BIOFILTRATION BASIN DETAIL

NOT TO SCALE

POLLUTANT CONTROL BMP SUMMARY TABLE

DMA	TOTAL AREA (AC)	TOTAL AREA (SF)	PERVIOUS AREA (SF)	PERMEABLE PAVERS (SF)	IMPERVIOUS AREA - ROOFS, CONCRETE, ASPHALT (SF)	POLLUTANT CONTROL: METHOD OF TREATMENT	DCV (CF)	TREATMENT REQUIRED BASIN AREA (SF) OR FLOWRATE (CFS)	TREATMENT PROVIDED BASIN AREA (SF) OR FLOWRATE (CFS)
1	2.31	100,570	100,570	0	0	Self Mitigating	0	-	-
2	10.85	472,658	111,331	0	361,326	Drains to BMP-2 (Bio-filtration)	14575	10,090	11,220
3	0.39	16,785	16,785	0	0	Self Mitigating	0	-	-
4	0.06	2,791	929	1,862	0	Self Retaining	21	21	186
5	0.11	4,693	3,318	0	1,375	Self Mitigating	0	-	-
6	0.06	2,739	2,450	0	289	Self Mitigating	0	-	-
7	0.26	11,296	915	0	10,381	Drains to BMP-7 (MWS)	409	0.065	0.073
8	0.01	619	0	619	0	Self Retaining	6	6	62
9	0.01	298	298	0	0	Self Mitigating	0	-	-

NO.	DATE	REVISION

SCALE: 1" = 40'

BELLA MAR

ATTACHMENT 1a & 1□ - DRAINAGE MANAGEMENT AREAS (DMA) EXHIBIT MAP

FUSCOE ENGINEERING
6390 Greenwich Dr., Suite 170
San Diego, California 92122
tel 858.554.1500 ◦ fax 858.597.0335
www.fuscoe.com

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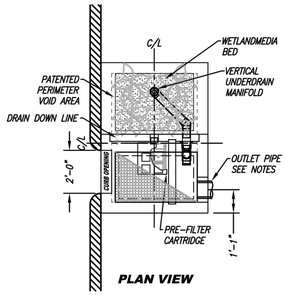
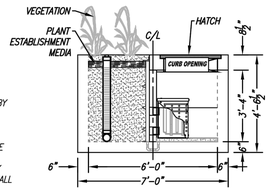
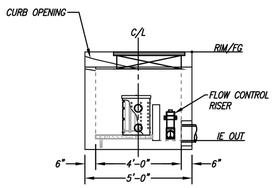
SITE SPECIFIC DATA			
PROJECT NUMBER			
ORDER NUMBER			
PROJECT NAME			
PROJECT LOCATION			
STRUCTURE ID			
TREATMENT REQUIRED			
VOLUME BASED (CF)		FLOW BASED (CFS)	
TREATMENT HGL AVAILABLE (FT)			
PEAK BYPASS REQUIRED (CFS) – IF APPLICABLE			
PIPE DATA	I.E.	MATERIAL	DIAMETER
INLET PIPE 1			
INLET PIPE 2			
OUTLET PIPE			
	PRETREATMENT	BIOFILTRATION	DISCHARGE
RIM ELEVATION			
SURFACE LOAD	PEDESTRIAN	OPEN PLANTER	PEDESTRIAN
FRAME & COVER	24" X 42"	N/A	N/A
WETLANDMEDIA VOLUME (CY)			TBD
ORIFICE SIZE (DIA. INCHES)			TBD
NOTES: PRELIMINARY NOT FOR CONSTRUCTION			



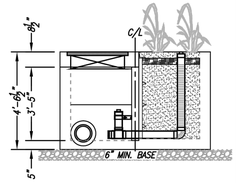
PLAN VIEW



LEFT END VIEW



ELEVATION VIEW



RIGHT 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.
3. 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 GAPS AROUND PIPES SHALL BE SEALED WATER TIGHT WITH A NON-SHRINK GROUT PER MANUFACTURERS STANDARD CONNECTION DETAIL AND SHALL MEET OR EXCEED REGIONAL PIPE CONNECTION STANDARDS.
4. CONTRACTOR TO SUPPLY AND INSTALL ALL EXTERNAL CONNECTING PIPES.
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. DRIP OR SPRAY IRRIGATION REQUIRED ON ALL UNITS WITH VEGETATION.
7. CONTRACTOR RESPONSIBLE FOR CONTACTING MODULAR WETLANDS FOR ACTIVATION OF UNIT. MANUFACTURES WARRANTY IS VOID WITH OUT PROPER ACTIVATION BY A MODULAR WETLANDS REPRESENTATNE.

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 MANUFACTURER.

TREATMENT FLOW (CFS)	0.073
OPERATING HEAD (FT)	3.4
PRETREATMENT LOADING RATE (GPM/SF)	2.6
WETLAND MEDIA LOADING RATE (GPM/SF)	1.0

THIS PRODUCT MAY BE PROTECTED BY ONE OR MORE OF THE FOLLOWING US PATENTS: 7,425,263, 7,470,362, 8,293,098, 8,802,616, RELATED FOREIGN PATENTS OR PATENTS PENDING WORLDWIDE

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MWS-L-4-6-C
STORMWATER BIOFILTRATION SYSTEM
STANDARD DETAIL

PROPRIETARY MODULAR WETLAND BIOFILTRATION BMP

NOT TO SCALE

NO.	DATE	REVISION

FUSCOE ENGINEERING
6390 Greenwich Dr., Suite 170
San Diego, California 92122
tel 858.554.1500 ◦ fax 858.597.0335
www.fuscoe.com

BELLA MAR

ATTACHMENT 1a & 1□ - DRAINAGE MANAGEMENT AREAS (DMA) EXHIBIT MAP

JOB NO. 1621-004

DRAWN BY: B.V.

SHEET 2 o□2

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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]
388 units x 1.5 residents/unit = approx 582 residents

582 residents x 9.3 gal/day (Table B.3-1) = 5413 gal/day = 901 cf/day
36 hr demand = 901 cf/day x 1.5 days = 1351 cf

3. Calculate the DCV using worksheet B-2.1.
DCV = 13,398 (cubic feet)
[Provide a summary of calculations here]
85th percentile 24-hr storm: d = 0.52 inches
Area tributary to BMP = 11.83 acres
Weighted runoff coefficient: C = 0.60
No tree credit or rain barrel credit

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

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	Worksheet C.4-1: Form I-8A[10]
Part 1 - Full Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
Bella Mar – 408 Hollister Street	Design
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[11]? ☒ 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.

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.

[10] 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.

[11] Available data include 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[10]
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.

Based on the USGS Soil Survey, the property possesses a Hydrologic Soil Group A classification. In addition, we encountered field infiltration rates of:

P-1: 0.30 inches/hour (0.15 with a FOS of 2.0)
P-2: 0.25 inches/hour (0.13 with a FOS of 2.0)
P-3: 0.14 inches/hour (0.07 with a FOS of 2.0)
P-4: 0.13 inches/hour (0.07 with a FOS of 2.0)
P-5: 0.20 inches/hour (0.10 with a FOS of 2.0)
P-6: 0.12 inches/hour (0.06 with a FOS of 2.0)

This results in an average infiltration rate of 0.19 inches/hour (0.10 with a FOS of 2.0).

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
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[10]	
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[10]	
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.

The site is underlain by fill soils, topsoil and alluvium to depths of approximately 35 feet overlying Old Paralic Deposits and San Diego Formation. We performed 6 infiltration tests within the alluvium and the results indicate rates less than 0.5 inches per hour (with an applied factor of safety of 2). Therefore, full infiltration is considered infeasible within the alluvium.

The project area is mapped within a liquefaction zone. In addition, our calculations show a potential for liquefaction exists within the alluvium underlying the property. Therefore, infiltration should be considered infeasible to help prevent an increased thickness of liquefiable soil. In addition, groundwater exists at depths ranging from approximately 8 and 16 feet below the existing ground surface (approximate elevations ranging from 4 and 12 feet MSL). The elevation where infiltration is feasible is limited to the required 10 feet above the groundwater elevation. There is likely not enough vertical space between planned bottom of basin elevations and 10 feet above the groundwater elevation. Therefore, full and partial infiltration devices should be considered infeasible for the property.

Part 1 Result – Full Infiltration Geotechnical Screening[12]	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

[12] 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[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
Bella Mar – 408 Hollister Street	Design

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).

Based on the USGS Soil Survey, the property possesses a Hydrologic Soil Group A classification. In addition, we encountered field infiltration rates of:

P-1: 0.30 inches/hour (0.15 with a FOS of 2.0)
P-2: 0.25 inches/hour (0.13 with a FOS of 2.0)
P-3: 0.14 inches/hour (0.07 with a FOS of 2.0)
P-4: 0.13 inches/hour (0.07 with a FOS of 2.0)
P-5: 0.20 inches/hour (0.10 with a FOS of 2.0)
P-6: 0.12 inches/hour (0.06 with a FOS of 2.0)

This results in an average infiltration rate of 0.19 inches/hour (0.10 with a FOS of 2.0).

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 4B. 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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
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
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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
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

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

The site is underlain by fill soils, topsoil and alluvium to depths of approximately 35 feet overlying Old Paralic Deposits and San Diego Formation. We performed 6 infiltration tests within the alluvium and the results indicate rates less than 0.5 inches per hour (with an applied factor of safety of 2). Therefore, full infiltration is considered infeasible within the alluvium.

The project area is mapped within a liquefaction zone. In addition, our calculations show a potential for liquefaction exists within the alluvium underlying the property. Therefore, infiltration should be considered infeasible to help prevent an increased thickness of liquefiable soil. In addition, groundwater exists at depths ranging from approximately 8 and 16 feet below the existing ground surface (approximate elevations ranging from 4 and 12 feet MSL). The elevation where infiltration is feasible is limited to the required 10 feet above the groundwater elevation. There is likely not enough vertical space between planned bottom of basin elevations and 10 feet above the groundwater elevation. Therefore, full and partial infiltration devices should be considered infeasible for the property.

Part 2 – Partial Infiltration Geotechnical Screening Result[13]	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

[13] 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 Groundwater and Water Balance Conditions		Worksheet C.4-2: Form I-8B[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
BMPs 2, 4, 7, 8		TM
Criteria 1: Groundwater Screening		
1A	**Groundwater Depth.** Is the depth to seasonally high groundwater tables (normal high depth during the wet season) beneath the base of any full infiltration BMP greater than 10 feet? ○ Yes; continue to Step 1B. ○ No; The depth to groundwater is less than or equal to 10 feet, but site layout changes or reasonable mitigation measures can be proposed to support full infiltration BMPs. Continue to step 1B. ⊙ No; The depth to groundwater is less than or equal to 10 feet and site layout changes or reasonable mitigation measures cannot be proposed to support full infiltration BMPs. Answer "No" for Criteria 1 Result.	
1B	**Contaminated Soil/Groundwater.** Are proposed full infiltration BMPs at least 250 feet away from contaminated soil or groundwater sites? This can be confirmed using GeoTracker (geotracker.waterboards.ca.gov) to identify open contaminated sites. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP. ⊙ Yes; continue to Step 1C. ○ No; However, site layout changes or reasonable mitigation measures can be proposed to support full infiltration BMPs. Continue to Step 1C. ○ No; Site layout changes or reasonable mitigation measures cannot be proposed to support full infiltration BMPs. Answer "No" to Criteria 1 Result.	

[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.

Categorization of Infiltration Feasibility Condition based on Groundwater and Water Balance Conditions	Worksheet C.4-2: Form I-8B[2]
1C	**Inadequate Soil Treatment Capacity.** Are full infiltration BMPs proposed in DMA soils that have adequate soil treatment capacity? The DMA has adequate soil treatment capacity if **ALL** of the following criteria (detailed in C.2.2.1) for all soil layers beneath the infiltrating surface are met: • USDA texture class is sandy loam or loam or silt loam or silt or sandy clay loam or clay loam or silty clay loam or sandy clay or silty clay or clay; and • Cation Exchange Capacity (CEC) greater than 5 milliequivalents/100g; and • Soil organic matter is greater than 1%; and • Groundwater table is equal to or greater than 10 feet beneath the base of the full infiltration BMP. ○ Yes; continue to Step 1D. ○ No; However, site layout changes or reasonable mitigation measures can be proposed to support full infiltration BMPs. Continue to Step 1D. ◉ No; Site layout changes or reasonable mitigation measures cannot be proposed to support full infiltration BMPs. Answer "No" to Criteria 1 Result.
1D	**Other Groundwater Contamination Hazards.** Are there site-specific groundwater contamination hazards not already mentioned (refer to Appendix C.2.2) that can be reasonably mitigated to support full infiltration BMPs? ○ Yes; there are other contamination hazards identified that can be mitigated. Answer "Yes" to Criteria 1 Result. ○ No; there are other contamination hazards identified that cannot be mitigated. Answer "No" to Criteria 1 Result. ○ N/A; no contamination hazards are identified. Answer "Yes" to Criteria 1 Result.
Criteria 1 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination that cannot be reasonably mitigated to an acceptable level? See Appendix C.2.2.8 for a list of typically reasonable and typically unreasonable mitigation measures. ○ Yes; Continue to Part 1, Criteria 2. ◉ No; Continue to Part 1 Result.

Categorization of Infiltration Feasibility Condition based on Groundwater and Water Balance Conditions	Worksheet C.4-2: Form I-8B[2]

Summarize groundwater quality and any mitigation measures proposed. Documentation should focus on groundwater table, mapped soil types and contaminated site locations.

The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be at least 10 feet for infiltration to be allowed. CWE encountered groundwater at depths ranging from approximately 8 to 13 feet at the subject property Partial infiltration would be feasible within the alluvium at an elevation of at least 14 feet MSL.

Categorization of Infiltration Feasibility Condition based on Groundwater and Water Balance Conditions	Worksheet C.4-2: Form I-8B[2]
Criteria 2: Water Balance Screening	
2A	**Ephemeral Stream Setback.** Does the proposed full infiltration BMP meet both the following? • The full infiltration BMP is located at least 250 feet away from an ephemeral stream; **AND** • The bottom surface of the full infiltration BMP is at a depth 20 feet or greater from seasonally high groundwater tables. ○ Yes; Answer "Yes" to Criteria 2 Result. ◉ No; Continue to Step 2B.
2B	**Mitigation Measures.** Can site layout changes be proposed to support full infiltration BMPs? ○ Yes; the site can be reconfigured to mitigate potential water balance issues. Answer "Yes" to Criteria 2 Result. ◉ No; the site cannot be reconfigured to mitigate potential water balance issues. Continue to Step 2C and provide discussion.
2C	**Additional studies.** Do additional studies support full infiltration BMPs? In the event that water balance effects are used to reject full infiltration (anticipated to be rare), additional analysis shall be completed and documented by a qualified professional indicating the site-specific information evaluated and the technical basis for this finding. ○ Yes; Answer "Yes" to Criteria 2 Result. ◉ No; Answer "No" to Criteria 2 Result.
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams? ○ Yes; Continue to Part 1 Result. ◉ No; Continue to Part 1 Result.

Categorization of Infiltration Feasibility Condition based on Groundwater and Water Balance Conditions	Worksheet C.4-2: Form I-8B[2]

Summarize potential water balance effects. Documentation should focus on mapping and soil data regarding proximity to ephemeral streams and groundwater depth.

The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be at least 10 feet for infiltration to be allowed. CWE encountered groundwater at depths ranging from approximately 8 to 13 feet at the subject property Partial infiltration would be feasible within the alluvium at an elevation of at least 14 feet MSL.

We do not expect full infiltration would cause water balance issues including change of ephemeral streams or discharge of contaminated water to surface waters.

Part 1 – Full Infiltration Groundwater and Water Balance Screening Result[3]	Result
If answers to Criteria 1 and 2 are "Yes", a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration based on groundwater conditions.	
If answer to Criteria 1 or Criteria 2 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design based on groundwater conditions. Proceed to Part 2.	○ Full Infiltration ◉ Complete Part 2

[3] 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 Groundwater and Water Balance Conditions	Worksheet C.4-2: Form I-8B[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
BMPs 2, 4, 7, 8	TM
Criteria 3: Groundwater Screening	
Contaminated Soil/Groundwater. Are partial infiltration BMPs proposed at least 100 feet away from contaminated soil or groundwater sites? This can be confirmed using GeoTracker (geotracker.waterboards.ca.gov) to identify open contaminated sites. This criterion is intentionally a smaller radius than full infiltration, as the potential quantity of infiltration from partial infiltration BMPs is smaller. ◉ Yes; Answer "Yes" to Criteria 3 Result. ○ No; However, site layout changes can be proposed to avoid contaminated soils or soils that lack adequate treatment capacity. Select "Yes" to Criteria 3 Result. It is a requirement for the SWQMP preparer to identify potential mitigation measures. ○ No; Contaminated soils or soils that lack adequate treatment capacity cannot be avoided and partial infiltration BMPs are not feasible. Select "No" to Criteria 3 Result.	
Criteria 3 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 risk of groundwater contamination that cannot be reasonably mitigated to an acceptable level? ◉ Yes; Continue to Part 2, Criteria 4. ○ No; Skip to Part 2 Result.	
Summarize findings and basis. Documentation should focus on mapped soil types and contaminated site locations.	

Categorization of Infiltration Feasibility Condition based on Groundwater and Water Balance Conditions	Worksheet C.4-2: Form I-8B[2]

Criteria 4: Water Balance Screening

Additional studies. In the event that water balance effects are used to reject partial infiltration (anticipated to be rare), a qualified professional must provide an analysis of the incremental effects of partial infiltration BMPs on the water balance compared to incidental infiltration under a no infiltration scenario (e.g. precipitation, irrigation, etc.).

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 causing potential water balance issues such as change of seasonality of ephemeral streams?

◉ Yes: Continue to Part 2 Result.

○ No: Continue to Part 2 Result.

Summarize potential water balance effects. Documentation should focus on mapping and soil data regarding proximity to ephemeral streams and groundwater depth.

The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be at least 10 feet for infiltration to be allowed. CWE encountered groundwater at depths ranging from approximately 8 to 13 feet at the subject property Partial infiltration would be feasible within the alluvium at an elevation of at least 14 feet MSL.

Part 2 – Partial Infiltration Groundwater and Water Balance Screening Result[4]	Result
If answers to Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration based on groundwater and water balance conditions. If answer to Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site. The feasibility screening category is No Infiltration based on groundwater or water balance condition.	◉ Partial Infiltration Condition ○ No Infiltration Condition

[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.

Attachment 1e

DMA 2 - BMP 2 (BIO-FILTRATION)

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.52	inches
2	Area tributary to BMP (s)	A=	10.85	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.71	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=	14,541	cubic-feet

BMP 2 (DMA 2) - C runoff Factor Calculations:

C roofs = 0.90

C landscape = 0.10

Total Area = 472,658 sf

Pervious Area = 111,331 sf

Impervious Area = 361,326 sf

Weighted Area = (111,331 x 0.10) + (361,326 x 0.90) = 336,327 sf

C = weighted area / total area

C = 336,327 / 427,658

C = 0.71

DMA 2 - BMP 2 (BIO-FILTRATION)

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

Sizing Method for Pollutant Removal Criteria		Worksheet B.5-1	
1	Area draining to the BMP	472,658	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.71	
3	85th percentile 24-hour rainfall depth	0.52	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	14,541	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	25	inches
7	Aggregate storage (also add ASTM No 8 stone) above underdrain invert (12 inches typical) – use 0 inches if the aggregate is not over the entire bottom surface area	12	inches
8	Aggregate storage below underdrain invert (3 inches minimum) – use 0 inches if the aggregate is not over the entire bottom surface area	3	inches
9	Freely drained pore storage of the media	0.2	in/in
10	Porosity of aggregate storage	0.4	in/in
11	Media filtration rate to be used for sizing (maximum filtration rate of 5 in/hr. with no outlet control; if the filtration rate is controlled by the outlet use the outlet controlled rate (includes infiltration into the soil and flow rate through the outlet structure) which will be less than 5 in/hr.)	0.19	in/hr.
Baseline Calculations			
12	Allowable routing time for sizing	6	hours
13	Depth filtered during storm [Line 11 x Line 12]	1.14	inches
14	Depth of Detention Storage [Line 5 + (Line 6 x Line 9) + (Line 7 x Line 10) + (Line 8 x Line 10)]	23	inches
15	Total Depth Treated [Line 13 + Line 14]	24.14	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	21,813	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	10,843	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]	10,907	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	7,699	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]	10,068	sq. ft.
22	Footprint of the BMP = Maximum (Minimum (Line 17, Line 19), Line 21)	10,843	sq. ft.
23	Provided BMP Footprint	11,220	sq. ft.
24	Is Line 23 ≥ Line 22? If Yes, then footprint criterion is met. If No, increase the footprint of the BMP.	☑ Yes	☐ No

The City of SAN DIEGO	Project Name	Bella Mar Apartments	
	BMP ID	BMP 2 (Biofiltration Basin)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	
1	Area draining to the BMP	472658	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.71	
3	85th percentile 24-hour rainfall depth	0.52	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	14542	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 or enter 0.05	0.19	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.095	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%	22.5	%
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	0.165	
10	Target volume retention [Line 9 x Line 4]	2399	cu. ft.

The City of SAN DIEGO	Project Name	Bella Mar Apartments
	BMP ID	BMP 2 (Biofiltration Basin)

Volume Retention for No Infiltration Condition			Worksheet B.5-6	
1	Area draining to the biofiltration BMP	472658		sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.71		
3	Effective impervious area draining to the BMP [Line 1 x Line 2]	335587		sq. ft.
4	Required area for Evapotranspiration [Line 3 x 0.03]	10068		sq. ft.
5	Biofiltration BMP Footprint	11220		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.)		11220				
7	Impervious area draining to the landscape area (sq. ft.)		361326				
8	Impervious to Pervious Area ratio [Line 7/Line 6]		32.20	0.00	0.00	0.00	0.00
9	Effective Credit Area If (Line 8 >1.5, Line 6, Line 7/1.5]		11220	0	0	0	0

10	Sum of Landscape area [sum of Line 9 Id's 1 to 5]	11220		sq. ft.
11	Provided footprint for evapotranspiration [Line 5 + Line 10]	22440		sq. ft.

Volume Retention Performance Standard

12	Is Line 11 ≥ Line 4?	Volume Retention Performance Standard is Met		
13	Fraction of the performance standard met through the BMP footprint and/or landscaping [Line 11/Line 4]	2.23		
14	Target Volume Retention [Line 10 from Worksheet B.5.2]	2399		cu. ft.
15	Volume retention required from other site design BMPs [(1-Line 13) x Line 14]	-2950.77		cu. ft.

Site Design BMP

	Identification	Site Design Type	Credit	
16	1			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.		0	cu. ft.
17	Is Line 16 ≥ Line 15?	Volume Retention Performance Standard is Met		

DMA 4 - BMP 4 (PERMEABLE PAVERS)

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.52	inches
2	Area tributary to BMP (s)	A=	0.06	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.10	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=	21	cubic-feet

BMP 4 (DMA 4) - C runoff Factor Calculations:

C roofs = 0.90

C landscape = 0.10

Total Area = 2,791 sf

Pervious Area = 2,791 sf

Impervious Area = 0 sf

Weighted Area = (2,791 x 0.10) + (0 x 0.90) = 279 sf

C = weighted area / total area

C = 279 / 2,791

C = 0.10

PROVIDED BMP VOLUME:

(40%)x(A)x(3in)=(40%)x(1862sf)(.25ft)=186 CF >DCV

The City of SAN DIEGO	Project Name	Bella Mar Apartments	
	BMP ID	BMP 4 (Permeable Pavers)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	
1	Area draining to the BMP	2791	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.1	
3	85th percentile 24-hour rainfall depth	0.52	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	12	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 or enter 0.05	0.19	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.095	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%	22.5	%
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	0.165	
10	Target volume retention [Line 9 x Line 4]	2	cu. ft.

The City of SAN DIEGO	Project Name	Bella Mar Apartments					
	BMP ID	BMP 4 (Permeable Pavers)					
Volume Retention for No Infiltration Condition					**Worksheet B.5-6**		
1	Area draining to the biofiltration BMP				2791	sq. ft.	
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)				0.1		
3	Effective impervious area draining to the BMP [Line 1 x Line 2]				279	sq. ft.	
4	Required area for Evapotranspiration [Line 3 x 0.03]				8	sq. ft.	
5	Biofiltration BMP Footprint				186	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.)		929				
7	Impervious area draining to the landscape area (sq. ft.)		0				
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]				186	sq. ft.	
Volume Retention Performance Standard							
12	Is Line 11 ≥ Line 4?		Volume Retention Performance Standard is Met				
13	Fraction of the performance standard met through the BMP footprint and/or landscaping [Line 11/Line 4]				22.21		
14	Target Volume Retention [Line 10 from Worksheet B.5.2]				2	cu. ft.	
15	Volume retention required from other site design BMPs [(1-Line 13) x Line 14]				-42.32593365	cu. ft.	
Site Design BMP							
	Identification	**Site Design Type**			**Credit**		
16	**1**					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.				0	cu. ft.	
17	Is Line 16 ≥ Line 15?		Volume Retention Performance Standard is Met				

DMA 7 - BMP 7 (MWS UNIT)

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.52	inches
2	Area tributary to BMP (s)	A=	0.26	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=	409	cubic-feet

BMP 7 (DMA 7) - C runoff Factor Calculations:

C roofs = 0.90

C landscape = 0.10

Total Area = 11,296 sf

Pervious Area = 915 sf

Impervious Area = 10,381 sf

Weighted Area = (915 x 0.10) + (10,381 x 0.90) = 9,435 sf

C = weighted area / total area

C = 9,435 / 11,296

C = 0.84

DMA 7 - BMP 7 (MWS UNIT)

Worksheet B.6-1: Flow-Thru Design Flows

Flow-thru Design Flows		Worksheet B.6-1		
1	DCV	DCV	409	cubic-feet
2	DCV retained	$DCV_{retained}$	0	cubic-feet
3	DCV biofiltered	$DCV_{biofiltered}$	0	cubic-feet
4	DCV requiring flow-thru (Line 1 – Line 2 – 0.67*Line 3)	$DCV_{flow-thru}$	409	cubic-feet
5	Adjustment factor (Line 4 / Line 1)	AF=	1.5	unitless
6	Design rainfall intensity	i=	0.20	in/hr.
7	Area tributary to BMP (s)	A=	0.26	acres
8	Area-weighted runoff factor (estimate using Appendix B.2)	C=	0.84	unitless
9	Calculate Flow Rate = AF x (C x i x A)	Q=	0.065	cfs

1. Adjustment factor shall be estimated considering only retention and biofiltration BMPs located upstream of flow-thru BMPs. That is, if the flow-thru BMP is upstream of the project's retention and biofiltration BMPs then the flow-thru BMP shall be sized using an adjustment factor of 1.
2. Volume based (e.g., dry extended detention basin) flow-thru treatment control BMPs shall be sized to the volume in Line 4 and flow based (e.g., vegetated swales) shall be sized to flow rate in Line 9. Sand filter and media filter can be designed either by volume in Line 4 or flow rate in Line 9.
3. Proprietary BMPs, if used, shall provide certified treatment capacity equal to or greater than the calculated flow rate in Line 9; certified treatment capacity per unit shall be consistent with third party certifications.

The City of SAN DIEGO	Project Name	Bella Mar Apartments	
	BMP ID	BMP 7 (MWS Unit)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	
1	Area draining to the BMP	11296	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.84	
3	85th percentile 24-hour rainfall depth	0.52	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	411	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 or enter 0.05	0.19	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.095	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%	22.5	%
9	Fraction of DCV to be retained (Figure B.5-3) When Line 8 > 8% = 0.0000013 x Line 8^3 - 0.000057 x Line 8^2 + 0.0086 x Line 8 - 0.014 When Line 8 ≤ 8% = 0.023	0.165	
10	Target volume retention [Line 9 x Line 4]	68	cu. ft.

The City of SAN DIEGO	Project Name	Bella Mar Apartments	
	BMP ID	BMP 7 (MWS Unit)	
Volume Retention From Amended Soils		Worksheet B.5-7	
1	Impervious area draining to the pervious area	2295	sq. ft.
2	Pervious area (must meet the requirements in SD-B and SD-F Fact Sheets)	349	sq. ft.
3	Dispersion Ratio [Line 1/Line 2] Note: This worksheet is not applicable when Line 3 > 50 or Line 3 < 0.25	6.58	
4	Adjusted runoff factor [(Line 1 * 0.9 + Line 2 * 0.1) / (Line 1 + Line 2)]	0.79	
5	85th percentile 24-hour rainfall depth	0.52	inches
6	Design capture volume [(Line 1 + Line 2) x Line 4 x (Line 5/12)]	91	cu. ft.
7	Amendment Depth (Choose from 3", 6", 9", 12", 15" and 18")	18	inches
8	Storage [(porosity – field capacity) + 0.5 * (field capacity – wilting point)]	0.25	in./in.
9	Pervious Storage [Line 2 * (Line 7/12) * Line 8]	131	cu. ft.
10	Fraction of DCV [Line 9 / Line 6]	1.44	
11	Measured Infiltration Rate 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 or enter 0.05	0.19	in/hr.
12	Factor of Safety	2	
13	Reliable Infiltration Rate [Line 11/Line 12]	0.095	in/hr.
14	Dispersion Credit (Based on Figures B.5.6 to B.5.11; Line 10 and Line 13)	0.698	
15	Volume retention due to amendment [Line 1 * (Line 5/12) * Line 14]	69	cu. ft.

DMA 8 - BMP 8 (PERMEABLE PAVERS)

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.52	inches
2	Area tributary to BMP (s)	A=	0.01	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.10	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=	6	cubic-feet

BMP 8 (DMA 8) - C runoff Factor Calculations:

C roofs = 0.90

C landscape = 0.10

Total Area = 619 sf

Pervious Area = 619 sf

Impervious Area = 0 sf

Weighted Area = (619 x 0.10) + (0 x 0.90) = 61.9 sf

C = weighted area / total area

C = 61.9 / 619

C = 0.10

PROVIDED BMP VOLUME:

(40%)x(A)x(3in)=(40%)x(619sf)(.25ft)=62 CF >DCV

The City of SAN DIEGO	Project Name	Bella Mar Apartments	
	BMP ID	BMP 8 (Permeable Pavers)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	
1	Area draining to the BMP	619	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.1	
3	85th percentile 24-hour rainfall depth	0.52	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	3	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 or enter 0.05	0.19	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.095	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%	22.5	%
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	0.165	
10	Target volume retention [Line 9 x Line 4]	0	cu. ft.

The City of SAN DIEGO	Project Name	Bella Mar Apartments					
	BMP ID	BMP 8 (Permeable Pavers)					
Volume Retention for No Infiltration Condition					**Worksheet B.5-6**		
1	Area draining to the biofiltration BMP					619	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)					0.1	
3	Effective impervious area draining to the BMP [Line 1 x Line 2]					62	sq. ft.
4	Required area for Evapotranspiration [Line 3 x 0.03]					2	sq. ft.
5	Biofiltration BMP Footprint					619	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]					619	sq. ft.
Volume Retention Performance Standard							
12	Is Line 11 ≥ Line 4?		Volume Retention Performance Standard is Met				
13	Fraction of the performance standard met through the BMP footprint and/or landscaping [Line 11/Line 4]					333.33	
14	Target Volume Retention [Line 10 from Worksheet B.5.2]					0	cu. ft.
15	Volume retention required from other site design BMPs [(1-Line 13) x Line 14]					-147.0842731	cu. ft.
Site Design BMP							
	Identification	**Site Design Type**				**Credit**	
16	**1**						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.					0	cu. ft.
17	Is Line 16 ≥ Line 15?		Volume Retention Performance Standard is Met				

HOLLISTER STREET - GREEN STREET EXEMPTION

BMP Applicability and Selection for Green Street Exemption	Form J-1
Project Identification	
Project Name: BELLA MAR	
Permit Application Number:	Date:
Project Characterization and Selection Synopsis	
The purpose of this form is to guide the selection of BMPs, given project specific constraints to meet the Green Streets exemption as defined in Appendix J.2 of the BMP Design Manual. In order to qualify for a PDP exemption, the project must incorporate all applicable Green Street BMP elements described in Appendix J.2, based on the applicability guidance provided in Appendix J.2. Complete the sections below providing detailed justification for each selection.	
Step 1: Does this project include retrofitting or redevelopment of an existing alley, street, or roadway criteria? Exemptions do not apply for projects that construct new alleys, streets, or roadways. See Appendix J for additional guidance on distinguishing between redevelopment of a street and new development. ☑ Yes ☐ No (if No is selected, the Green Street exemption is not applicable)	
Provide a brief overview of the project, key details, and site-specific opportunities and constraints: The project improvements include street improvements, new storm drain, buildings, hardscape, and landscape.	
Step 2: Complete the BMP-specific applicability checklists on the following pages and attach them to this form. Complete forms for all BMPs, including those that were used and those that were not used.	
Step 3: Summarize the BMP(s) that were selected through the guidance process (Select all that apply):	

BMP Type	Applicable?	Used?	Summary of justification for Inclusion or Finding of Non-applicability
Vegetated Swales	☐	☐	inadequate width, not applicable.
Sidewalk Planters	☑	☑	Portion of proposed planter adjacent to our project to include a sidewalk planter
Curb Extensions	☐	☐	Not applicable (inadequate width)
Permeable Surfaces	☑	☑	parkway planter and 4' bench along Eastern edge of street to be a non-erodible permeable strip
Green Gutters	☐	☐	not applicable
Rain Gardens	☐	☐	Not applicable, inadequate width
Trees	☑	☑	Trees to be planted along property frontage
Other_____	☐	☐	

Form J-1 Page 2 of 8: Vegetated Swale

Brief Description: Vegetated Swales are shallow, open channels that are designed to remove storm water pollutants by physically straining/filtering runoff through vegetation in the channel.

Site Type (Check all that apply):	Street Type	Rating[1]	Present in Project?
	Residential Streets	◉	☐
	Commercial Street/ Business District	○	☐
	Collector Street	◉	☑
	Arterial and Boulevard	◉	☐
	Alleys	○	☐
	Parking Areas	◉	☐
Key Opportunities for Vegetated Swales (Check all that apply):	Parkway strips		☑
	Medians		☐
	Long, mostly continuous space		☑
	Other (must justify below)		☐
Site-Specific Factors (Check all that apply):	**Favorable Conditions for Vegetated Swales**		
	Slope > 1% and <3%		☑
	Conveying run-on to a site		☐
	Infiltration is partially feasible or not feasible		☐
	Long continuous segments available		☑
	More parkway width		☑
	Unfavorable Conditions for Vegetated Swales		
	Available width is < 8 feet		☑
	Frequent driveway interruption		☐
	ROW width too limited		☐

Summary of Findings:

Were Vegetated Swales determined to be applicable as part of the Green Streets BMP plan? ☐ Yes ☑ No	If yes, were they used? ☐ Yes ☐ No

Provide discussion/justifications for selections and decisions above:
the parkway width is not wide enough for a vegetated swale.

[1] ● High applicability within this category, however may still be limited by site-specific factors
◉ Generally applicable in this category; largely dependent on site-specific factors
○ Limited applicability within this category; may still be applicable in some cases; should be considered

Form J-1 Page 3 of 8: Sidewalk Planters

Brief Description: A planter imbedded in the sidewalk designed to manage storm water runoff from the adjacent roadway and sidewalk.

	Street Type	Rating[2]	Present in Project?
Site Type (Check all that apply):	Residential Streets	◉	☐
	Commercial Street/ Business District	◉	☐
	Collector Street	●	☑
	Arterial and Boulevard	●	☐
	Alleys	○	☐
	Parking Areas	◉	☐
Key Opportunities for Sidewalk Planters (Check all that apply):	Parkway strips		☑
	Medians		☐
	Between driveways		☐
	Other (must justify below)		☐
Site-Specific Factors (Check all that apply):	**Favorable Conditions for Sidewalk Planters**		
	Slope <4%		☑
	Wide sidewalks		☐
	More parkway width		☐
	Unfavorable Conditions for Sidewalk Planters		
	Conflicts with car egress		☐
	ROW width too limited		☐

Summary of Findings:

Were Sidewalk Planters determined to be applicable as part of the Green Streets BMP plan?	If yes, were they used?
☑ Yes ☐ No	☑ Yes ☐ No

Provide discussion/justifications for selections and decisions above:

drainage from sidewalk will flow over sidewalk planters before entering curb and gutter.

[2] ● High applicability within this category, however may still be limited by site-specific factors
◉ Generally applicable in this category; largely dependent on site-specific factors
○ Limited applicability within this category; may still be applicable in some cases; should be considered

Form J-1 Page 4 of 8: Curb Extensions

Brief Description: Curb extensions expand the edge of the sidewalk into the roadway or parking area and allow storm water runoff to collect and infiltrate through a detention area of porous media.

	Street Type	Rating[3]	Present in Project?
Site Type (Check all that apply):	Residential Streets	●	☐
	Commercial Street/ Business District	●	☐
	Collector Street	◉	☑
	Arterial and Boulevard	◉	☐
	Alleys	○	☐
	Parking Areas	◉	☐
Key Opportunities for Curb Extensions (Check all that apply):	Intersections		☐
	Parking area		☐
	Other (must justify below)		☐
Site-Specific Factors (Check all that apply):	**Favorable Conditions for Curb Extensions**		
	Slope <4%		☑
	Traffic calming needed		☐
	Unfavorable Conditions for Curb Extensions		
	Conflicts with bike lanes		☑
	Site distance issues at intersection		☐

Summary of Findings:

Were Curb Extensions determined to be applicable as part of the Green Streets BMP plan?	If yes, were they used?
☐ Yes ☑ No	☐ Yes ☑ No

Provide discussion/justifications for selections and decisions above:
inadequate right of way width. curb extensions would obstruct bike lane.

[3] ● High applicability within this category, however may still be limited by site-specific factors
◉ Generally applicable in this category; largely dependent on site-specific factors
○ Limited applicability within this category; may still be applicable in some cases; should be considered

Form J-1 Page 5 of 8: Permeable Surfaces

Brief Description: Permeable surfaces are pavement that allows for percolation through void spaces into subsurface layers.

Site Type (Check all that apply):	Street Type	Rating[4]	Present in Project?
	Residential Streets	●	☐
	Commercial Street/ Business District	●	☐
	Collector Street	◉	☑
	Arterial and Boulevard	◉	☐
	Alleys	●	☐
	Parking Areas	◉	☐
Key Opportunities for Permeable Surfaces (Check all that apply):	Sidewalks		☑
	Parking strips		☑
	Shoulders		☑
	Low traffic roadways		☐
	Other (must justify below)		☐
Site-Specific Factors (Check all that apply):	**Favorable Conditions for Permeable Surfaces**		
	Slope < 2-3%		☑
	Conveying limited run-on to a site		☑
	Low traffic area		☐
	Unfavorable Conditions for Permeable Surfaces		
	High traffic area		☐
	Run-on has high sediment load		☐

Summary of Findings:

Were Permeable Surfaces determined to be applicable as part of the Green Streets BMP plan?	If yes, were they used?
☑ Yes ☐ No	☑ Yes ☐ No

Provide discussion/justifications for selections and decisions above:

4' wide bench along Eastern edge of street to have non-erodible strip of open graded rock.

[4] ● High applicability within this category, however may still be limited by site-specific factors
◉ Generally applicable in this category; largely dependent on site-specific factors
○ Limited applicability within this category; may still be applicable in some cases; should be considered

Form J-1 Page 6 of 8: Green Gutters

Brief Description: Green Gutters are shallow and narrow strips of landscaping in a typical curb and gutter location with a lower elevation than the street gutter elevation to allow capture of storm water from the sidewalk and street.

Site Type (Check all that apply):	Street Type	Rating[5]	Present in Project?
	Residential Streets	○	☐
	Commercial Street/ Business District	◉	☐
	Collector Street	●	☑
	Arterial and Boulevard	●	☐
	Alleys	◉	☐
	Parking Areas	○	☐
Key Opportunities for Green Gutters (Check all that apply):	Parkway strips		☑
	Medians		☐
	Long, mostly continuous space		☑
	Other (must justify below)		☐
Site-Specific Factors (Check all that apply):	Favorable Conditions for Green Gutters		
	Slope > 1% and <3%		☑
	Conveying run-on to a site		☐
	Infiltration is partially feasible or not feasible		☐
	Long continuous segments available		☑
	Narrower spaces (as little as 2 to 3 feet)		☐
	Unfavorable Conditions for Green Gutters		
	Frequent driveway interruption		☑
	ROW width too limited		☑

Summary of Findings:

Were Green Gutters determined to be applicable as part of the Green Streets BMP plan? ☑ Yes ☐ No	If yes, were they used? ☐ Yes ☑ No

Provide discussion/justifications for selections and decisions above:
frequent driveway interruption preclude the use of this BMP

[5] ● High applicability within this category, however may still be limited by site-specific factors
◉ Generally applicable in this category; largely dependent on site-specific factors
○ Limited applicability within this category; may still be applicable in some cases; should be considered

Form J-1 Page 7 of 8: Rain Gardens

Brief Description: Rain Gardens are shallow detention basins with vegetation that temporarily store water to allow for infiltration of the stored volume. Rain Gardens could be bioretention or biofiltration with partial retention or a biofiltration BMP.

Site Type (Check all that apply):	Street Type	Rating[6]	Present in Project?
	Residential Streets	◉	☐
	Commercial Street/ Business District	◉	☐
	Collector Street	◉	☑
	Arterial and Boulevard	◉	☐
	Alleys	○	☐
	Parking Areas	●	☐
Key Opportunities for Rain Gardens (Check all that apply):	Irregularly shaped areas in ROW		☐
	Broad and flat areas		☐
	Other (must justify below)		☐
Site-Specific Factors (Check all that apply):	Favorable Conditions for Rain Gardens		
	Slope <2%		☑
	Infiltration is partially feasible or not feasible		☐
	Large area available		
	Unfavorable Conditions for Rain Gardens		
	Slope > 2%		☐
	ROW too limited		☑

Summary of Findings:

Were Rain Gardens determined to be applicable as part of the Green Streets BMP plan? ☐ Yes ☑ No	If yes, were they used? ☐ Yes ☑ No

Provide discussion/justifications for selections and decisions above:
inadequate right of way width.

[6] ● High applicability within this category, however may still be limited by site-specific factors
◉ Generally applicable in this category; largely dependent on site-specific factors
○ Limited applicability within this category; may still be applicable in some cases; should be considered

Form J-1 Page 8 of 8: Trees

Brief Description: Trees planted in the sidewalk right-of-way provide rainfall interception and infiltration benefits and typically supplement other storm water management tools.

Site Type (Check all that apply):	Street Type	Rating[7]	Present in Project?
	Residential Streets	●	☐
	Commercial Street/ Business District	◉	☐
	Collector Street	◉	☑
	Arterial and Boulevard	◉	☐
	Alleys	◉	☐
	Parking Areas	●	☐
Key Opportunities for Trees (Check all that apply):	Parkway strips		☑
	Medians		☐
	Irregularly shaped areas		☐
	Extra ROW on back side of sidewalk		☐
	Other (must justify below)		☐
Site-Specific Factors (Check all that apply):	Favorable Conditions for Trees		
	Located outside of clear zone		☑
	Infiltration is feasible		☑
	ROW not limiting		
	Unfavorable Conditions for Trees		
	Limited space for root growth		☐
	Clear zone issues		☐

Summary of Findings:

Were Trees determined to be applicable as part of the Green Streets BMP plan? ☑ Yes ☐ No	If yes, were they used? ☑ Yes ☐ No

Provide discussion/justifications for selections and decisions above:
Trees are proposed in the parkway.

[7] ● High applicability within this category, however may still be limited by site-specific factors
◉ Generally applicable in this category; largely dependent on site-specific factors
○ Limited applicability within this category; may still be applicable in some cases; should be considered

DMA 2 - BMP 2 (BIO-FILTRATION)

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

Sizing Method for Pollutant Removal Criteria		Worksheet B.5-1	
1	Area draining to the BMP	472,658	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.71	
3	85th percentile 24-hour rainfall depth	0.52	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	14,541	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	25	inches
7	Aggregate storage (also add ASTM No 8 stone) above underdrain invert (12 inches typical) – use 0 inches if the aggregate is not over the entire bottom surface area	12	inches
8	Aggregate storage below underdrain invert (3 inches minimum) – use 0 inches if the aggregate is not over the entire bottom surface area	3	inches
9	Freely drained pore storage of the media	0.2	in/in
10	Porosity of aggregate storage	0.4	in/in
11	Media filtration rate to be used for sizing (maximum filtration rate of 5 in/hr. with no outlet control; if the filtration rate is controlled by the outlet use the outlet controlled rate (includes infiltration into the soil and flow rate through the outlet structure) which will be less than 5 in/hr.)	0.19	in/hr.
Baseline Calculations			
12	Allowable routing time for sizing	6	hours
13	Depth filtered during storm [Line 11 x Line 12]	1.14	inches
14	Depth of Detention Storage [Line 5 + (Line 6 x Line 9) + (Line 7 x Line 10) + (Line 8 x Line 10)]	23	inches
15	Total Depth Treated [Line 13 + Line 14]	24.14	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	21,813	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	10,843	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]	10,907	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	7,699	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]	10,068	sq. ft.
22	Footprint of the BMP = Maximum (Minimum (Line 17, Line 19), Line 21)	10,843	sq. ft.
23	Provided BMP Footprint	11,220	sq. ft.
24	Is Line 23 ≥ Line 22? If Yes, then footprint criterion is met. If No, increase the footprint of the BMP.	☑ Yes	☐ No

DMA 4 - BMP 4 (PERMEABLE PAVERS)

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.52	inches
2	Area tributary to BMP (s)	A=	0.06	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.10	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=	21	cubic-feet

BMP 4 (DMA 4) - C runoff Factor Calculations:

C roofs = 0.90

C landscape = 0.10

Total Area = 2,791 sf

Pervious Area = 2,791 sf

Impervious Area = 0 sf

Weighted Area = (2,791 x 0.10) + (0 x 0.90) = 279 sf

C = weighted area / total area

C = 279 / 2,791

C = 0.10

PROVIDED BMP VOLUME:

(40%)x(A)x(3in)=(40%)x(1862sf)(.25ft)=186 CF >DCV THEREFORE SELF-RETAINING

DMA 7 - BMP 7 (MWS UNIT)

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.52	inches
2	Area tributary to BMP (s)	A=	0.26	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=	409	cubic-feet

BMP 7 (DMA 7) - C runoff Factor Calculations:

C roofs = 0.90

C landscape = 0.10

Total Area = 11,296 sf

Pervious Area = 915 sf

Impervious Area = 10,381 sf

Weighted Area = (915 x 0.10) + (10,381 x 0.90) = 9,435 sf

C = weighted area / total area

C = 9,435 / 11,296

C = 0.84

DMA 7 - BMP 7 (MWS UNIT)

Worksheet B.6-1: Flow-Thru Design Flows

Flow-thru Design Flows		Worksheet B.6-1		
1	DCV	DCV	409	cubic-feet
2	DCV retained	$DCV_{retained}$	0	cubic-feet
3	DCV biofiltered	$DCV_{biofiltered}$	0	cubic-feet
4	DCV requiring flow-thru (Line 1 – Line 2 – 0.67*Line 3)	$DCV_{flow-thru}$	409	cubic-feet
5	Adjustment factor (Line 4 / Line 1)	AF=	1.5 *	unitless
6	Design rainfall intensity	i=	0.20	in/hr.
7	Area tributary to BMP (s)	A=	0.26	acres
8	Area-weighted runoff factor (estimate using Appendix B.2)	C=	0.84	unitless
9	Calculate Flow Rate = AF x (C x i x A)	Q=	0.065	cfs

1. Adjustment factor shall be estimated considering only retention and biofiltration BMPs located upstream of flow-thru BMPs. That is, if the flow-thru BMP is upstream of the project's retention and biofiltration BMPs then the flow-thru BMP shall be sized using an adjustment factor of 1.
2. Volume based (e.g., dry extended detention basin) flow-thru treatment control BMPs shall be sized to the volume in Line 4 and flow based (e.g., vegetated swales) shall be sized to flow rate in Line 9. Sand filter and media filter can be designed either by volume in Line 4 or flow rate in Line 9.
3. Proprietary BMPs, if used, shall provide certified treatment capacity equal to or greater than the calculated flow rate in Line 9; certified treatment capacity per unit shall be consistent with third party certifications.

* Use 1.5 for proprietary biofiltration

DMA 8 - BMP 8 (PERMEABLE PAVERS)

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	0.52	inches
2	Area tributary to BMP (s)	A=	0.01	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.10	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=	6	cubic-feet

BMP 8 (DMA 8) - C runoff Factor Calculations:

C roofs = 0.90

C landscape = 0.10

Total Area = 619 sf

Pervious Area = 619 sf

Impervious Area = 0 sf

Weighted Area = (619 x 0.10) + (0 x 0.90) = 61.9 sf

C = weighted area / total area

C = 61.9 / 619

C = 0.10

PROVIDED BMP VOLUME:

(40%)x(A)x(3in)=(40%)x(619sf)(.25ft)=62 CF >DCV

THEREFORE SELF-RETAINING

Project Name: Bella Mar Apartments

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	2.31	0	0	A	0.10	436	-	SD-B	2
DMA 2	10.85	361,326	76	A	0.71	14575	BMP 2	BF-1	1
DMA 3	0.39	0	0	A	0.10	73	-	SD-B	1
DMA 4	0.06	0	0	A	0.10	21	BMP 4	INF-3	1
DMA 5	0.11	1,375	29	A	0.33	69	-	SD-B	1
DMA 6	0.06	289	11	A	0.18	22	-	SD-B	1
DMA 7	0.26	10,381	92	A	0.84	409	BMP 7	FT-5	2
DMA 8	0.01	0	0	A	0.10	4	-	SD-B	2
DMA 9	0.01	0	0	A	0.10	1	-	SD-B	2
Summary of DMA Information (Must match project description and SWQMP Narrative)									
No. of DMAs	Total DMA Area (acres)	Total Impervious Area (acres)	% Imp		Area Weighted Runoff Coefficient	Total DCV (cubic feet)	Total Area Treated (acres)		No. of POCs
9	14.06	8.57	61		0.69	18,354	11,468		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

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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.

From: Gefrom, Walter <WGefrom@sandiego.gov>
Sent: Thursday, December 13, 2018 5:06 PM
To: Bryan Smith
Cc: Vera, Karen; Martin Jones; Mike Hoe
Subject: RE: Bella Mar HMP - PTS 598995
Attachments: Bella Mar Hydromod Exempt Memo.pdf

Categories: Filed by Newforma

Bryan,

My review of this and comments apply to the project once it comes through my group for Ministerial review/approval. Currently, it appears to be under review/approval through preliminary/discretionary. Also, keep in mind that any revisions to the State storm water permit before you acquire a grading/building permit may require me to void the memo.

Based on the exhibit and supporting documentation provided, the discharge of any storm water treated flows will not require HMP. You will need to revise the memo per my markups based on the updated Storm Water Standards Manual and also update the last page (Manual excerpt) with the newest language. Mike Hoe won't need to sign the memo if he sends the memo directly to me through e-mail instead of a cc. Or, he may acknowledge that he's seen it.

Thanks,

Walter C. Gefrom, PE, QSD, CFM
Deputy City Engineer

Development Services Department - Engineering Division
1222 First Avenue | San Diego | CA | 92101
MS 501

Visit OpenDSD for project info: https://www.sandiego.gov/development-services/opendsd

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From: Bryan Smith [mailto:bsmith@fuscoe.com]
Sent: Tuesday, December 11, 2018 7:59 AM
To: Gefrom, Walter <WGefrom@sandiego.gov>
Cc: Vera, Karen <KVera@sandiego.gov>
Subject: RE: Bella Mar HMP - PTS 598995

Hi Walter,

Just wanted to follow up on this. Can you please review and get back to me when you have the chance?

Thanks,

BRYAN D. SMITH, PE | *Project Manager*

FUSCOE ENGINEERING, INC.
an employee owned company

full circle thinking®

858.554.1500

From: Bryan Smith
Sent: Friday, November 30, 2018 11:05 AM
To: 'Gefrom, Walter' <WGefrom@sandiego.gov>
Cc: 'Vera, Karen' <KVera@sandiego.gov>
Subject: Bella Mar HMP - PTS 598995

Walter,

Hope all is well with you. Over a year ago, we met and discussed this multifamily residential project in Otay Nestor at 408 Hollister. I don't expect that you recall the original meeting, but we discussed a possible HMP exemption for this site. The project went on hold for some time but has since been restarted. Most recently, we went through preliminary review and were assigned the above PTS number and also met with you to discuss stockpiling in the FEMA Floodplain, as you probably recall. The stockpiling idea has been put on hold but we are working toward an entitlement submittal for a Tentative Map.

Our discussion last year was based on the Hydromod exemption. The site discharges through a Caltrans culvert under the I-5 free and discharges to an unlined channel to the West. The unlined channel and culvert outlet is within the 10-year flood plain elevation associated with the Otay River (see attached Memo documenting this). According to our meeting, you believed this would qualify for an exemption from HMP requirements but you asked that we document it in a memo and send to you.

Please find the attached memo which we will include the SWQMP. If you could please take a quick review when you have the chance and confirm our understanding it would be much appreciated.

Best,

25 YEARS

BRYAN D. SMITH, PE | *Project Manager*
bsmith@fuscoe.com
CONNECT WITH ME

FUSCOE ENGINEERING, INC.
an employee owned company

6390 Greenwich Drive, Suite 170, San Diego, California 92122
858.554.1500 | fuscoe.com

IRVINE . **SAN DIEGO** . ONTARIO . LOS ANGELES . EL CENTRO . SAN RAMON . MISSION HILLS

Hydromodification Exemption Memo - Bella Mar

To: Walter Gefrom, P.E., City of San Diego DSD

From: Michael Hoe, P.E., Fuscoe Engineering, Inc.

Date: May 17, 2017, Revision Date: December 14, 2018

The subject property is located at 408 Hollister Street in the City of San Diego, County of San Diego. The project site is bordered by private properties to the North and South, Hollister Street on the East and the Interstate 5 Freeway on the West. See the attached project site exhibit on the following sheet. Stormwater runoff on the subject property flows from east to west and discharges into an existing 24" storm drain culvert which runs below the I-5 Interchange bridge. The runoff eventually discharges into the Otay River and ultimately into the San Diego Bay.

Per City of San Diego Storm Water Standards Section 1.6, the Otay River is classified as a hydromodification exempt body of water.

"*Designated exempt river reaches within City of San Diego jurisdiction include the Otay River downstream of Lower Otay Reservoir Dam (Savage Dam). To qualify as a direct discharge to this exempt river reach, the invert elevation of the direct discharge conveyance system (at the point of discharge to the exempt river reach) should be equal to or below the 10-year floodplain elevation. The City Engineer may require additional analysis of the potential for erosion between the outfall and the 10-year floodplain elevation*."

The flowline elevation at the outlet of the existing 24" storm drain culvert is 12.7' NGVD29 or 14.9' NAVD 88 (see conversion table on the next sheet) per Caltrans As-built Drawing Document Number A-0002600. See the attached as-built drawing for reference.

Based on the most recent Flood Insurance Study (revised May 16, 2012), the 10-year water surface elevation below the Interstate 5 Bridge at the storm drain outlet is 14.9' (NAVD 88). See the flood profile for the Otay River in the following attachments. The storm drain outfall elevation is the same elevation as the 10-year base flood elevation therefore, the project should be considered exempt from Hydromodification Management requirements.

Attachments:

1- Site Plan
2- Caltrans As-Built
3- Flood Insurance Study
4- BMP Manual Excerpt

ATTACHMENT 1
SITE PLAN



ATTACHMENT 2
CALTRANS AS-BUILT

DIST.	COUNTY	ROUTE	POST MILES—TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
11	SD	5	4.4/9.0	42	437

DISTRICT DESIGN ENGINEER Henry R. Obrecht REGISTERED CIVIL ENGINEER No. 15768
DATE APPROVED July 27, 1970

14.9' IE-OUT (NAVD 88)

16.2' IE-IN (NAVD 88)

L0
Δ = 04° 36' 28"
R = 10,000'
T = 402.32'
L = 804.21'

275+59.38 EC L0

Otay Riv Br
Br. No. 57-246R/L

Otay Riv Overflow Br
Br. No. 57-263R/L

R/W

S Dike

Graded "V" Gutter 2:1 SS

N06°54'00"W 285

2693.09'

Profile Grade

Ground Line

-0.38%

NOTE
Profile Grade is 15' Lt & Rt of ℄ L0

U.S.C & G.S. Datum

"AS BUILT"
CONTRACT 122404
YEAR COMPLETED 8-1-73

DRAINAGE DETAILS AND CONTOUR GRADING PLANS

Project Engineer	Date	Design Engineer	Date	Approval Recommended By	Date

AS BUILT PLANS
Contract No. 11-122404
Date Completed 07-70
Document No. 11-000260

FORM WH23A-PF

4

I HEREBY CERTIFY THAT THIS IS A TRUE AND ACCURATE COPY OF THE ABOVE DOCUMENT TAKEN UNDER MY DIRECTION AND CONTROL ON THIS DATE IN SACRAMENTO, CALIFORNIA PURSUANT TO AUTHORIZATION BY THE DIRECTOR OF TRANSPORTATION.

DATE 8-26-74 SIGNATURE [signature] TITLE [illegible]

SCALE 1"=50' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=20' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=10' or 1"=100' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=40' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM

ATTACHMENT 3 :
FLOOD INSURANCE
STUDY

CHARLES AVENUE

LOURET AVENUE

INTERSTATE HIGHWAY 5

INTERSTATE HIGHWAY 5

14.9' 10 YEAR FLOOD ELEVATION @ OUTLET

15.1' 10-YEAR FLOOD ELEVATION @ INLET

200'

ELEVATION IN FEET (NAVD)

30

20

10

0

2600 2800 3000 3200 3400 3600 3800 4000 4200 4400 4600 4800 5000 5200

STREAM DISTANCE IN FEET ABOVE CROSS SECTION A

D E F G

LEGEND

- 0.2% ANNUAL CHANCE FLOOD
- 1% ANNUAL CHANCE FLOOD
- 2% ANNUAL CHANCE FLOOD
- 10% ANNUAL CHANCE FLOOD
- STREAM BED
- CROSS SECTION LOCATION

FLOOD PROFILES

OTAY RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN DIEGO COUNTY, CA
(AND INCORPORATED AREAS)

259P

TABLE 12: FLOODING SOURCE DATUM SHIFT VALUES

Stream Name	Elevation (feet NAVD above NGVD)
Moosa Creek (North Branch)	+2.3
Moosa Creek (South Branch)	+2.3
Murphy Canyon Creek	+2.1
Murray Canyon Creek	+2.1
Nestor Creek	+2.1
North Avenue Tributary	+2.3
North Branch Poway Creek	+2.1
North Tributary to Santa Maria Creek	+2.2
Olive Creek	+2.4
Otay River	+2.2
Pala Mesa Creek	+2.2
Paradise Creek	+2.1
Paradise Creek – Valley Road Branch	+2.1
Pilgrim Creek	+2.3
Poggi Canyon Creek	+2.2
Pomerado Creek	+2.1
Poway Creek	+2.1
Rainbow Creek (Main Branch)	+2.3
Rainbow Creek (West Branch)	+2.3
Rattlesnake Creek	+2.1
Rattlesnake Creek Split Flow at Heritage Hills	+2.1
Rattlesnake Creek Split Flow at Midland Road	+2.1
Reidy Creek	+2.3
Reidy Creek Split Flow	+2.3
Rice Canyon Creek	+2.1
Rincon Avenue Tributary	+2.3
Rose Canyon Creek	+2.1
Samagutuma Creek	+2.4
San Clemente Canyon Creek	+2.1
San Diego Bay	+2.2
San Diego River	+2.1
San Dieguito River	+2.1
San Elijo Creek	+2.2
San Luis Rey River	+2.3
San Marcos Creek	+2.3
San Marcos Creek (Below Lake San Marcos)	+2.3
San Marcos Creek Highway 78 Split Flow	+2.3

- This exemption is subject to the following conditions:
  (a) A properly sized energy dissipation system must be provided in accordance with the City design standards to mitigate outlet discharge velocity from the direct discharge to the water storage reservoir or lake for the ultimate condition peak design flow of the direct discharge,
  (b) The invert elevation of the direct discharge conveyance system (at the point of discharge to the water storage reservoir or lake) should be equal to or below the lowest normal operating water surface elevation at the point of discharge, unless the outfall discharges to quay or other non-erodible shore protection. Normal operating water surface elevation may vary by season; contact the reservoir operator to determine the elevation. For cases in which the direct discharge conveyance system outlet invert elevation is above the lowest normal operating water surface elevation but below the reservoir spillway elevation, additional analysis is required to determine if energy dissipation should be extended between the conveyance system outlet and the elevation associated with the lowest normal operating water surface level.
- No exemption may be granted for conveyance system outlet invert elevations located above the reservoir spillway elevation.

- **Figure 1-2, Node 5** – As allowed by the MS4 Permit, projects discharging directly to an area identified as appropriate for an exemption in the WMAA for the watershed in which the project resides are exempt. Refer to the WMAA for any updates to exempt river reaches. Discharging directly refers to either a) existing underground storm drain systems; or b) conveyance channels whose bed and bank are concrete-lined all the way from the point of discharge to the designated area.
  - Designated exempt river reaches identified in the WMAA and approved by the RWQCB within City of San Diego jurisdiction:
    (a) San Dieguito River downstream of Lake Hodges
    (b) San Diego River downstream of confluence with San Vicente Creek
    (c) Sweetwater River downstream of Sweetwater Reservoir
    (d) Otay River downstream of Lower Otay Reservoir Dam
  - To qualify as a direct discharge to an exempt river reach:
    (a) A properly sized energy dissipation system must be provided to mitigate outlet discharge velocity from the direct discharge to the exempt river reach for the ultimate condition peak design flow of the direct discharge,
    (b) The invert elevation of the direct discharge conveyance system (at the point of discharge to the exempt river reach) should be equal to or below the 10-year floodplain elevation. Exceptions may be made at the discretion of the City Engineer, but shall never exceed the 100-year floodplain elevation. The City Engineer may require additional analysis of the potential for erosion between the outfall and the 10-year floodplain elevation.
  - No exemption may be granted for conveyance system outlet invert elevations located above the 100-year floodplain elevation.

General note regarding HMP: New outfalls shall meet requirements for energy dissipation size in the Drainage Design Manual regardless of the addition of hydromodification controls. Existing outfalls that are insufficient to accommodate additional flows from proposed upstream development projects

Project Name: Bella Mar Apartments

Indicate which Items are Included:

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

LEGEND

CRITICAL COURSE SEDIMENT YIELD AREAS



BELLA MAR

ATTACHMENT 2b - CRITICAL COURSE SEDIMENT YEILD AREAS EXHIBIT

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6390 Greenwich Dr., Suite 170, San Diego, California 92122
tel 858.554.1500 • fax 858.597.0335 • www.fuscoe.com

F:\PROJECTS\1621\001_SUPPORT FILES\REPORTS\SWQMP\ATTACHMENTS\CRITICAL COURSE SEDIMENT YIELD AREAS MAP EXHIBIT.DWG (02-20-2019 2:02:19PM) Plotted by: Brianne VanGorder

Project Name: Bella Mar Apartments

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)
- [x] 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).

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Attachment 3
Structural BMP Maintenance Information

This is the cover sheet for Attachment 3.

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Project Name: Bella Mar Apartments

Indicate which Items are Included:

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

WILL PROVIDE AT FINAL SUBMITTAL

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).

Attachment 4
Copy of Plan Sheets Showing Permanent Storm Water BMPs

This is the cover sheet for Attachment 4.

Project Name: Bella Mar Apartments

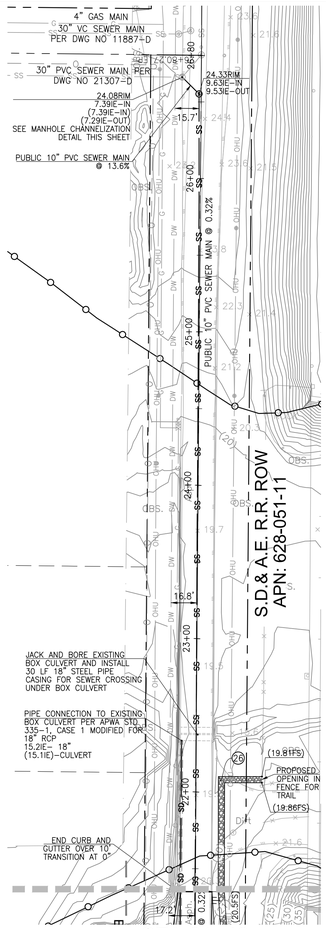
Use this checklist to ensure the required information has been included on the plans:

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.

Attachment 5
Drainage Report

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



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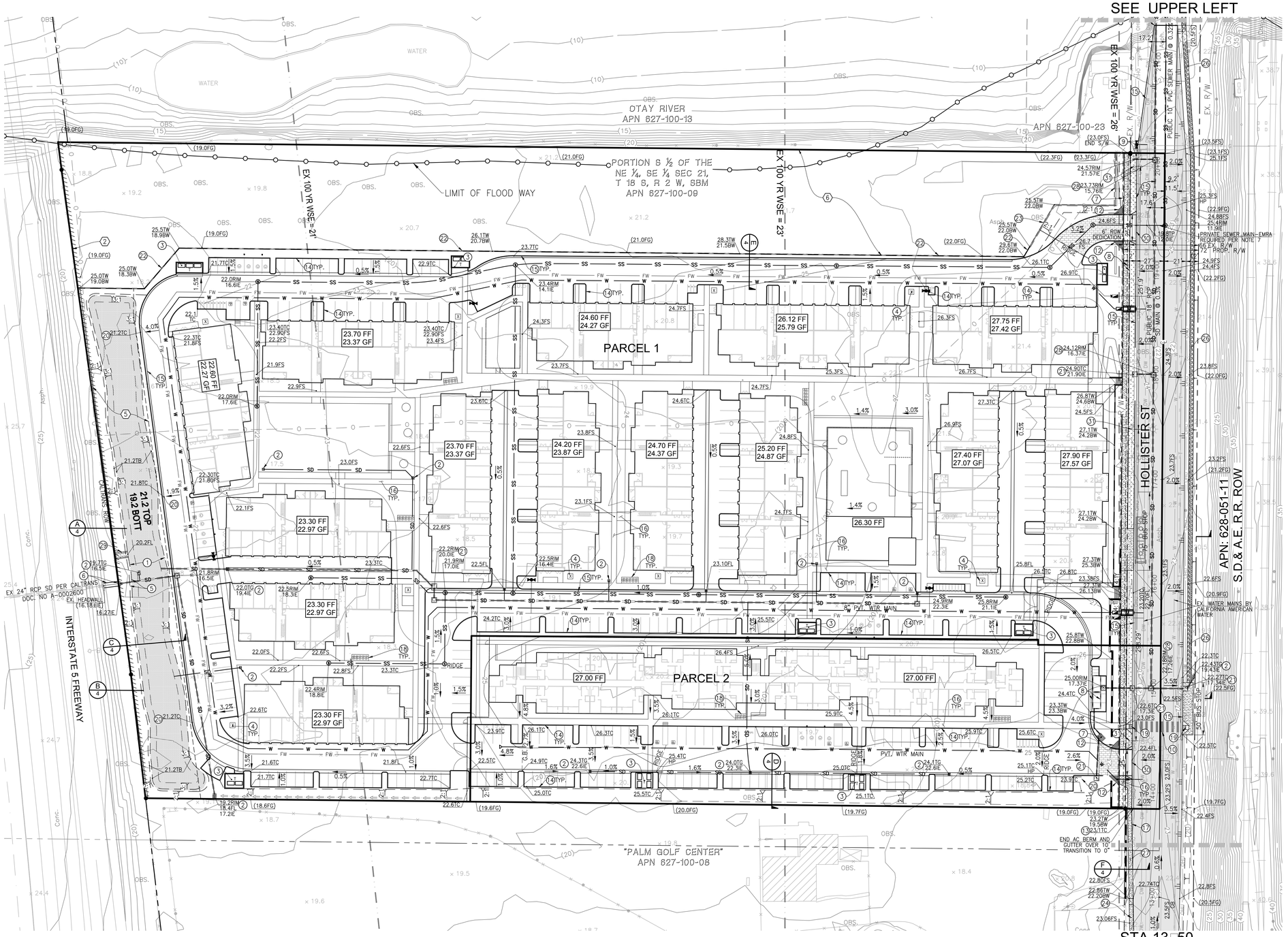
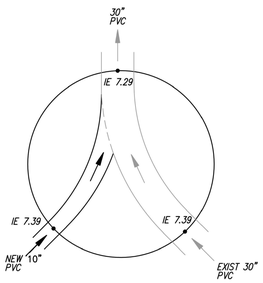
SEE RIGHT



STA 13□50
SEE SHEET 4 OF 5

CONSTRUCTION NOTES

1. BIOFILTRATION BASIN PER DETAIL C/4
2. CATCH BASIN PER RSD D-8
3. TRASH ENCLOSURE
4. ELECTRICAL TRANSFORMER
5. SPILLWAY PER DETAIL C/4
6. HEADWALL TO DISCHARGE TO 4.5' WIDE CONCRETE DRAINAGE DITCH TO EX. CULVERT (CALTRANS ENCROACHMENT)
7. PROPRIETARY MODULAR WETLAND BIOFILTRATION BMP
8. UNDERGROUND DETENTION VAULTS
9. RELOCATE EX. FH
10. MID BLOCK CROSSWALK PER SDM-116
12. VISIBILITY AREA TRIANGLES. NO OBSTRUCTION INCLUDING SOLID WALLS IN THE VISIBILITY AREA SHALL EXCEED 3FT IN HEIGHT
13. TRANSITION FROM C&G TO TEMP AC BERM
14. 6" CURB PER SDG-150
15. 6" CURB AND GUTTER PER SDG-151
16. 4" PCC SIDEWALK PER SDG-155
17. TEMPORARY 6" TYPE 'A' AC BERM PER G-05
18. BIKE RACKS PER LANDSCAPE PLANS
19. CURB RAMP TYPE 'B' PER SDG-133
20. 3' CURB OPENING
21. CURB INLET PER RSD D-1
22. RETAINING WALL WITH 3' FREESTANDING WALL AND FENCING PER DETAIL E/4
23. 3' FREESTANDING WALL AND FENCING PER LANDSCAPE ARCH PLANS
24. EXISTING 35' DRIVEWAY TO BE MODIFIED PER SDG-162
25. CURB RAMP TYPE 'C' PER G-29
26. 4' WIDE MULTI-USE DG PATH PER DETAIL H/4
27. TEMPORARY 4" AC SIDEWALK PER SDG-155
28. STORM DRAIN CLEANOUT PER RSD D-9
29. CONCRETE SPILLWAY (6' WIDE X 6" DEEP) FOR EMERGENCY/SECONDARY OVERFLOW
30. 25' DRIVEWAY PER SDG-161
31. PROPOSED STREET LIGHT INSTALLED PER STREET DESIGN MANUAL



MANHOLE RECHANNELIZATION DETAIL
NO SCALE

EASEMENT NOTES

2. ABUTTER'S RIGHTS OF INGRESS AND EGRESS TO OR FROM THE STREET, HIGHWAY, OR FREEWAY ABUTTING THE WESTERLY BOUNDARY OF SAID PROPERTY RECORDED IN BOOK 3918, PAGE 199 O.R. RECORDED JANUARY 3, 1951.
5. AN EASEMENT, OF VARIABLE WIDTH, GRANTED TO THE CITY OF SAN DIEGO FOR STORAGE OF FLOOD WATERS PER DOC. REC. SEP. 22, 2010 AS DOC. NO. 2010-0503414 O.R.
6. AN EASEMENT, OF VARIABLE WIDTH, GRANTED TO THE CITY OF SAN DIEGO FOR CONSERVATION PURPOSES WITH THE RIGHT OF INGRESS AND EGRESS PER DOC. REC. SEP. 22, 2010 AS DOC. NO. 2010-0503415 O.R.
7. A 5.0' EASEMENT GRANTED TO THE CITY OF SAN DIEGO FOR PUBLIC UTILITIES PER DOC. REC. SEP. 22, 2010 AS DOC. NO. 2010-0503416 O.R. TO BE VACATED.
8. AN IRREVOCABLE OFFER TO DEDICATE AN EASEMENT OVER A PORTION OF LAND FOR PUBLIC STREET PER DOC. REC. SEP. 22, 2010 AS DOC. NO. 2010-0503417 O.R.
9. AN EASEMENT GRANTED TO SPRINT COMMUNICATIONS COMPANY L.P., FOR PERMANENT TELECOMMUNICATIONS EASEMENT AND INCIDENTAL PURPOSES PER DOC. REC. DECEMBER 12, 2013 AS DOC. NO. 2013-0717457 O.R. (NON PLOTTABLE)

NOTE:

1. ALL FINISHED FLOORS DESIGNED 2' ABOVE EXISTING 100 YEAR WSE
2. NO PERMITS SHALL BE ISSUED FOR GRADING OR OTHER WORK IN THE FLOODPLAIN OF THE OTAY RIVER UNTIL THE PERMITTEE OBTAINS AN APPROVED NO RISE CERTIFICATION PER THE CITY OF SAN DIEGO
3. WHEN AS-BUILT GRADING AND PUBLIC IMPROVEMENT PLANS ARE AVAILABLE, THE PERMITTEE MUST SUBMIT A REQUEST FOR THE FINAL LOMA OR FINAL LOMR TO FEMA VIA THE FLOODPLAIN MANAGEMENT SECTION OF THE DEVELOPMENT SERVICES DEPARTMENT. THE PERMITTEE MUST PROVIDE ALL DOCUMENTATION, ENGINEERING CALCULATIONS, AND FEES WHICH ARE REQUIRED BY FEMA
4. THE BOND FOR THIS PROJECT WILL NOT BE RELEASED UNTIL THE FINAL LOMA OR FINAL LOMR IS ISSUED BY FEMA. THE FLOODPLAIN MANAGEMENT SECTION OF THE CITY OF SAN DIEGO'S PUBLIC WORKS/ENGINEERING AND CAPITAL PROJECTS WILL NOTIFY THE DEVELOPMENT SERVICES DEPARTMENT OF SUCH ISSUANCE AS SOON AS IT IS INFORMED BY FEMA
5. 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
6. ALL PRIVATE ENCROACHMENTS IN THE ROW WILL REQUIRE AN EMRA WITH PUBLIC IMPROVEMENT PLANS
7. ALL PROPOSED IMPROVEMENTS WITHIN THE ROW SHALL BE CONSTRUCTED PER CURRENT CITY OF SAN DIEGO STANDARDS.
8. WATER PURVEYOR IS CALIFORNIA AMERICAN WATER.
9. THE SUBDIVIDER SHALL UNDERGROUND EXISTING AND/OR PROPOSED PUBLIC UTILITY SYSTEMS AND SERVICE FACILITIES IN ACCORDANCE WITH THE SAN DIEGO MUNICIPAL CODE.
10. THE SUBDIVIDER SHALL ENSURE THAT ALL ONSITE UTILITIES SERVING THE SUBDIVISION SHALL BE UNDERGROUNDED WITH THE APPROPRIATE PERMITS. THE SUBDIVIDER SHALL PROVIDE WRITTEN CONFIRMATION FROM APPLICABLE UTILITIES THAT THE CONVERSION HAS TAKEN PLACE, OR PROVIDE OTHER MEANS TO ASSURE THE UNDERGROUNDING, SATISFACTORY TO THE CITY ENGINEER.

REGISTERED PROFESSIONAL ENGINEER BRIAN D. SMITH C 75822 EXP. 06-30-22 CIVIL STATE OF CALIFORNIA

FUSCOE ENGINEERING
6390 Greenwich Drive, Suite 170
San Diego, California 92122
tel 858.554.1500 ◦ fax 858.597.0335
www.fuscoe.com

SCALE: 1" = 40'

TENTATIVE PARCEL MAP NO. 2361780

BELLA MAR

CONCEPTUAL GRADING & UTILITIES

SHEET 3 OF 5 SHEETS

DESCRIPTION	BY	DATE

I.O. NO. 24007769
PTS NO. 631240
T.P.M. NO. 2361780
1794-6299 NAD83 COORDINATES
154-1739 LAMBERT COORDINATES

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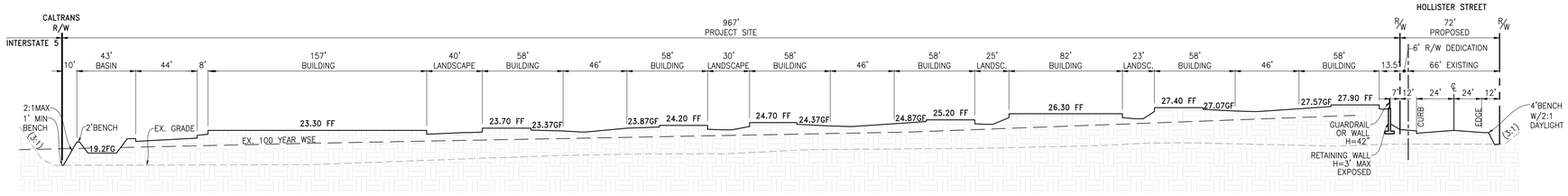

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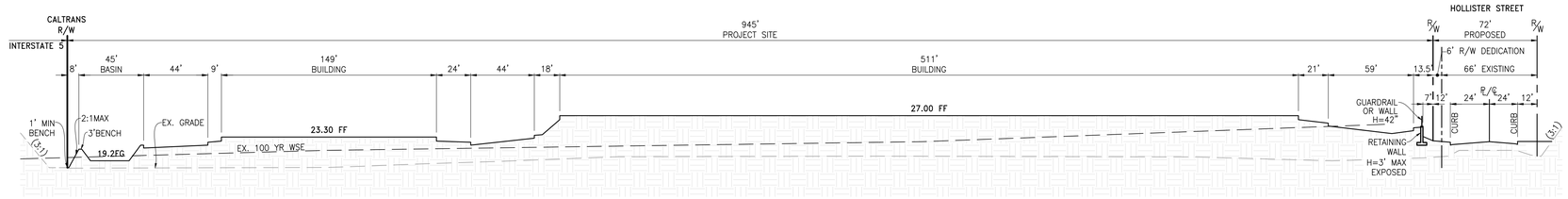

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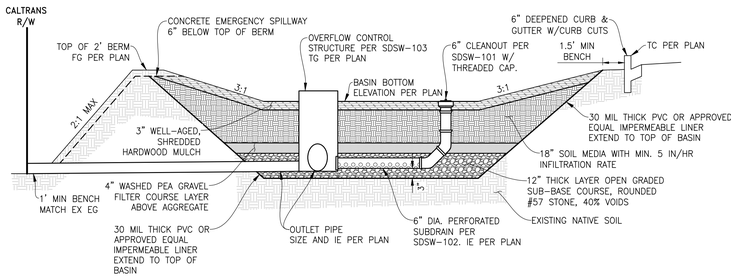

BIOFILTRATION BASIN DETAIL
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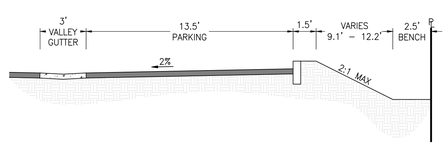

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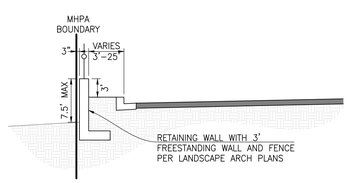

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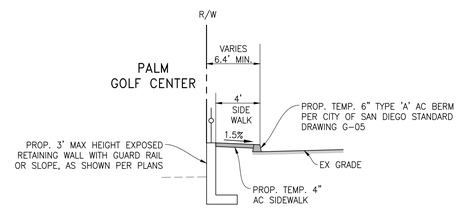

HOLLISTER ST TEMPORARY SIDEWALK CROSS SECTION
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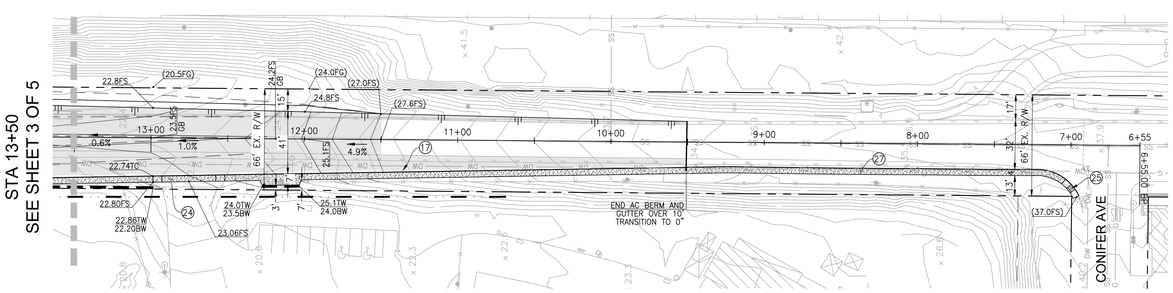

HOLLISTER AVE STA 6+55 TO STA 13+50
SCALE: 1"=40'
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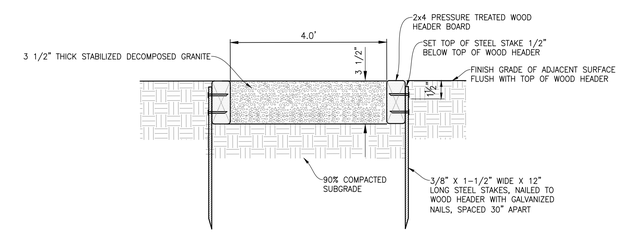
CONSTRUCTION NOTES

(17) TEMPORARY 6" TYPE 'A' AC BERM PER G-05
(24) EXISTING 35' DRIVEWAY TO BE MODIFIED PER SDG-162
(25) CURB RAMP TYPE 'C' PER G-29
(27) TEMPORARY 4" AC SIDEWALK PER SDG-155



MULTI-USE DG PATH
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REGISTERED PROFESSIONAL ENGINEER
BRIAN D. SMITH
C 75822
EXP. 06-30-22
CIVIL
STATE OF CALIFORNIA

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San Diego, California 92122
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www.fuscoe.com

TENTATIVE PARCEL MAP NO. 2361780

BELLA MAR

CONCEPTUAL CROSS SECTIONS

SHEET 4 OF 5 SHEETS

DESCRIPTION	BY	DATE

J.O. NO.	24007769
PTS NO.	631240
T.P.M. NO.	2361780
	1794-6299
	NAD83 COORDINATES
	154-1739
	LAMBERT COORDINATES

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FUSCOE ENGINEERING

PRELIMINARY DRAINAGE STUDY

BELLA MAR

PREPARED FOR

RED TAIL ACQUISITIONS, LLC
2082 MICHELSON DRIVE, 4TH FLOOR
IRVINE, CA 92612

FUSCOE ENGINEERING, INC
6390 GREENWICH DR. STE 170
SAN DIEGO, CA 92122

PROJECT MANAGER:
BRYAN D. SMITH , P.E.

DATE PREPARED: FEBRUARY 2019
FEI# 1621-001-01

full circle thinking®

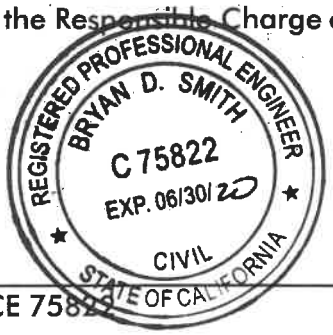
PRELIMINARY DRAINAGE STUDY

BELLA MAR APARTMENTS

408 HOLLISTER STREET
SAN DIEGO, CA 92154

APN#627-100-09-00

Prepared by Jesus Garcia Under the Responsible Charge of:



Bryan D. Smith, PE RCE 75822 EXP: 06-30-20

Fuscoe Engineering, San Diego, Inc.
6390 Greenwich Dr., Ste 170 San Diego, CA 92122
858-554-1500
bsmith@fuscoe.com

For

Red Tail Acquisitions, LLC
2082 Michelson Dr, 4th Floor
Irvine, CA 92612

FEBRUARY 2019

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1. INTRODUCTION

The purpose of this preliminary drainage study is to present the preliminary drainage design for the Bella Mar Entitlements Project (Project) and to demonstrate that the project will comply with the City of San Diego Drainage Design Manual (SDDDM) 2017 Criteria.

1.1 Project Description

The project proposes entitlements including a rezone and Tentative Map to support a medium density residential development including 380 units on approximately 14.1 acres located at 408 Hollister Street, San Diego, California. The site is bordered by Hollister Street on the east, Interstate 5 on the west, Otay River on the north, and an existing driving range on the south.

The project does not propose to dredge or fill any waters of the U.S.; therefore, the project is not required to obtain approval from the Regional Water Quality Board under Federal Clean Water Act Section 401 or 404.

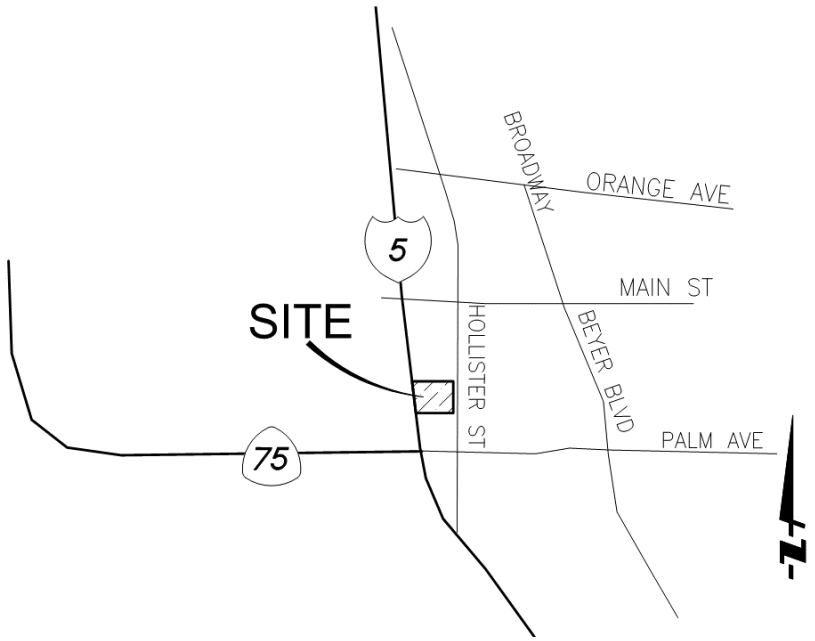


Figure 1. Vicinity Map

1.2 Existing Conditions

The existing project site is a vacant rural lot south of the Otay River which was previously developed as a go-kart race track. The site is mostly pervious and is covered by light vegetation with mild slopes averaging less than 1%. Refer to the Existing Conditions Drainage Map in Appendix 1.

Most of the site (Basin A) flows west towards I-5 to an existing 24" culvert, designated as Point of Compliance (POC)-1, prior to discharging into the Otay River west of I-5. Portions of the south neighboring property drains through the site (Basin B) and are tributary to POC-1. A smaller area along the northern boundary (Basin C) sheet flows into the Otay River at POC-2. The remainder of the site fronting Hollister Street (Basin D) combines with public street runoff and surface flows through the adjacent private property to the south toward an existing 36" RCP culvert which crosses the I-5 prior to discharging toward the Otay River. The analysis point for the limits of basin disturbance that contribute to the existing 36" RCP culvert is designated as POC-3.

Hollister Street (Basins E & F) does not have a defined storm drainage system. The street is crowned with low points along the project frontage. Runoff ponds on the East side of Hollister adjacent to the MTS right of way until it overtops the crown and drains through the neighboring south property into POC-3. The north end of Hollister Street (Basin G) surface flows north into the Otay River crossing on Hollister identified as POC-4.

1.3 Proposed Conditions

The project proposes to entitle the site for mixed use residential (RM-2-5) with 380 units over 15 buildings with associated recreation facilities, parking, and infrastructure as shown in the Tentative Parcel Map (TPM) included in Appendix 2. The project also includes modifications to Hollister to widen the road to a two-lane collector along the project frontage.

The project is located within the FEMA Floodplain and will fill the site to provide 2 feet of freeboard above the 100-year flood base elevations per City of San Diego Municipal Code requirements. A CLOMR-F will be required to be processed with FEMA.

The Project will maintain existing drainage patterns to the maximum extend practical. Basins A-1 through A-5 will be collected and conveyed west to a biofiltration basin which will provide treatment and peak flow attenuation before discharging into POC-1. Basin B includes portions of the adjacent site to the south and will be collected via a catch basin and bypass the proposed basin to discharge at POC 1. Basin C, which sheet flows directly into the Otay River at POC-2, will be increased in area by approximately 1 acre to maintain a drainage delineation for the MHPA area.

Runoff from Hollister Street (Basins E, F, and G) will be rerouted into a proposed storm drain and will tie into a culvert in Hollister Street to eliminate the conveyance of the public street drainage through private property. Basin H includes the private driveway which is routed through the proposed storm drain in Hollister Street and contributes to POC-4. The onsite Basin D tributary to POC-3 will be reduced (over 95%) to an isolated slope that runs off through the neighboring south property.

1.4 Proposed Green Street Improvements for Hollister Street

The improvements to Hollister Street include the road widening to a two-lane collector along the project frontage, installing curb & gutter, sidewalk, public storm drain system, and implementing Green Street BMP's to meet the PDP Exemption Category 2 for redevelopment of existing paved streets under The City of San Diego Storm Water Standards BMP Design Manual, October 2018 Edition. ~~Bioretention basins have been sized to treat the entire street frontage area and proposed to be installed in the parkway with pop-outs in the parking lanes. Opposite the site frontage, impervious area dispersion is being implemented for redundancy by allowing half the street to sheet flow into hydrologic type A soils for 10-year storm runoff events, while higher flows are collected into a catch basin~~. See the separate preliminary SQWMP report for this project.

2. METHODOLOGY

2.1 Rational Method

The site is inundated for the 100-year storm event of the Otay River, however for the period before the Otay River's peak time of concentration, this report analyses the proposed developed storm runoff for the site's relatively smaller time of concentration. Runoff was calculated using the Modified Rational Method equation below:

$Q = C \times I \times A$

Where:
Q = Flow rate in cubic feet per second (cfs)
C = Runoff coefficient
I = Rainfall Intensity in inches per hour (in/hr)
A = Drainage basin area in acres, (ac)

Modified Rational Method calculations were performed using the Advanced Engineering Software AES 2014) computer program. To perform the hydrology routing, the total watershed area was divided into sub-areas which discharge at designated nodes. The procedure for the sub-area summation model is as follows:

(1) Subdivide the watershed into an initial sub-area (generally 1 lot) and subsequent sub-areas, which are generally less than 10 acres in size. Assign upstream and downstream node numbers to each sub-area.

(2) Estimate an initial T_c by using the appropriate nomograph or overland flow velocity estimation. The minimum T_c considered is 5.0 minutes.

(3) Using the initial T_c, determine the corresponding values of I. Then Q = CIA.

(4) Using Q, estimate the travel time between this node and the next by Manning's equation as applied to particular channel or conduit linking the two nodes. Then, repeat the calculation for Q based on the revised intensity (which is a function of the revised time of concentration)

The nodes are joined together by links, which may be street gutter flows, drainage swales, drainage ditches, pipe flow, or various channel flows. The AES 2014 computer software sub-area menu is as follows:

SUBAREA HYDROLOGIC PROCESS

1. Confluence analysis at node.
2. Initial sub-area analysis (including time of concentration calculation).
3. Pipe flow travel time (computer estimated).
4. Pipe flow travel time (user specified).
5. Trapezoidal channel travel time.
6. Street flow analysis through sub-area.
7. User-specified information at node.
8. Addition of sub-area runoff to main line.
9. V-gutter flow through area.
10. Copy main stream data to memory bank
11. Confluence main stream data with a memory bank
12. Clear a memory bank

At the confluence point of two or more basins, the following procedure is used to combine peak flow rates to account for differences in the basin's times of concentration. This adjustment is based on the assumption that each basin's hydrographs are triangular in shape.

(1). If the collection streams have the same times of concentration, then the Q values are directly summed,

$$Q_p = Q_a + Q_b;\ T_p = T_a = T_b$$

(2). If the collection streams have different times of concentration, the smaller of the tributary Q values may be adjusted as follows:

(i). The most frequent case is where the collection stream with the longer time of concentration has the larger Q. The smaller Q value is adjusted by a ratio of rainfall intensities.

$$Q_p = Q_b + Q_a\,(I_b/I_a);\ T_p = T_a$$

(ii). In some cases, the collection stream with the shorter time of concentration has the larger Q. Then the smaller Q is adjusted by a ratio of the T values.

$$Q_p = Q_b + Q_a\,(T_b/T_a);\ T_p = T_b$$

2.2 Runoff Coefficient

A weighted runoff coefficient was determined for both existing and proposed conditions based on the Table A-1 in the SDDDM. In existing conditions, the site is mostly vacant and undeveloped. The rural runoff coefficient C=0.45 was used for existing conditions for onsite conditions and the offsite area just south of the site (Basin B). A runoff design coefficient of C=0.50 was used for the existing and proposed conditions as a minimum set limit for the Hollister Street (Basin E, F, and H) since actual impervious calculations for pre- and post-development deviated significantly and both weighted C values resulted lower than the allowed (impervious) minimum per the SDDM. The proposed conditions, the site is considered multi-unit residential corresponding to an equivalent percentage of impervious. Therefore, the onsite runoff coefficient of C=0.70 was used per Table 1 in the SDDDM. The runoff coefficient for the northern portion of the site (Basin C) and southern portion adjacent to the site (Basin B) will remain the same as existing conditions C=0.45. See Appendix 3 for runoff coefficient calculations.

2.3 Rainfall Intensity

Rainfall intensity was determined by AES using the Intensity-Duration Chart per Figure A-1 of the SDDDM.

2.4 Tributary Areas

Drainage basins are delineated on the Existing and Proposed Hydrology Condition Maps in Appendix 1. Bold lines graphically portray the tributary area for the drainage basin.

2.5 Hydraulic Calculations

Autodesk Hydraflow Hydrographs was used to design & analyze the proposed detention basin and its outlet control structure in order to attenuate the developed onsite runoff conditions for the 100-year, 6-hour storm event. The detention basin is a dual purpose design providing mitigation for the increased onsite runoff and storm water treatment for the proposed development. For the analysis results see section 3.1.

A hydraulic analysis using FlowMaster was performed to check the capacity of the proposed public storm drain in Hollister Street. For the analysis results see section 3.2.

3. CALCULATIONS/RESULTS

3.1 Peak Flow Comparison

The project results in a decrease of the total 100-year storm runoff by 2.24 cfs by implementing a private onsite detention basin and installing a public storm drain system in Hollister Street. The Hollister drainage improvements will eliminate the long-term ponding along Hollister and the uncontrolled conveyance of public drainage through private property.

Tables 1 & 2 summarize the existing and proposed peak flow rates at each point of compliance (POC). Table 2 presents the mitigated conditions flowrate. The detention basin results are summarized in Table 3.

Table 1. EXISTING CONDITIONS HYDROLOGY SUMMARY FOR 100-YR STORM EVENT

POC	**NODE**	**BASIN (Description)**	**AREA (ac)**	**Q100 (cfs)**
POC-1 (24″ Culvert under I-5)	100	A +B (Onsite + South Offsite)	11.11	17.75
POC-2 (Otay River)	300	C (Site Along Otay River)	1.22	1.88
POC-3 (36″ Culvert under I-5)	400	D+E+F (Site Frontage + Hollister)	5.44	8.42
POC-4 (Hollister & Otay River Culvert)	600	G (Hollister)	0.31	0.63
PROJECT TOTAL			18.08	28.68

Table 2. PROPOSED CONDITIONS HYDROLOGY SUMMARY FOR 100-YR STORM EVENT

POC	**NODE**	**BASIN (Description)**	**AREA (ac)**	**Q100 (cfs)**
POC-1 (24″ Culvert under I-5)	100	A +B (Onsite + South Offsite)	12.07	17.52 (Mitigated)
POC-2 (Otay River)	300	C (Site Along Otay River)	2.29	3.52
POC-3 (36″ Culvert under I-5)	400	D (South Site Slope)	0.09	0.15
POC-4 (Hollister & Otay River Culvert)	600	E+F+G+H (Site Frontage + Hollister)	3.63	5.25
PROJECT TOTAL			18.08	26.44
DIFFERENCE FROM EXISTING			0	- 2.24

The drainage improvements require a minor re-routing of drainage area to direct portions of Hollister Ave to the box culvert at the Otay River crossing. The increase of 100-year storm runoff at POC-4 equates to less than a 0.02% of the existing flowrate in the Otay River at this location of 22,000 cfs per the FEMA floodway studies. Therefore, this impact is considered de-minimus

3.2 Private Detention Basin

The private detention basin is designed to provide stormwater treatment and attenuate the 100-year storm runoff for the proposed development. The total basin depth provided is 2 feet from bottom elevation of 19.2 feet. The bottom 6" of the basin are reserved to meet the ponding requirement for water quality treatment, therefore outlet riser/control structure is raised a minimum of 6" from the basin bottom. The remaining 18" of the basin stores 100-year storm runoff volume and mitigate the proposed flowrate below existing conditions. Per the routing analysis the detention basins fills about 12" above the grate and leaves about 6" of freeboard. The basin storage volume used is 8,446 cf.

Table 3. DETENTION BASIN ATTENUATION FOR 100-YR STORM EVENT AT POC-1

POC	NODE	BASIN (Description)	EXISTING AREA (ac)	PROPOSED AREA (ac)	EXISTING Q100 (cfs)	UNMITIGATED Q100 (cfs)	MITIGATED Q100 (cfs)
POC-1 (24" Culvert under I-5)	105	A (Onsite)	10.20	10.97	16.53	21.95	15.86
POC-1 (24" Culvert under I-5)	200	B (South Slope + Offsite)	0.91	1.10	1.39	1.68	1.68 (no attenuation)
POC-1 CONFLUENCE TOTAL	100	A +B (Onsite + South Offsite)	11.11	12.07	17.75	23.61	17.52
DIFFERENCE FROM EXISTING				+0.96		+5.69	- 0.40

Basin attenuation occurs at Node 105 of the hydrology analysis for proposed conditions. Per table 3, at Node 105 the acreage is increased by 0.77 acres but the proposed runoff is mitigated down by 6.09 cfs. At Node 100 (representing POC-1), the mitigated basin outlet flow (for Basins A1-A4) is confluence with the offsite Basin B and additional south slopes. When compared to existing conditions, the development results (at POC-1) is an increase of 0.96 acres but a decreased flowrate by 0.40 cfs.

3.3 Public Storm Drain

A hydraulic analysis using FlowMaster was performed to check the capacity of the proposed public storm drain in Hollister Street. At a minimum slope of 0.3% an 18" RCP pipe is 84% full with the project's proposed flow rates. See Appendix 7 for the results.

4. CONCLUSION

The project will match existing drainage patterns to the maximum extent feasible. The project will result in a total net decrease of 2.24 cfs in the 100-year peak runoff from the studied area of 18.08 acres by providing an onsite private detention system and installing a public storm drain in Hollister. The buildings will be elevated a minimum 2ft above the FEMA 100-year water surface elevation. A CLOMR-F will be processed to document the fill within the flood plain.

The project is anticipated to improve the drainage conditions of the site by reducing the peak flowrate through the detention basins, alleviating long term ponding along Hollister Ave, and eliminating the uncontrolled public drainage flowing through private property.

APPENDIX 1

EXISTING HYDROLOGY MAP

PROPOSED HYDROLOGY MAP

OTAY RIVER

INTERSTATE 5

HOLLISTER ST

MHPA LINE

NODE 600/POC-4:
A=0.31ac
Tc=6.95MIN
Q_{100}=0.63CFS

NODE 300/POC-2:
A=1.22ac
Tc=10.57MIN
Q_{100}=1.88CFS

NODE 100:
A=10.20ac
Tc=14.27MIN
Q_{100}=16.53CFS

POC-1:
A=11.11ac
Tc=14.27MIN
Q_{100}=17.75CFS

NODE 200:
A=0.91ac
Tc=11.83MIN
Q_{100}=1.39CFS

NODE 400:
A=2.73ac
Tc=14.76MIN
Q_{100}=4.34CFS

NODE 500:
A=2.71ac
Tc=13.56
Q_{100}=4.12CFS

POC-3:
A=5.44ac
Tc=15.07MIN
Q_{100}=8.42CFS

BASIN A
10.20 AC

BASIN B
0.91 AC

BASIN C
1.22 AC

BASIN D
2.73 AC

BASIN E
1.55 AC

BASIN F
1.16 AC

BASIN G
0.31 AC

EX 24" RCP SD
PER CALTRANS
CONTRACT NO. 122404
DOCUMENT NO. A-0002600

EX 36" RCP SD

L=50'
A=1.02ac

L=155'
A=0.20ac

L=100'
A=0.40ac

L=730'
A=9.8ac

L=360'

L=90'
A=0.71ac

A=0.20ac L=180'

L=460'
A=2.13ac

L=350' A=0.93ac

L=75'
A=0.21ac

L=100'
A=0.25ac

L=170'
A=0.60ac

L=500' A=1.30ac

L=100'
A=0.23ac

LEGEND

EXISTING CONTOURS
BASIN LIMITS
SUB-BASIN LIMITS
INTIAL AREA LIMITS
FLOW PATH
FLOW DIRECTION
HYDROLOGY NODE
EXISTING STORM DRAIN

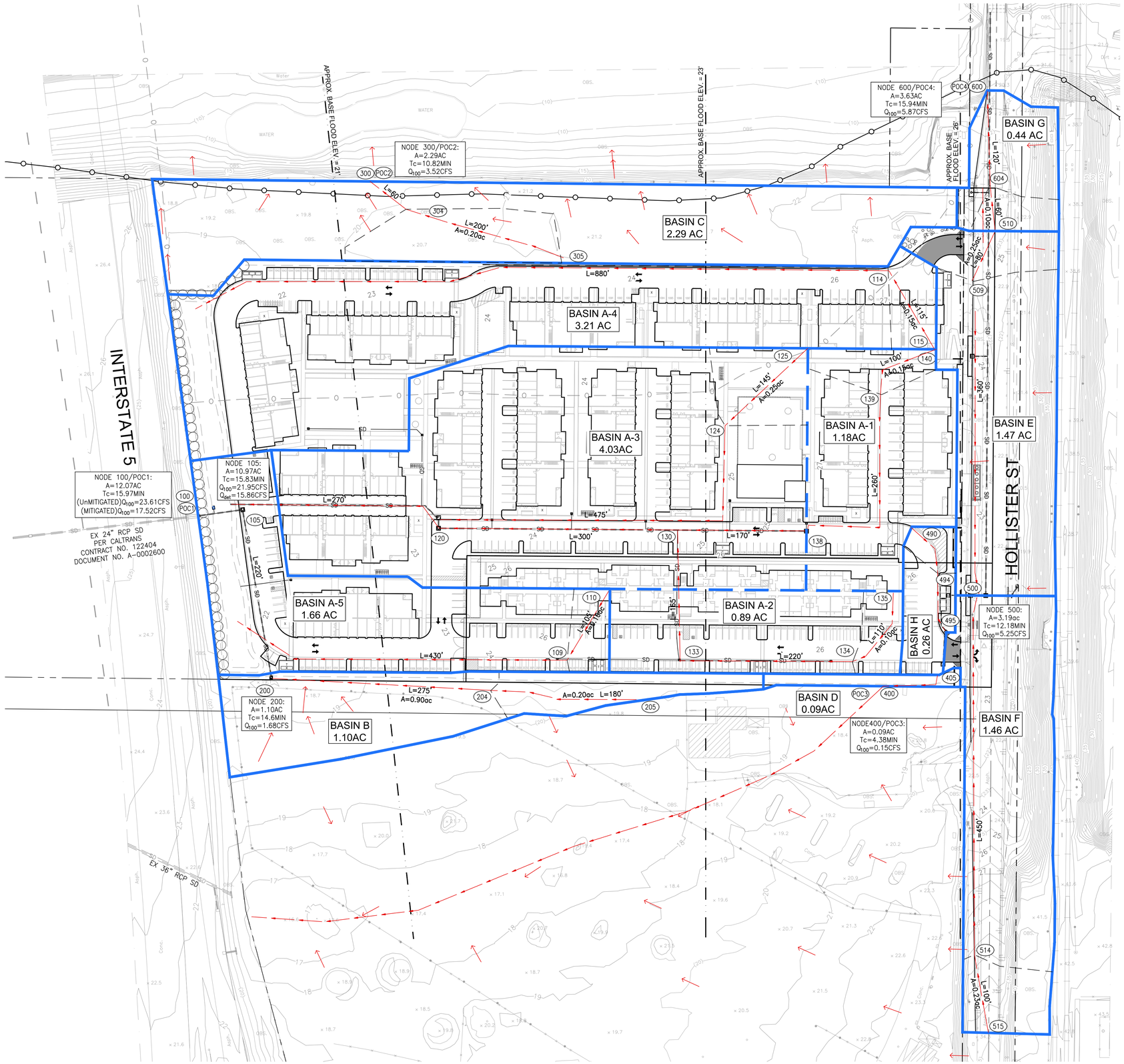
60' 0' 30' 60'
SCALE: 1" = 60'

FUSCOE ENGINEERING
6390 Greenwich Dr., Suite 170, San Diego, California 92122
tel 858.554.1500 ◦ fax 858.597.0335 ◦ www.fuscoe.com

EXISTING HYDROLOGY MAP

BELLA MAR
FEBRUARY 2019

F:\Projects\1621\001_Support Files\Reports\Hydrology\Figures\Existing Hydrology.dwg (7/21/2019 2:24 PM) Plotted by: Jesus Garcia



LEGEND

- EXISTING CONTOURS
- PROPOSED CONTOURS
- BASIN LIMITS
- SUB-BASIN LIMITS
- INTIAL AREA LIMITS
- FLOW PATH
- FLOW DIRECTION
- HYDROLOGY NODE
- EXISTING STORM DRAIN
- PROPOSED STORM DRAIN

60' 0' 30' 60'

SCALE: 1" = 60'

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6390 Greenwich Dr., Suite 170, San Diego, California 92122
tel 858.554.1500 ◦ fax 858.597.0335 ◦ www.fuscoe.com

PROPOSED HYDROLOGY MAP

BELLA MAR
FEBRUARY 2019

F:\Projects\1621\001_Support Files\Reports\Hydrology\Figures\Proposed Hydrology.dwg (7/16/2019 11:25 AM) Plotted by: Brianna VanGorder

APPENDIX 2

RUNOFF COEFFICIENT CALCULATIONS

FUSCOE ENGINEERING

Job Name: BELLA MAR
Job #: 1621-001
Date: 2/12/2019

Runoff Coefficient Calculations

Runoff Coefficent Variables Per City of San Diego Drainage Design Manual (January '17)

Assumptions: D soils per City Drainage Manual

EXISTING CONDITIONS: RURAL (ONSITE)

Rural C =	0.45	Per Drainage Design Manual Appendix A Table A-1

EXISTING CONDITIONS: RURAL (OFFSITE (SOUTH))

Rural C =	0.45	Per Drainage Design Manual Appendix A Table A-1

EXISTING CONDITIONS: HOLLISTER ST (OFFSITE)

Area Impervious =	32620	sf	25%
Area Pervious =	98980	sf	75%
Total Area =	131600	sf	

Industrial C =	0.95	Per Drainage Design Manual Appendix A Table A-1
Tabulated % Impervious =	90%	
Acutal % Impervious =	25%	
Calculated Cweighted =	0.26	
**Design C =	0.50	

** Per Note (2) of Table A-1, no weighed C for commerical or industrial shall be less than C=0.5

PROPOSED CONDITIONS: MULTI-USE RESIDENTIAL (ONSITE)

Area Impervious =	373370	sf	73%
Area Pervious =	138210	sf	27%
Total Area =	511580	sf	

Multi-Use Residential C=	0.70	Per Drainage Design Manual Appendix A Table A-1
Acutal % Impervious =	73%	
Design C=	0.70	

PROPOSED CONDITIONS: RURAL (OFFSITE)

Rural C =	0.45	Per Drainage Design Manual Appendix A Table A-1

PROPOSED CONDITIONS: HOLLISTER ST (OFFSITE)

Area Impervious =	55540	sf	38%
Area Pervious =	91120	sf	62%
Total Area =	146660	sf	

Industrial C =	0.95	Per Drainage Design Manual Appendix A Table A-1
Tabulated % Impervious =	90%	
Acutal % Impervious =	38%	
Calculated Cweighted =	0.40	
**Design C =	0.50	

** Per Note (2) of Table A-1, no weighed C for commerical or industrial shall be less than C=0.5

APPENDIX 3

EXISTING HYDROLOGY CALCULATIONS

FUSCOE ENGINEERING

Job Name: BELLA MAR
Job #: 1621-001
Run Name: BMEX
Date: 11/13/2018

EXISTING HYDROLOGY - 100 YR

Node to Node		Code	Elev 1 (feet)	Elev 2 (feet)	Length (feet)	Runoff Coeff.	Area (ac.)	Comments	BANK		
									1	2	3
105	104	2	22.9	21.9	100.0	0.45	0.10	Basin A: Initial Area			
104	100	5	21.9	16.7	801.0	0.45	9.83	Sheet Flow			
100	100	1						Node 100 Confluence: 1 of 2			
205	204	2	19.5	19.1	100.0	0.45	0.13	Basin B: Initial Area (offsite)			
204	200	5	19.1	18.9	154.0	0.45	0.41	Sheet Flow (offsite)			
200	100	5	18.9	16.7	240.0	0.00	0.00	Sheet Flow			
100	100	1						Node 100 Confluence: 2 of 2			
305	304	2	23.0	21.9	216.0	0.45	0.07	Basin C: Initial Area			
304	300	5	21.9	18.8	803.0	0.45	1.11	Sheet Flow (flows offsite)			
405	404	2	23.0	22.5	75.0	0.45	0.09	Basin D: Initial Area			
404	400	5	22.5	18.7	597.0	0.45	2.95	Sheet Flow (flows offsite)			
400	400	1						Confluence 400: 1 of 3			
505	504	2	23.8	23.0	60.0	0.50	0.09	Basin E: Initial Area (Hollister St)			
504	500	5	23.0	19.5	396.0	0.50	1.08	Sheet Flow (road)			
500	400	5	19.5	18.7	202.0	0.45	0.00	Sheet Flow			
400	400	1						Confluence 400: 2 of 3			
605	604	2	31.5	28.5	65.0	0.50	0.09	Basin F: Initial Area (Hollister St)			
604	600	5	28.5	19.0	286.0	0.50	1.20				
600	400	5	19.0	18.7	106.0	0.45	0.00				

```
******************************************************************************

            RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                         2003,1985,1981 HYDROLOGY MANUAL
          (c) Copyright 1982-2014 Advanced Engineering Software (aes)
              Ver. 21.0 Release Date: 06/01/2014  License ID 1355

                              Analysis prepared by:

                                Fuscoe Engineering
                           6390 Greenwich Drive, Suite 170
                                 San Diego, CA 92122


 ************************* DESCRIPTION OF STUDY *************************
* BELLA MAR                                                             *
* EXISTING CONDITIONS - 100 YR                                          *
*                                                                       *
 *************************************************************************

  FILE NAME: BMEX.DAT
  TIME/DATE OF STUDY: 14:39 11/21/2018
-------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
-------------------------------------------------------------------------
  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  SPECIFIED MINIMUM PIPE SIZE(INCH) =   6.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  RAINFALL-INTENSITY ADJUSTMENT FACTOR = 1.000
  *USER SPECIFIED:
  NUMBER OF [TIME,INTENSITY] DATA PAIRS = 10
   1)   5.000;  4.400
   2)  10.000;  3.450
   3)  20.000;  2.500
   4)  30.000;  2.000
   5)  40.000;  1.700
   6)  50.000;  1.500
   7)  60.000;  1.310
   8) 120.000;  0.860
   9) 180.000;  0.660
  10) 240.000;  0.560
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: ONLY PEAK CONFLUENCE VALUES CONSIDERED
  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
```

NO.	(FT)	(FT)	SIDE / SIDE/ WAY	(FT)	(FT)	(FT)	(FT)	(n)
1	30.0	20.0	0.018/0.018/0.020	0.67	2.00	0.0313	0.167	0.0150

```
  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.00 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*

****************************************************************************
  FLOW PROCESS FROM NODE    105.00 TO NODE    104.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =     22.90
  DOWNSTREAM ELEVATION(FEET) =     21.90
  ELEVATION DIFFERENCE(FEET) =      1.00
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    9.789
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    70.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.490
  SUBAREA RUNOFF(CFS) =      0.16
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.16

****************************************************************************
  FLOW PROCESS FROM NODE    104.00 TO NODE    100.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     21.90  DOWNSTREAM(FEET) =     16.70
  CHANNEL LENGTH THRU SUBAREA(FEET) =   801.00   CHANNEL SLOPE =  0.0065
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   1.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.278
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       5.37
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.91
```

```
 AVERAGE FLOW DEPTH(FEET) =   0.16   TRAVEL TIME(MIN.) =  14.64
 Tc(MIN.) =   24.43
 SUBAREA AREA(ACRES) =     9.83       SUBAREA RUNOFF(CFS) =   10.08
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.450
 TOTAL AREA(ACRES) =        9.9         PEAK FLOW RATE(CFS) =      10.18

 END OF SUBAREA CHANNEL FLOW HYDRAULICS:
 DEPTH(FEET) =  0.22   FLOW VELOCITY(FEET/SEC.) =   1.12
 LONGEST FLOWPATH FROM NODE    105.00 TO NODE    100.00 =     901.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    100.00 TO NODE    100.00 IS CODE =   1
----------------------------------------------------------------------------
 >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
 TOTAL NUMBER OF STREAMS =  2
 CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
 TIME OF CONCENTRATION(MIN.) =   24.43
 RAINFALL INTENSITY(INCH/HR) =   2.28
 TOTAL STREAM AREA(ACRES) =     9.93
 PEAK FLOW RATE(CFS) AT CONFLUENCE =     10.18

****************************************************************************
 FLOW PROCESS FROM NODE    205.00 TO NODE    204.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
 UPSTREAM ELEVATION(FEET) =     19.50
 DOWNSTREAM ELEVATION(FEET) =     19.10
 ELEVATION DIFFERENCE(FEET) =      0.40
 URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =   10.423
 WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
          THE MAXIMUM OVERLAND FLOW LENGTH =    50.00
          (Reference: Table 3-1B of Hydrology Manual)
          THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.410
 SUBAREA RUNOFF(CFS) =      0.20
 TOTAL AREA(ACRES) =      0.13   TOTAL RUNOFF(CFS) =      0.20

****************************************************************************
 FLOW PROCESS FROM NODE    204.00 TO NODE    200.00 IS CODE =  51
----------------------------------------------------------------------------
 >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
```

```
==========================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     19.10  DOWNSTREAM(FEET) =     18.90
  CHANNEL LENGTH THRU SUBAREA(FEET) =   154.00   CHANNEL SLOPE =  0.0013
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   1.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.458
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.43
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.25
  AVERAGE FLOW DEPTH(FEET) =   0.07   TRAVEL TIME(MIN.) =  10.42
  Tc(MIN.) =   20.85
  SUBAREA AREA(ACRES) =     0.41       SUBAREA RUNOFF(CFS) =    0.45
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.450
  TOTAL AREA(ACRES) =        0.5         PEAK FLOW RATE(CFS) =       0.60

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.08   FLOW VELOCITY(FEET/SEC.) =   0.27
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    200.00 =     254.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    200.00 TO NODE    100.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
==========================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     19.00  DOWNSTREAM(FEET) =     16.70
  CHANNEL LENGTH THRU SUBAREA(FEET) =   240.00   CHANNEL SLOPE =  0.0096
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   1.00
  CHANNEL FLOW THRU SUBAREA(CFS) =       0.60
  FLOW VELOCITY(FEET/SEC.) =   0.57   FLOW DEPTH(FEET) =   0.04
  TRAVEL TIME(MIN.) =   6.97   Tc(MIN.) =   27.82
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    100.00 =     494.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    100.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
==========================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =   27.82
  RAINFALL INTENSITY(INCH/HR) =   2.11
  TOTAL STREAM AREA(ACRES) =     0.54
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      0.60
```

```
  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       10.18    24.43        2.278          9.93
      2        0.60    27.82        2.109          0.54

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1       10.71    24.43       2.278
      2       10.02    27.82       2.109

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      10.71   Tc(MIN.) =   24.43
  TOTAL AREA(ACRES) =       10.5
  LONGEST FLOWPATH FROM NODE    105.00 TO NODE    100.00 =     901.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    305.00 TO NODE    304.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   216.00
  UPSTREAM ELEVATION(FEET) =     23.00
  DOWNSTREAM ELEVATION(FEET) =     21.90
  ELEVATION DIFFERENCE(FEET) =      1.10
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =   10.398
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    50.37
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.412
  SUBAREA RUNOFF(CFS) =      0.11
  TOTAL AREA(ACRES) =      0.07   TOTAL RUNOFF(CFS) =      0.11

****************************************************************************
  FLOW PROCESS FROM NODE    304.00 TO NODE    300.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
```

```
  ELEVATION DATA: UPSTREAM(FEET) =     21.90  DOWNSTREAM(FEET) =     18.80
  CHANNEL LENGTH THRU SUBAREA(FEET) =   803.00   CHANNEL SLOPE =  0.0039
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
  MANNING'S FACTOR = 0.300   MAXIMUM DEPTH(FEET) =   1.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  0.599
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.29
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.06
  AVERAGE FLOW DEPTH(FEET) =   0.13   TRAVEL TIME(MIN.) = 206.31
  Tc(MIN.) =  216.71
  SUBAREA AREA(ACRES) =     1.11       SUBAREA RUNOFF(CFS) =    0.30
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.450
  TOTAL AREA(ACRES) =        1.2         PEAK FLOW RATE(CFS) =       0.32

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.14   FLOW VELOCITY(FEET/SEC.) =   0.07
  LONGEST FLOWPATH FROM NODE    305.00 TO NODE    300.00 =    1019.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    405.00 TO NODE    404.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    75.00
  UPSTREAM ELEVATION(FEET) =     23.00
  DOWNSTREAM ELEVATION(FEET) =     22.50
  ELEVATION DIFFERENCE(FEET) =      0.50
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =   10.082
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    56.67
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.442
  SUBAREA RUNOFF(CFS) =      0.14
  TOTAL AREA(ACRES) =      0.09   TOTAL RUNOFF(CFS) =      0.14

****************************************************************************
  FLOW PROCESS FROM NODE    404.00 TO NODE    400.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     22.50  DOWNSTREAM(FEET) =     18.70
```

```
 CHANNEL LENGTH THRU SUBAREA(FEET) =   597.00   CHANNEL SLOPE =  0.0064
 CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
 MANNING'S FACTOR = 0.300   MAXIMUM DEPTH(FEET) =   1.00
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  0.978
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
 S.C.S. CURVE NUMBER (AMC II) =   0
 TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.90
 TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.11
 AVERAGE FLOW DEPTH(FEET) =   0.21   TRAVEL TIME(MIN.) =  94.20
 Tc(MIN.) =  104.28
 SUBAREA AREA(ACRES) =     2.95       SUBAREA RUNOFF(CFS) =    1.30
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.450
 TOTAL AREA(ACRES) =        3.0         PEAK FLOW RATE(CFS) =       1.34

 END OF SUBAREA CHANNEL FLOW HYDRAULICS:
 DEPTH(FEET) =  0.25   FLOW VELOCITY(FEET/SEC.) =   0.12
 LONGEST FLOWPATH FROM NODE    405.00 TO NODE    400.00 =     672.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    400.00 TO NODE    400.00 IS CODE =   1
----------------------------------------------------------------------------
 >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
 TOTAL NUMBER OF STREAMS =  3
 CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
 TIME OF CONCENTRATION(MIN.) =  104.28
 RAINFALL INTENSITY(INCH/HR) =   0.98
 TOTAL STREAM AREA(ACRES) =     3.04
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      1.34

****************************************************************************
 FLOW PROCESS FROM NODE    505.00 TO NODE    504.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (7.3 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
 S.C.S. CURVE NUMBER (AMC II) =   0
 INITIAL SUBAREA FLOW-LENGTH(FEET) =    60.00
 UPSTREAM ELEVATION(FEET) =     23.80
 DOWNSTREAM ELEVATION(FEET) =     23.00
 ELEVATION DIFFERENCE(FEET) =      0.80
 URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    7.601
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.906
 SUBAREA RUNOFF(CFS) =      0.18
 TOTAL AREA(ACRES) =      0.09   TOTAL RUNOFF(CFS) =      0.18
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    504.00 TO NODE    500.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     23.00  DOWNSTREAM(FEET) =     19.50
  CHANNEL LENGTH THRU SUBAREA(FEET) =   396.00   CHANNEL SLOPE =  0.0088
  CHANNEL BASE(FEET) =    2.00   "Z" FACTOR =  50.000
  MANNING'S FACTOR = 0.300   MAXIMUM DEPTH(FEET) =   0.50
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  1.345
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (7.3 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.58
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.13
  AVERAGE FLOW DEPTH(FEET) =   0.28   TRAVEL TIME(MIN.) =  50.58
  Tc(MIN.) =   58.18
  SUBAREA AREA(ACRES) =     1.08       SUBAREA RUNOFF(CFS) =    0.73
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.500
  TOTAL AREA(ACRES) =        1.2         PEAK FLOW RATE(CFS) =       0.79

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.32   FLOW VELOCITY(FEET/SEC.) =   0.14
  LONGEST FLOWPATH FROM NODE    505.00 TO NODE    500.00 =     456.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    500.00 TO NODE    400.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     19.50  DOWNSTREAM(FEET) =     18.70
  CHANNEL LENGTH THRU SUBAREA(FEET) =   202.00   CHANNEL SLOPE =  0.0040
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   1.00
  CHANNEL FLOW THRU SUBAREA(CFS) =       0.79
  FLOW VELOCITY(FEET/SEC.) =   0.45   FLOW DEPTH(FEET) =   0.07
  TRAVEL TIME(MIN.) =   7.47   Tc(MIN.) =   65.65
  LONGEST FLOWPATH FROM NODE    505.00 TO NODE    400.00 =     658.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    400.00 TO NODE    400.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
```

```
  TIME OF CONCENTRATION(MIN.) =   65.65
  RAINFALL INTENSITY(INCH/HR) =   1.27
  TOTAL STREAM AREA(ACRES) =     1.17
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      0.79

****************************************************************************
  FLOW PROCESS FROM NODE    605.00 TO NODE    604.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (7.3 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    65.00
  UPSTREAM ELEVATION(FEET) =     31.50
  DOWNSTREAM ELEVATION(FEET) =     28.50
  ELEVATION DIFFERENCE(FEET) =      3.00
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    5.230
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.356
  SUBAREA RUNOFF(CFS) =      0.20
  TOTAL AREA(ACRES) =      0.09   TOTAL RUNOFF(CFS) =      0.20

****************************************************************************
  FLOW PROCESS FROM NODE    604.00 TO NODE    600.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     28.50  DOWNSTREAM(FEET) =     19.50
  CHANNEL LENGTH THRU SUBAREA(FEET) =   286.00   CHANNEL SLOPE =  0.0315
  CHANNEL BASE(FEET) =    2.00   "Z" FACTOR =  50.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   0.50
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.722
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (7.3 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.32
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.43
  AVERAGE FLOW DEPTH(FEET) =   0.12   TRAVEL TIME(MIN.) =   3.34
  Tc(MIN.) =    8.57
  SUBAREA AREA(ACRES) =     1.20       SUBAREA RUNOFF(CFS) =    2.23
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.500
  TOTAL AREA(ACRES) =        1.3         PEAK FLOW RATE(CFS) =       2.40

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.15   FLOW VELOCITY(FEET/SEC.) =   1.69
  LONGEST FLOWPATH FROM NODE    605.00 TO NODE    600.00 =     351.00 FEET.
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    600.00 TO NODE    400.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     19.00  DOWNSTREAM(FEET) =     18.70
  CHANNEL LENGTH THRU SUBAREA(FEET) =   406.00   CHANNEL SLOPE =  0.0007
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   1.00
  CHANNEL FLOW THRU SUBAREA(CFS) =       2.40
  FLOW VELOCITY(FEET/SEC.) =   0.34   FLOW DEPTH(FEET) =   0.19
  TRAVEL TIME(MIN.) =  20.14   Tc(MIN.) =   28.71
  LONGEST FLOWPATH FROM NODE    605.00 TO NODE    400.00 =     757.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    400.00 TO NODE    400.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  3 ARE:
  TIME OF CONCENTRATION(MIN.) =   28.71
  RAINFALL INTENSITY(INCH/HR) =   2.06
  TOTAL STREAM AREA(ACRES) =     1.29
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      2.40

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1        1.34   104.28        0.978          3.04
      2        0.79    65.65        1.268          1.17
      3        2.40    28.71        2.065          1.29

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  3 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1        3.11    28.71       2.065
      2        3.10    65.65       1.268
      3        3.08   104.28       0.978

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       3.11   Tc(MIN.) =   28.71
  TOTAL AREA(ACRES) =        5.5
```

```
                                        BMEX
  LONGEST FLOWPATH FROM NODE    605.00 TO NODE    400.00 =     757.00 FEET.
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =         5.5  TC(MIN.) =     28.71
  PEAK FLOW RATE(CFS)   =       3.11
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

APPENDIX 4

PROPOSED HYDROLOGY CALCULATIONS

FUSCOE ENGINEERING

Job Name: BELLA MAR
Job #: 1621-001
Run Name: BMPR
Date: 2/11/2019

UNMITIGATED PROPOSED HYDROLOGY - 100 YR

Node to Node		Code	Elev 1 (feet)	Elev 2 (feet)	Length (feet)	Runoff Coeff.	Area (ac.)	Comments	BANK 1	BANK 2	BANK 3
140	139	2	27.4	27.3	100.0	0.70	0.15	Basin A-1: Initial Area			
139	138	5	27.3	24.7	260.0	0.70	1.03	Open Channel Flow			
138	130	3	22.3	20.0	170.0			Pipe Flow			
130	130	1						Confluence: 1 of 2			
135	134	2	26.3	25.8	110.0	0.70	0.10	Basin A-2: Initial Area			
134	133	5	25.8	24.7	220.0	0.70	0.79	Open Channel Flow			
133	130	3	22.3	21.5	168.0			Pipe Flow			
130	130	1						Confluence: 2 of 2			
130	120	3	21.5	20.0	300.0			Pipe Flow			
120	120	1						Confluence: 1 of 2			
125	124	2	25.6	24.5	145.0	0.70	0.25	Basin A-3: Initial Area			
124	120	5	24.5	22.2	475.0	0.70	3.78	Open Channel Flow			
120	120	1						Confluence: 2 of 2			
120	105	3	20.0	16.5	270.0			Pipe Flow			
105	105	1						Confluence: 1 of 3			
115	114	2	27.0	26.1	115.0	0.70	0.15	Basin A-4: Initial Area			
114	105	5	26.1	21.7	880.0	0.70	3.06	Open Channel Flow			
105	105	1						Confluence: 2 of 3			
110	109	2	26.3	25.6	105.0	0.70	0.10	Basin A-5: Initial Area			
109	105	5	25.6	21.7	430.0	0.70	1.56	Open Channel Flow			
105	105	1						Confluence: 3 of 3			
							10.97	Total Tributary Area to Basin			
105	100	3	16.5	16.3	50.0			Pipe Flow			
100	100	1						Confluence: 1 of 2			
205	204	2	20.0	19.8	180.0	0.45	0.20	Basin B: Initial Area			
204	204	8				0.45	0.90	Addition Subarea			
204	200	5	19.8	18.6	275.0			Open Channel Flow			
200	100	3	17.1	16.3	220.0			Pipe Flow			
100	100	1						Confluence: 2 of 2: POC1			
							12.07	Total Tributary Area to POC1			
305	304	2	21.0	20.1	200.0	0.45	0.20	Basin C: Initial Area			
304	304	8				0.45	2.09	Addition Subarea			
304	300	5	20.1	19.1	60.0			Open Channel Flow: POC2			

Job Name: BELLA MAR
Job #: 1621-001
Run Name: BMPR
Date: 2/11/2019

UNMITIGATED PROPOSED HYDROLOGY - 100 YR

Node to Node		Code	Elev 1 (feet)	Elev 2 (feet)	Length (feet)	Runoff Coeff.	Area (ac.)	Comments	BANK 1	BANK 2	BANK 3
405	400	2	24.0	18.7	60.0	0.45	0.09	Basin D: Initial Area : POC3			
490	494	2	26.8	25.4	70.0	0.70	0.10	Basin H: Initial Area			
494	495	5	25.4	23.2	75.0	0.70	0.16	Open Channel Flow			
495	500	3	18.3	17.5	80.0			Pipe Flow			
500	500	1						Confluence: 1 of 3			
510	509	2	25.0	23.4	80.0	0.58	0.25	Basin E: Initial Area			
509	500	6	23.4	22.1	360.0	0.58	1.22	Street Flow			
500	500	1						Confluence: 2 of 3			
515	514	2	34.8	31.4	100.0	0.58	0.23	Basin F: Initial Area			
514	500	6	31.4	22.1	450.0	0.58	1.23	Street Flow			
500	500	1						Confluence: 3 of 3			
500	600	3	17.2	15.4	750.0			Pipe Flow			
600	600	1						Confluence: 1 of 2			
510	604	2	25.0	23.5	60.0	0.58	0.10	Basin G: Initial Area			
604	600	5	23.5	20.3	120.0	0.58	0.34	Open Channel Flow			
600	600	1						Confluence: 2 of 2			
							3.63	Total Tributary Area to POC2			

```
**************************************************************************

            RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                         2003,1985,1981 HYDROLOGY MANUAL
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                              Analysis prepared by:

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 ************************* DESCRIPTION OF STUDY *************************
* BELLA MAR                                                             *
* UNMITIGATED DEVELOPED CONDITIONS - 100 YR                             *
*                                                                       *
 *************************************************************************

  FILE NAME: BMPR.DAT
  TIME/DATE OF STUDY: 08:11 02/12/2019
--------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
--------------------------------------------------------------------------
  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  SPECIFIED MINIMUM PIPE SIZE(INCH) =   6.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  RAINFALL-INTENSITY ADJUSTMENT FACTOR = 1.000
  *USER SPECIFIED:
  NUMBER OF [TIME,INTENSITY] DATA PAIRS = 20
   1)   5.000;  4.400
   2)   6.000;  4.200
   3)   7.000;  3.900
   4)   8.000;  3.750
   5)   9.000;  3.600
   6)  10.000;  3.450
   7)  11.000;  3.300
   8)  12.000;  3.200
   9)  14.000;  3.000
  10)  15.000;  2.900
  11)  16.000;  2.800
  12)  17.000;  2.700
  13)  19.000;  2.600
  14)  20.000;  2.550
  15)  25.000;  2.230
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  16)  30.000;  2.000
  17)  40.000;  1.700
  18)  50.000;  1.500
  19)  60.000;  1.310
  20) 120.000;  0.860
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: ONLY PEAK CONFLUENCE VALUES CONSIDERED
  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  =====  ======  =====  =======
  1   27.0     22.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0150

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.00 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*

****************************************************************************
  FLOW PROCESS FROM NODE    140.00 TO NODE    139.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
  S.C.S. CURVE NUMBER (AMC II) =  45
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =     27.40
  DOWNSTREAM ELEVATION(FEET) =     27.30
  ELEVATION DIFFERENCE(FEET) =      0.10
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.414
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    50.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.076
  SUBAREA RUNOFF(CFS) =      0.43
  TOTAL AREA(ACRES) =      0.15   TOTAL RUNOFF(CFS) =      0.43

****************************************************************************
  FLOW PROCESS FROM NODE    139.00 TO NODE    138.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
```

```
 ELEVATION DATA: UPSTREAM(FEET) =     27.30  DOWNSTREAM(FEET) =     24.70
 CHANNEL LENGTH THRU SUBAREA(FEET) =   260.00   CHANNEL SLOPE =  0.0100
 CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
 MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   1.00
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.431
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
 S.C.S. CURVE NUMBER (AMC II) =  45
 TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.68
 TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.17
 AVERAGE FLOW DEPTH(FEET) =   0.06   TRAVEL TIME(MIN.) =   3.71
 Tc(MIN.) =   10.12
 SUBAREA AREA(ACRES) =     1.03       SUBAREA RUNOFF(CFS) =    2.47
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.700
 TOTAL AREA(ACRES) =        1.2         PEAK FLOW RATE(CFS) =       2.83

 END OF SUBAREA CHANNEL FLOW HYDRAULICS:
 DEPTH(FEET) =  0.07   FLOW VELOCITY(FEET/SEC.) =   1.46
 LONGEST FLOWPATH FROM NODE    140.00 TO NODE    138.00 =     360.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    138.00 TO NODE    130.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    22.30  DOWNSTREAM(FEET) =    20.00
 FLOW LENGTH(FEET) =   170.00   MANNING'S N =  0.011
 DEPTH OF FLOW IN  12.0 INCH PIPE IS   6.8 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   6.21
 ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =       2.83
 PIPE TRAVEL TIME(MIN.) =   0.46    Tc(MIN.) =   10.58
 LONGEST FLOWPATH FROM NODE    140.00 TO NODE    130.00 =     530.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    130.00 TO NODE    130.00 IS CODE =   1
----------------------------------------------------------------------------
 >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
 TOTAL NUMBER OF STREAMS =  2
 CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
 TIME OF CONCENTRATION(MIN.) =   10.58
 RAINFALL INTENSITY(INCH/HR) =   3.36
 TOTAL STREAM AREA(ACRES) =     1.18
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      2.83

****************************************************************************
```

```
 FLOW PROCESS FROM NODE    135.00 TO NODE    134.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
 S.C.S. CURVE NUMBER (AMC II) =  45
 INITIAL SUBAREA FLOW-LENGTH(FEET) =   110.00
 UPSTREAM ELEVATION(FEET) =     26.30
 DOWNSTREAM ELEVATION(FEET) =     25.80
 ELEVATION DIFFERENCE(FEET) =      0.50
 URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.414
 WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
          THE MAXIMUM OVERLAND FLOW LENGTH =    50.00
          (Reference: Table 3-1B of Hydrology Manual)
          THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.076
 SUBAREA RUNOFF(CFS) =      0.29
 TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.29

****************************************************************************
 FLOW PROCESS FROM NODE    134.00 TO NODE    133.00 IS CODE =  51
----------------------------------------------------------------------------
 >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =     25.80  DOWNSTREAM(FEET) =     24.70
 CHANNEL LENGTH THRU SUBAREA(FEET) =   220.00   CHANNEL SLOPE =  0.0050
 CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
 MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   1.00
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.343
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
 S.C.S. CURVE NUMBER (AMC II) =  45
 TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.23
 TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   0.85
 AVERAGE FLOW DEPTH(FEET) =   0.06   TRAVEL TIME(MIN.) =   4.30
 Tc(MIN.) =   10.72
 SUBAREA AREA(ACRES) =     0.79       SUBAREA RUNOFF(CFS) =    1.85
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.700
 TOTAL AREA(ACRES) =        0.9         PEAK FLOW RATE(CFS) =       2.08

 END OF SUBAREA CHANNEL FLOW HYDRAULICS:
 DEPTH(FEET) =  0.07   FLOW VELOCITY(FEET/SEC.) =   1.04
 LONGEST FLOWPATH FROM NODE    135.00 TO NODE    133.00 =     330.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    133.00 TO NODE    130.00 IS CODE =  31
```

```
------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
========================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    22.30  DOWNSTREAM(FEET) =    21.50
  FLOW LENGTH(FEET) =   168.00   MANNING'S N =  0.011
  DEPTH OF FLOW IN  12.0 INCH PIPE IS   7.8 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   3.85
  ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       2.08
  PIPE TRAVEL TIME(MIN.) =   0.73    Tc(MIN.) =   11.44
  LONGEST FLOWPATH FROM NODE    135.00 TO NODE    130.00 =     498.00 FEET.

************************************************************************
  FLOW PROCESS FROM NODE    130.00 TO NODE    130.00 IS CODE =   1
------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
========================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =   11.44
  RAINFALL INTENSITY(INCH/HR) =   3.26
  TOTAL STREAM AREA(ACRES) =     0.89
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      2.08

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1        2.83    10.58        3.363          1.18
      2        2.08    11.44        3.256          0.89

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1        4.76    10.58       3.363
      2        4.83    11.44       3.256

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       4.83   Tc(MIN.) =   11.44
  TOTAL AREA(ACRES) =        2.1
  LONGEST FLOWPATH FROM NODE    140.00 TO NODE    130.00 =     530.00 FEET.

************************************************************************
  FLOW PROCESS FROM NODE    130.00 TO NODE    120.00 IS CODE =  31
```

```
------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
========================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    21.50  DOWNSTREAM(FEET) =    20.00
  FLOW LENGTH(FEET) =   300.00   MANNING'S N =  0.011
  DEPTH OF FLOW IN  15.0 INCH PIPE IS  11.6 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.74
  ESTIMATED PIPE DIAMETER(INCH) =  15.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       4.83
  PIPE TRAVEL TIME(MIN.) =   1.05    Tc(MIN.) =   12.50
  LONGEST FLOWPATH FROM NODE    140.00 TO NODE    120.00 =     830.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =   1
------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
========================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =   12.50
  RAINFALL INTENSITY(INCH/HR) =   3.15
  TOTAL STREAM AREA(ACRES) =     2.07
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      4.83

****************************************************************************
  FLOW PROCESS FROM NODE    125.00 TO NODE    124.00 IS CODE =  21
------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
========================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =  45
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   145.00
  UPSTREAM ELEVATION(FEET) =     25.60
  DOWNSTREAM ELEVATION(FEET) =     24.50
  ELEVATION DIFFERENCE(FEET) =      1.10
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    9.000
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    57.76
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.600
  SUBAREA RUNOFF(CFS) =      0.45
  TOTAL AREA(ACRES) =      0.25   TOTAL RUNOFF(CFS) =      0.45

****************************************************************************
  FLOW PROCESS FROM NODE    124.00 TO NODE    120.00 IS CODE =  51
```

```
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     24.50  DOWNSTREAM(FEET) =     22.20
  CHANNEL LENGTH THRU SUBAREA(FEET) =   475.00   CHANNEL SLOPE =  0.0048
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
  MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   1.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.869
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
  S.C.S. CURVE NUMBER (AMC II) =  45
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       4.25
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.25
  AVERAGE FLOW DEPTH(FEET) =   0.11   TRAVEL TIME(MIN.) =   6.31
  Tc(MIN.) =   15.31
  SUBAREA AREA(ACRES) =     3.78       SUBAREA RUNOFF(CFS) =    7.59
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.688
  TOTAL AREA(ACRES) =        4.0         PEAK FLOW RATE(CFS) =       7.95

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.15   FLOW VELOCITY(FEET/SEC.) =   1.52
  LONGEST FLOWPATH FROM NODE    125.00 TO NODE    120.00 =     620.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    120.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =   15.31
  RAINFALL INTENSITY(INCH/HR) =   2.87
  TOTAL STREAM AREA(ACRES) =     4.03
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      7.95

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1        4.83    12.50        3.150          2.07
      2        7.95    15.31        2.869          4.03

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
```

```
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1       11.31    12.50       3.150
      2       12.35    15.31       2.869

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      12.35   Tc(MIN.) =   15.31
  TOTAL AREA(ACRES) =        6.1
  LONGEST FLOWPATH FROM NODE    140.00 TO NODE    120.00 =     830.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    105.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    20.00  DOWNSTREAM(FEET) =    16.50
  FLOW LENGTH(FEET) =   270.00   MANNING'S N =  0.011
  DEPTH OF FLOW IN  18.0 INCH PIPE IS  13.6 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   8.61
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      12.35
  PIPE TRAVEL TIME(MIN.) =   0.52    Tc(MIN.) =   15.83
  LONGEST FLOWPATH FROM NODE    140.00 TO NODE    105.00 =    1100.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    105.00 TO NODE    105.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =   15.83
  RAINFALL INTENSITY(INCH/HR) =   2.82
  TOTAL STREAM AREA(ACRES) =     6.10
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     12.35

****************************************************************************
  FLOW PROCESS FROM NODE    115.00 TO NODE    114.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
  S.C.S. CURVE NUMBER (AMC II) =  45
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   115.00
  UPSTREAM ELEVATION(FEET) =     27.00
  DOWNSTREAM ELEVATION(FEET) =     26.10
  ELEVATION DIFFERENCE(FEET) =      0.90
```

```
 URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    5.975
 WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
          THE MAXIMUM OVERLAND FLOW LENGTH =    58.48
          (Reference: Table 3-1B of Hydrology Manual)
          THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.205
 SUBAREA RUNOFF(CFS) =      0.44
 TOTAL AREA(ACRES) =      0.15   TOTAL RUNOFF(CFS) =      0.44

****************************************************************************
 FLOW PROCESS FROM NODE    114.00 TO NODE    105.00 IS CODE =  51
----------------------------------------------------------------------------
 >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =     26.10  DOWNSTREAM(FEET) =     21.70
 CHANNEL LENGTH THRU SUBAREA(FEET) =   880.00   CHANNEL SLOPE =  0.0050
 CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
 MANNING'S FACTOR = 0.011   MAXIMUM DEPTH(FEET) =   1.00
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  2.880
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
 S.C.S. CURVE NUMBER (AMC II) =  45
 TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       3.61
 TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.59
 AVERAGE FLOW DEPTH(FEET) =   0.08   TRAVEL TIME(MIN.) =   9.23
 Tc(MIN.) =   15.20
 SUBAREA AREA(ACRES) =     3.06       SUBAREA RUNOFF(CFS) =    6.17
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.700
 TOTAL AREA(ACRES) =        3.2         PEAK FLOW RATE(CFS) =       6.47

 END OF SUBAREA CHANNEL FLOW HYDRAULICS:
 DEPTH(FEET) =  0.11   FLOW VELOCITY(FEET/SEC.) =   1.87
 LONGEST FLOWPATH FROM NODE    115.00 TO NODE    105.00 =     995.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    105.00 TO NODE    105.00 IS CODE =   1
----------------------------------------------------------------------------
 >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
 TOTAL NUMBER OF STREAMS =  3
 CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
 TIME OF CONCENTRATION(MIN.) =   15.20
 RAINFALL INTENSITY(INCH/HR) =   2.88
 TOTAL STREAM AREA(ACRES) =     3.21
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      6.47

****************************************************************************
```

```
 FLOW PROCESS FROM NODE    110.00 TO NODE    109.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
 S.C.S. CURVE NUMBER (AMC II) =  45
 INITIAL SUBAREA FLOW-LENGTH(FEET) =   105.00
 UPSTREAM ELEVATION(FEET) =     26.30
 DOWNSTREAM ELEVATION(FEET) =     25.60
 ELEVATION DIFFERENCE(FEET) =      0.70
 URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.112
 WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
          THE MAXIMUM OVERLAND FLOW LENGTH =    55.00
          (Reference: Table 3-1B of Hydrology Manual)
          THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.166
 SUBAREA RUNOFF(CFS) =      0.29
 TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.29

****************************************************************************
 FLOW PROCESS FROM NODE    109.00 TO NODE    105.00 IS CODE =  51
----------------------------------------------------------------------------
 >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =     25.60  DOWNSTREAM(FEET) =     21.70
 CHANNEL LENGTH THRU SUBAREA(FEET) =   430.00   CHANNEL SLOPE =  0.0091
 CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
 MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   1.00
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.205
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
 S.C.S. CURVE NUMBER (AMC II) =  45
 TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       2.06
 TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.23
 AVERAGE FLOW DEPTH(FEET) =   0.06   TRAVEL TIME(MIN.) =   5.84
 Tc(MIN.) =   11.95
 SUBAREA AREA(ACRES) =     1.56       SUBAREA RUNOFF(CFS) =    3.50
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.700
 TOTAL AREA(ACRES) =        1.7         PEAK FLOW RATE(CFS) =       3.72

 END OF SUBAREA CHANNEL FLOW HYDRAULICS:
 DEPTH(FEET) =  0.09   FLOW VELOCITY(FEET/SEC.) =   1.50
 LONGEST FLOWPATH FROM NODE    110.00 TO NODE    105.00 =     535.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    105.00 TO NODE    105.00 IS CODE =   1
```

```
------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
========================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  3 ARE:
  TIME OF CONCENTRATION(MIN.) =   11.95
  RAINFALL INTENSITY(INCH/HR) =   3.20
  TOTAL STREAM AREA(ACRES) =     1.66
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      3.72

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       12.35    15.83        2.817          6.10
      2        6.47    15.20        2.880          3.21
      3        3.72    11.95        3.205          1.66

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  3 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1       19.66    11.95       3.205
      2       21.89    15.20       2.880
      3       21.95    15.83       2.817

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      21.95   Tc(MIN.) =   15.83
  TOTAL AREA(ACRES) =       11.0
  LONGEST FLOWPATH FROM NODE    140.00 TO NODE    105.00 =    1100.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    105.00 TO NODE    100.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    16.50  DOWNSTREAM(FEET) =    16.30
  FLOW LENGTH(FEET) =    50.00   MANNING'S N =  0.011
  DEPTH OF FLOW IN  27.0 INCH PIPE IS  22.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   6.30
  ESTIMATED PIPE DIAMETER(INCH) =  27.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      21.95
  PIPE TRAVEL TIME(MIN.) =   0.13    Tc(MIN.) =   15.97
  LONGEST FLOWPATH FROM NODE    140.00 TO NODE    100.00 =    1150.00 FEET.
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    100.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =   15.97
  RAINFALL INTENSITY(INCH/HR) =   2.80
  TOTAL STREAM AREA(ACRES) =    10.97
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     21.95

****************************************************************************
  FLOW PROCESS FROM NODE    205.00 TO NODE    204.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =  45
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   180.00
  UPSTREAM ELEVATION(FEET) =     20.00
  DOWNSTREAM ELEVATION(FEET) =     19.80
  ELEVATION DIFFERENCE(FEET) =      0.20
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =   10.423
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    50.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.387
  SUBAREA RUNOFF(CFS) =      0.30
  TOTAL AREA(ACRES) =      0.20   TOTAL RUNOFF(CFS) =      0.30

****************************************************************************
  FLOW PROCESS FROM NODE    204.00 TO NODE    204.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.387
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =  45
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4500
  SUBAREA AREA(ACRES) =    0.90   SUBAREA RUNOFF(CFS) =    1.37
  TOTAL AREA(ACRES) =        1.1   TOTAL RUNOFF(CFS) =       1.68
  TC(MIN.) =   10.42

****************************************************************************
```

```
 FLOW PROCESS FROM NODE    204.00 TO NODE    200.00 IS CODE =  52
----------------------------------------------------------------------------
 >>>>>COMPUTE NATURAL VALLEY CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =     19.80  DOWNSTREAM(FEET) =     18.60
 CHANNEL LENGTH THRU SUBAREA(FEET) =   275.00   CHANNEL SLOPE =  0.0044
 CHANNEL FLOW THRU SUBAREA(CFS) =       1.68
 FLOW VELOCITY(FEET/SEC) =   1.10 (PER LACFCD/RCFC&WCD HYDROLOGY MANUAL)
 TRAVEL TIME(MIN.) =   4.18   Tc(MIN.) =   14.60
 LONGEST FLOWPATH FROM NODE    205.00 TO NODE    200.00 =     455.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    200.00 TO NODE    100.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    17.10  DOWNSTREAM(FEET) =    16.30
 FLOW LENGTH(FEET) =   220.00   MANNING'S N =  0.011
 DEPTH OF FLOW IN  12.0 INCH PIPE IS   7.4 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   3.32
 ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =       1.68
 PIPE TRAVEL TIME(MIN.) =   1.11    Tc(MIN.) =   15.71
 LONGEST FLOWPATH FROM NODE    205.00 TO NODE    100.00 =     675.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    100.00 TO NODE    100.00 IS CODE =   1
----------------------------------------------------------------------------
 >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
 >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
============================================================================
 TOTAL NUMBER OF STREAMS =  2
 CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
 TIME OF CONCENTRATION(MIN.) =   15.71
 RAINFALL INTENSITY(INCH/HR) =   2.83
 TOTAL STREAM AREA(ACRES) =     1.10
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      1.68

 ** CONFLUENCE DATA **
 STREAM     RUNOFF       Tc      INTENSITY      AREA
 NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
     1       21.95    15.97        2.803         10.97
     2        1.68    15.71        2.829          1.10

 RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
 CONFLUENCE FORMULA USED FOR  2 STREAMS.
```

```
  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1       23.43    15.71       2.829
      2       23.61    15.97       2.803

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      23.61   Tc(MIN.) =   15.97
  TOTAL AREA(ACRES) =       12.1
  LONGEST FLOWPATH FROM NODE    140.00 TO NODE    100.00 =    1150.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    305.00 TO NODE    304.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =  45
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   200.00
  UPSTREAM ELEVATION(FEET) =     21.00
  DOWNSTREAM ELEVATION(FEET) =     20.10
  ELEVATION DIFFERENCE(FEET) =      0.90
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =   10.423
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    50.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.387
  SUBAREA RUNOFF(CFS) =      0.30
  TOTAL AREA(ACRES) =      0.20   TOTAL RUNOFF(CFS) =      0.30

****************************************************************************
  FLOW PROCESS FROM NODE    304.00 TO NODE    304.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.387
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =  45
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4500
  SUBAREA AREA(ACRES) =    2.09   SUBAREA RUNOFF(CFS) =    3.22
  TOTAL AREA(ACRES) =        2.3   TOTAL RUNOFF(CFS) =       3.52
  TC(MIN.) =   10.42

****************************************************************************
```

```
 FLOW PROCESS FROM NODE    304.00 TO NODE    300.00 IS CODE =  52
----------------------------------------------------------------------------
 >>>>>COMPUTE NATURAL VALLEY CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =     20.10  DOWNSTREAM(FEET) =     19.10
 CHANNEL LENGTH THRU SUBAREA(FEET) =    60.00   CHANNEL SLOPE =  0.0167
 CHANNEL FLOW THRU SUBAREA(CFS) =       3.52
 FLOW VELOCITY(FEET/SEC) =   2.51 (PER LACFCD/RCFC&WCD HYDROLOGY MANUAL)
 TRAVEL TIME(MIN.) =   0.40   Tc(MIN.) =   10.82
 LONGEST FLOWPATH FROM NODE    305.00 TO NODE    300.00 =     260.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    405.00 TO NODE    400.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
 S.C.S. CURVE NUMBER (AMC II) =  45
 INITIAL SUBAREA FLOW-LENGTH(FEET) =    60.00
 UPSTREAM ELEVATION(FEET) =     24.00
 DOWNSTREAM ELEVATION(FEET) =     18.70
 ELEVATION DIFFERENCE(FEET) =      5.30
 URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    4.384
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.400
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      0.18
 TOTAL AREA(ACRES) =      0.09   TOTAL RUNOFF(CFS) =      0.18

****************************************************************************
 FLOW PROCESS FROM NODE    490.00 TO NODE    494.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
 S.C.S. CURVE NUMBER (AMC II) =  45
 INITIAL SUBAREA FLOW-LENGTH(FEET) =    70.00
 UPSTREAM ELEVATION(FEET) =     26.80
 DOWNSTREAM ELEVATION(FEET) =     25.40
 ELEVATION DIFFERENCE(FEET) =      1.40
 URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    4.781
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.400
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      0.31
 TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.31
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    494.00 TO NODE    495.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     25.40  DOWNSTREAM(FEET) =     23.20
  CHANNEL LENGTH THRU SUBAREA(FEET) =    75.00   CHANNEL SLOPE =  0.0293
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  99.000
  MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   1.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.242
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (24. DU/AC OR LESS) RUNOFF COEFFICIENT = .7000
  S.C.S. CURVE NUMBER (AMC II) =  45
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.55
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.24
  AVERAGE FLOW DEPTH(FEET) =   0.02   TRAVEL TIME(MIN.) =   1.01
  Tc(MIN.) =    5.79
  SUBAREA AREA(ACRES) =     0.16       SUBAREA RUNOFF(CFS) =    0.48
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.700
  TOTAL AREA(ACRES) =        0.3         PEAK FLOW RATE(CFS) =       0.77

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.03   FLOW VELOCITY(FEET/SEC.) =   1.33
  LONGEST FLOWPATH FROM NODE    490.00 TO NODE    495.00 =     145.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    495.00 TO NODE    500.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    18.30  DOWNSTREAM(FEET) =    17.50
  FLOW LENGTH(FEET) =    80.00   MANNING'S N =  0.011
  DEPTH OF FLOW IN   9.0 INCH PIPE IS   4.0 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.02
  ESTIMATED PIPE DIAMETER(INCH) =   9.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       0.77
  PIPE TRAVEL TIME(MIN.) =   0.33    Tc(MIN.) =    6.12
  LONGEST FLOWPATH FROM NODE    490.00 TO NODE    500.00 =     225.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    500.00 TO NODE    500.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
```

```
  TIME OF CONCENTRATION(MIN.) =    6.12
  RAINFALL INTENSITY(INCH/HR) =   4.16
  TOTAL STREAM AREA(ACRES) =     0.26
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      0.77

****************************************************************************
  FLOW PROCESS FROM NODE    510.00 TO NODE    509.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (14.5 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =  45
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    80.00
  UPSTREAM ELEVATION(FEET) =     25.00
  DOWNSTREAM ELEVATION(FEET) =     23.40
  ELEVATION DIFFERENCE(FEET) =      1.60
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    7.667
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.800
  SUBAREA RUNOFF(CFS) =      0.47
  TOTAL AREA(ACRES) =      0.25   TOTAL RUNOFF(CFS) =      0.47

****************************************************************************
  FLOW PROCESS FROM NODE    509.00 TO NODE    500.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =   23.40  DOWNSTREAM ELEVATION(FEET) =   22.10
  STREET LENGTH(FEET) =   360.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 27.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  22.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0150
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.45
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.32
    HALFSTREET FLOOD WIDTH(FEET) =    9.86
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    1.33
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.43
  STREET FLOW TRAVEL TIME(MIN.) =   4.51   Tc(MIN.) =   12.18
```

```
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.182
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (14.5 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
 S.C.S. CURVE NUMBER (AMC II) =  45
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.500
 SUBAREA AREA(ACRES) =    1.22      SUBAREA RUNOFF(CFS) =    1.94
 TOTAL AREA(ACRES) =        1.5        PEAK FLOW RATE(CFS) =       2.34

 END OF SUBAREA STREET FLOW HYDRAULICS:
 DEPTH(FEET) = 0.37   HALFSTREET FLOOD WIDTH(FEET) =  12.00
 FLOW VELOCITY(FEET/SEC.) =  1.50   DEPTH*VELOCITY(FT*FT/SEC.) =   0.55
 LONGEST FLOWPATH FROM NODE    510.00 TO NODE    500.00 =     440.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    500.00 TO NODE    500.00 IS CODE =   1
----------------------------------------------------------------------------
 >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
 TOTAL NUMBER OF STREAMS =  3
 CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
 TIME OF CONCENTRATION(MIN.) =   12.18
 RAINFALL INTENSITY(INCH/HR) =   3.18
 TOTAL STREAM AREA(ACRES) =     1.47
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      2.34

****************************************************************************
 FLOW PROCESS FROM NODE    515.00 TO NODE    514.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 RESIDENTIAL (14.5 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
 S.C.S. CURVE NUMBER (AMC II) =  45
 INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
 UPSTREAM ELEVATION(FEET) =     34.80
 DOWNSTREAM ELEVATION(FEET) =     31.40
 ELEVATION DIFFERENCE(FEET) =      3.40
 URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.889
 WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
          THE MAXIMUM OVERLAND FLOW LENGTH =    92.00
          (Reference: Table 3-1B of Hydrology Manual)
          THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
 100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.933
 SUBAREA RUNOFF(CFS) =      0.45
 TOTAL AREA(ACRES) =      0.23   TOTAL RUNOFF(CFS) =      0.45

****************************************************************************
 FLOW PROCESS FROM NODE    514.00 TO NODE    500.00 IS CODE =  62
```

```
------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  1 USED)<<<<<
========================================================================
  UPSTREAM ELEVATION(FEET) =   31.40  DOWNSTREAM ELEVATION(FEET) =   22.10
  STREET LENGTH(FEET) =   450.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 27.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  22.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  1
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0150
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0200

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.53
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.26
    HALFSTREET FLOOD WIDTH(FEET) =    6.76
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.66
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.70
  STREET FLOW TRAVEL TIME(MIN.) =   2.82   Tc(MIN.) =    9.71
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.493
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (14.5 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =  45
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.500
  SUBAREA AREA(ACRES) =    1.23      SUBAREA RUNOFF(CFS) =    2.15
  TOTAL AREA(ACRES) =        1.5        PEAK FLOW RATE(CFS) =       2.55

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.30   HALFSTREET FLOOD WIDTH(FEET) =   8.57
  FLOW VELOCITY(FEET/SEC.) =  2.99   DEPTH*VELOCITY(FT*FT/SEC.) =   0.89
  LONGEST FLOWPATH FROM NODE    515.00 TO NODE    500.00 =     550.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    500.00 TO NODE    500.00 IS CODE =   1
------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
========================================================================
  TOTAL NUMBER OF STREAMS =  3
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  3 ARE:
  TIME OF CONCENTRATION(MIN.) =    9.71
  RAINFALL INTENSITY(INCH/HR) =   3.49
  TOTAL STREAM AREA(ACRES) =     1.46
```

```
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      2.55

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1        0.77     6.12        4.164          0.26
      2        2.34    12.18        3.182          1.47
      3        2.55     9.71        3.493          1.46

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  3 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1        3.56     6.12       4.164
      2        5.06     9.71       3.493
      3        5.25    12.18       3.182

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       5.25   Tc(MIN.) =   12.18
  TOTAL AREA(ACRES) =        3.2
  LONGEST FLOWPATH FROM NODE    515.00 TO NODE    500.00 =     550.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    500.00 TO NODE    600.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    17.20  DOWNSTREAM(FEET) =    15.40
  FLOW LENGTH(FEET) =   750.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  21.0 INCH PIPE IS  13.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   3.33
  ESTIMATED PIPE DIAMETER(INCH) =  21.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       5.25
  PIPE TRAVEL TIME(MIN.) =   3.76    Tc(MIN.) =   15.94
  LONGEST FLOWPATH FROM NODE    515.00 TO NODE    600.00 =    1300.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    600.00 TO NODE    600.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =   15.94
  RAINFALL INTENSITY(INCH/HR) =   2.81
```

```
  TOTAL STREAM AREA(ACRES) =     3.19
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      5.25

****************************************************************************
  FLOW PROCESS FROM NODE    510.00 TO NODE    604.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (14.5 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =  45
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    60.00
  UPSTREAM ELEVATION(FEET) =     25.00
  DOWNSTREAM ELEVATION(FEET) =     23.50
  ELEVATION DIFFERENCE(FEET) =      1.50
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.164
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.151
  SUBAREA RUNOFF(CFS) =      0.21
  TOTAL AREA(ACRES) =      0.10   TOTAL RUNOFF(CFS) =      0.21

****************************************************************************
  FLOW PROCESS FROM NODE    604.00 TO NODE    600.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     23.50  DOWNSTREAM(FEET) =     20.30
  CHANNEL LENGTH THRU SUBAREA(FEET) =   120.00   CHANNEL SLOPE =  0.0267
  CHANNEL BASE(FEET) =   20.00   "Z" FACTOR =  50.000
  MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   0.50
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.788
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (14.5 DU/AC OR LESS) RUNOFF COEFFICIENT = .5000
  S.C.S. CURVE NUMBER (AMC II) =  45
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       0.53
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.26
  AVERAGE FLOW DEPTH(FEET) =   0.02   TRAVEL TIME(MIN.) =   1.59
  Tc(MIN.) =    7.75
  SUBAREA AREA(ACRES) =     0.34       SUBAREA RUNOFF(CFS) =    0.64
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.500
  TOTAL AREA(ACRES) =        0.4         PEAK FLOW RATE(CFS) =       0.83

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.03   FLOW VELOCITY(FEET/SEC.) =   1.41
  LONGEST FLOWPATH FROM NODE    510.00 TO NODE    600.00 =     180.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    600.00 TO NODE    600.00 IS CODE =   1
```

```
--------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
==========================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =    7.75
  RAINFALL INTENSITY(INCH/HR) =   3.79
  TOTAL STREAM AREA(ACRES) =     0.44
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      0.83

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1        5.25    15.94        2.806          3.19
      2        0.83     7.75        3.788          0.44

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1        4.72     7.75       3.788
      2        5.87    15.94       2.806

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       5.87   Tc(MIN.) =   15.94
  TOTAL AREA(ACRES) =        3.6
  LONGEST FLOWPATH FROM NODE    515.00 TO NODE    600.00 =    1300.00 FEET.
==========================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        3.6  TC(MIN.) =     15.94
  PEAK FLOW RATE(CFS)   =       5.87
==========================================================================
==========================================================================
  END OF RATIONAL METHOD ANALYSIS
```

APPENDIX 5

PROPOSED MITIGATED HYDROLOGY CALCULATIONS

Job Name: BELLA MAR
Job #: 1621-001
Run Name: BMPR
Date: 2/11/2019

MITIGATED PROPOSED HYDROLOGY - 100 YR

Node to Node		Code	Elev 1 (feet)	Elev 2 (feet)	Length (feet)	Runoff Coeff.	Area (ac.)	Comments	BANK 1	2	3
105	105	7	A=10.97	Tc=15.83	Q=15.86		**10.97**	**Detention Basin Outflow**			
105	100	3	16.5	16.3	50.0			Pipe Flow			
100	100	1						Confluence: 1 of 2			
205	204	2	20.0	19.8	180.0	0.45	0.20	Basin B: Initial Area			
204	204	8				0.45	0.90	Addition Subarea			
204	200	5	19.8	18.6	275.0			Open Channel Flow			
200	100	3	17.1	16.3	220.0			Pipe Flow			
100	100	1						Confluence: 2 of 2: POC1			
							12.07	**Total Tributary Area to POC1**			

```
**************************************************************************

            RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                         2003,1985,1981 HYDROLOGY MANUAL
          (c) Copyright 1982-2014 Advanced Engineering Software (aes)
              Ver. 21.0 Release Date: 06/01/2014  License ID 1355

                              Analysis prepared by:

                                Fuscoe Engineering
                               6390 Greenwich Drive
                                    Suite 170
                               San Diego, CA 92122

 ************************* DESCRIPTION OF STUDY *************************
* BELLA MAR                                                             *
* MITIGATED DEVELOPED CONDITIONS - 100 YR                               *
* DETENTION AT NODE 105                                                 *
 *************************************************************************

  FILE NAME: BMPRMIT.DAT
  TIME/DATE OF STUDY: 07:38 02/13/2019
--------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
--------------------------------------------------------------------------
  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  SPECIFIED MINIMUM PIPE SIZE(INCH) =   6.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  RAINFALL-INTENSITY ADJUSTMENT FACTOR = 1.000
  *USER SPECIFIED:
  NUMBER OF [TIME,INTENSITY] DATA PAIRS = 20
   1)   5.000;  4.400
   2)   6.000;  4.200
   3)   7.000;  3.900
   4)   8.000;  3.750
   5)   9.000;  3.600
   6)  10.000;  3.450
   7)  11.000;  3.300
   8)  12.000;  3.200
   9)  14.000;  3.000
  10)  15.000;  2.900
  11)  16.000;  2.800
  12)  17.000;  2.700
  13)  19.000;  2.600
  14)  20.000;  2.550
  15)  25.000;  2.230
```

```
  16)  30.000;  2.000
  17)  40.000;  1.700
  18)  50.000;  1.500
  19)  60.000;  1.310
  20) 120.000;  0.860
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: ONLY PEAK CONFLUENCE VALUES CONSIDERED
  *USER-DEFINED STREET-SECTIONS FOR COUPLED PIPEFLOW AND STREETFLOW MODEL*
     HALF-  CROWN TO   STREET-CROSSFALL:   CURB  GUTTER-GEOMETRIES:  MANNING
     WIDTH  CROSSFALL  IN-  / OUT-/PARK-  HEIGHT  WIDTH  LIP   HIKE  FACTOR
NO.   (FT)     (FT)    SIDE / SIDE/ WAY    (FT)    (FT)  (FT)  (FT)    (n)
===  =====  =========  =================  ======  ===== ====== ===== =======
  1   27.0     22.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0150

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  0.00 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  6.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*

****************************************************************************
  FLOW PROCESS FROM NODE    105.00 TO NODE    105.00 IS CODE =   7
----------------------------------------------------------------------------
  >>>>>USER SPECIFIED HYDROLOGY INFORMATION AT NODE<<<<<
============================================================================
  USER-SPECIFIED VALUES ARE AS FOLLOWS:
  TC(MIN) =  15.83   RAIN INTENSITY(INCH/HOUR) =  2.82
  TOTAL AREA(ACRES) =    10.97   TOTAL RUNOFF(CFS) =     15.86

****************************************************************************
  FLOW PROCESS FROM NODE    105.00 TO NODE    100.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    16.50  DOWNSTREAM(FEET) =    16.30
  FLOW LENGTH(FEET) =    50.00   MANNING'S N =  0.011
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  19.4 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   5.82
  ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      15.86
  PIPE TRAVEL TIME(MIN.) =   0.14    Tc(MIN.) =   15.97
  LONGEST FLOWPATH FROM NODE      0.00 TO NODE    100.00 =      50.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    100.00 IS CODE =   1
----------------------------------------------------------------------------
```

```
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
==============================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  1 ARE:
  TIME OF CONCENTRATION(MIN.) =   15.97
  RAINFALL INTENSITY(INCH/HR) =   2.80
  TOTAL STREAM AREA(ACRES) =    10.97
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     15.86

******************************************************************************
  FLOW PROCESS FROM NODE    205.00 TO NODE    204.00 IS CODE =  21
------------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
==============================================================================
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   180.00
  UPSTREAM ELEVATION(FEET) =     20.00
  DOWNSTREAM ELEVATION(FEET) =     19.80
  ELEVATION DIFFERENCE(FEET) =      0.20
  URBAN SUBAREA OVERLAND TIME OF FLOW(MIN.) =   10.423
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    50.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.387
  SUBAREA RUNOFF(CFS) =      0.30
  TOTAL AREA(ACRES) =      0.20   TOTAL RUNOFF(CFS) =      0.30

******************************************************************************
  FLOW PROCESS FROM NODE    204.00 TO NODE    204.00 IS CODE =  81
------------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.387
  *USER SPECIFIED(SUBAREA):
  RESIDENTIAL (1. DU/AC OR LESS) RUNOFF COEFFICIENT = .4500
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.4500
  SUBAREA AREA(ACRES) =    0.90   SUBAREA RUNOFF(CFS) =    1.37
  TOTAL AREA(ACRES) =        1.1   TOTAL RUNOFF(CFS) =       1.68
  TC(MIN.) =   10.42

******************************************************************************
  FLOW PROCESS FROM NODE    204.00 TO NODE    200.00 IS CODE =  52
------------------------------------------------------------------------------
  >>>>>COMPUTE NATURAL VALLEY CHANNEL FLOW<<<<<
```

```
  >>>>>TRAVELTIME THRU SUBAREA<<<<<
==========================================================================
  ELEVATION DATA: UPSTREAM(FEET) =     19.80  DOWNSTREAM(FEET) =     18.60
  CHANNEL LENGTH THRU SUBAREA(FEET) =   275.00   CHANNEL SLOPE =  0.0044
  CHANNEL FLOW THRU SUBAREA(CFS) =       1.68
  FLOW VELOCITY(FEET/SEC) =   1.10 (PER LACFCD/RCFC&WCD HYDROLOGY MANUAL)
  TRAVEL TIME(MIN.) =   4.18   Tc(MIN.) =   14.60
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    200.00 =     455.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    200.00 TO NODE    100.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
==========================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    17.10  DOWNSTREAM(FEET) =    16.30
  FLOW LENGTH(FEET) =   220.00   MANNING'S N =  0.011
  DEPTH OF FLOW IN  12.0 INCH PIPE IS   7.4 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   3.32
  ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       1.68
  PIPE TRAVEL TIME(MIN.) =   1.11    Tc(MIN.) =   15.71
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    100.00 =     675.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    100.00 TO NODE    100.00 IS CODE =   1
----------------------------------------------------------------------------
  >>>>>DESIGNATE INDEPENDENT STREAM FOR CONFLUENCE<<<<<
  >>>>>AND COMPUTE VARIOUS CONFLUENCED STREAM VALUES<<<<<
==========================================================================
  TOTAL NUMBER OF STREAMS =  2
  CONFLUENCE VALUES USED FOR INDEPENDENT STREAM  2 ARE:
  TIME OF CONCENTRATION(MIN.) =   15.71
  RAINFALL INTENSITY(INCH/HR) =   2.83
  TOTAL STREAM AREA(ACRES) =     1.10
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      1.68

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       15.86    15.97        2.803         10.97
      2        1.68    15.71        2.829          1.10

  RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO
  CONFLUENCE FORMULA USED FOR  2 STREAMS.

  ** PEAK FLOW RATE TABLE **
  STREAM     RUNOFF      Tc      INTENSITY
```

```
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)
      1       17.27    15.71       2.829
      2       17.52    15.97       2.803

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      17.52   Tc(MIN.) =   15.97
  TOTAL AREA(ACRES) =       12.1
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    100.00 =     675.00 FEET.
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =       12.1  TC(MIN.) =     15.97
  PEAK FLOW RATE(CFS)   =      17.52
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```



APPENDIX 6

HYDRAULIC CALCULATIONS

DETENTION BASIN ATTENUATION

HOLLISTER PUBLIC STORM DRAIN PIPE

DETENTION BASIN 100-YR STORM ATTENUATION

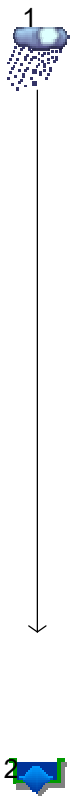
DETENTION BASIN INFLOW HYDROGRAPH

RATIONAL METHOD HYDROGRAPH PROGRAM
COPYRIGHT 1992, 2001 RICK ENGINEERING COMPANY

RUN DATE 2/12/2019
HYDROGRAPH FILE NAME Text1
TIME OF CONCENTRATION 16 MIN.
6 HOUR RAINFALL 2.5 INCHES
BASIN AREA 10.97 ACRES
RUNOFF COEFFICIENT 0.7
PEAK DISCHARGE 21.95 CFS

TIME (MIN) = 0	DISCHARGE (CFS) = 0
TIME (MIN) = 16	DISCHARGE (CFS) = 1.1
TIME (MIN) = 32	DISCHARGE (CFS) = 1.2
TIME (MIN) = 48	DISCHARGE (CFS) = 1.2
TIME (MIN) = 64	DISCHARGE (CFS) = 1.3
TIME (MIN) = 80	DISCHARGE (CFS) = 1.4
TIME (MIN) = 96	DISCHARGE (CFS) = 1.5
TIME (MIN) = 112	DISCHARGE (CFS) = 1.6
TIME (MIN) = 128	DISCHARGE (CFS) = 1.8
TIME (MIN) = 144	DISCHARGE (CFS) = 1.9
TIME (MIN) = 160	DISCHARGE (CFS) = 2.1
TIME (MIN) = 176	DISCHARGE (CFS) = 2.3
TIME (MIN) = 192	DISCHARGE (CFS) = 2.8
TIME (MIN) = 208	DISCHARGE (CFS) = 3.2
TIME (MIN) = 224	DISCHARGE (CFS) = 4.7
TIME (MIN) = 240	DISCHARGE (CFS) = 8.6
TIME (MIN) = 256	DISCHARGE (CFS) = 21.95
TIME (MIN) = 272	DISCHARGE (CFS) = 3.8
TIME (MIN) = 288	DISCHARGE (CFS) = 2.5
TIME (MIN) = 304	DISCHARGE (CFS) = 2
TIME (MIN) = 320	DISCHARGE (CFS) = 1.7
TIME (MIN) = 336	DISCHARGE (CFS) = 1.4
TIME (MIN) = 352	DISCHARGE (CFS) = 1.3
TIME (MIN) = 368	DISCHARGE (CFS) = 1.2
TIME (MIN) = 384	DISCHARGE (CFS) = 0

Watershed Model Schematic



Legend

Hyd.	Origin	Description
1	Manual	Unmitigated Inflow
2	Reservoir	Mitigated

Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020

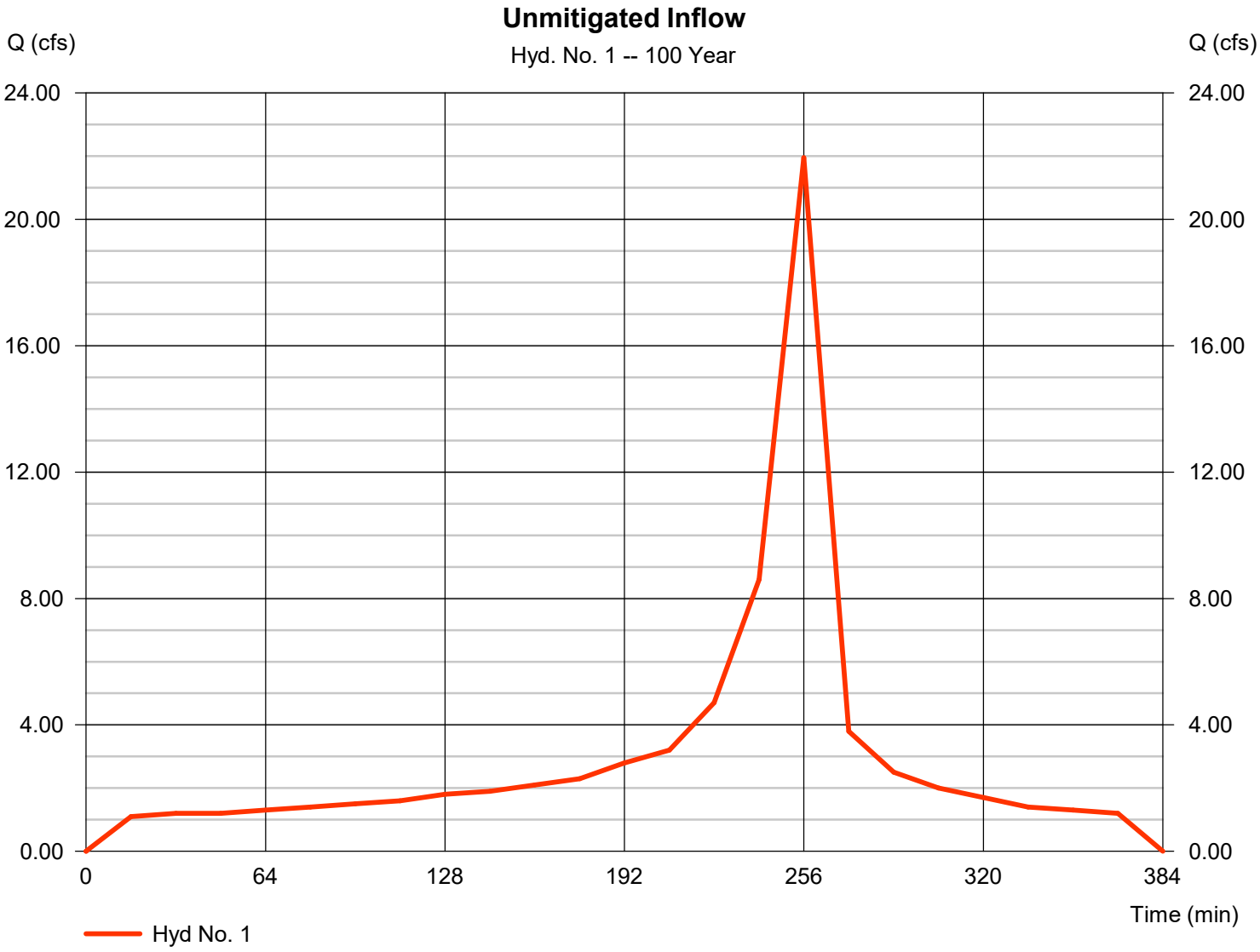
Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	Manual	21.95	16	256	69,648	------	------	------	Unmitigated Inflow
2	Reservoir	15.86	16	256	69,644	1	20.51	8,446	Mitigated

Hydrograph Report

Hyd. No. 1

Unmitigated Inflow

Hydrograph type	= Manual	Peak discharge	= 21.95 cfs
Storm frequency	= 100 yrs	Time to peak	= 256 min
Time interval	= 16 min	Hyd. volume	= 69,648 cuft



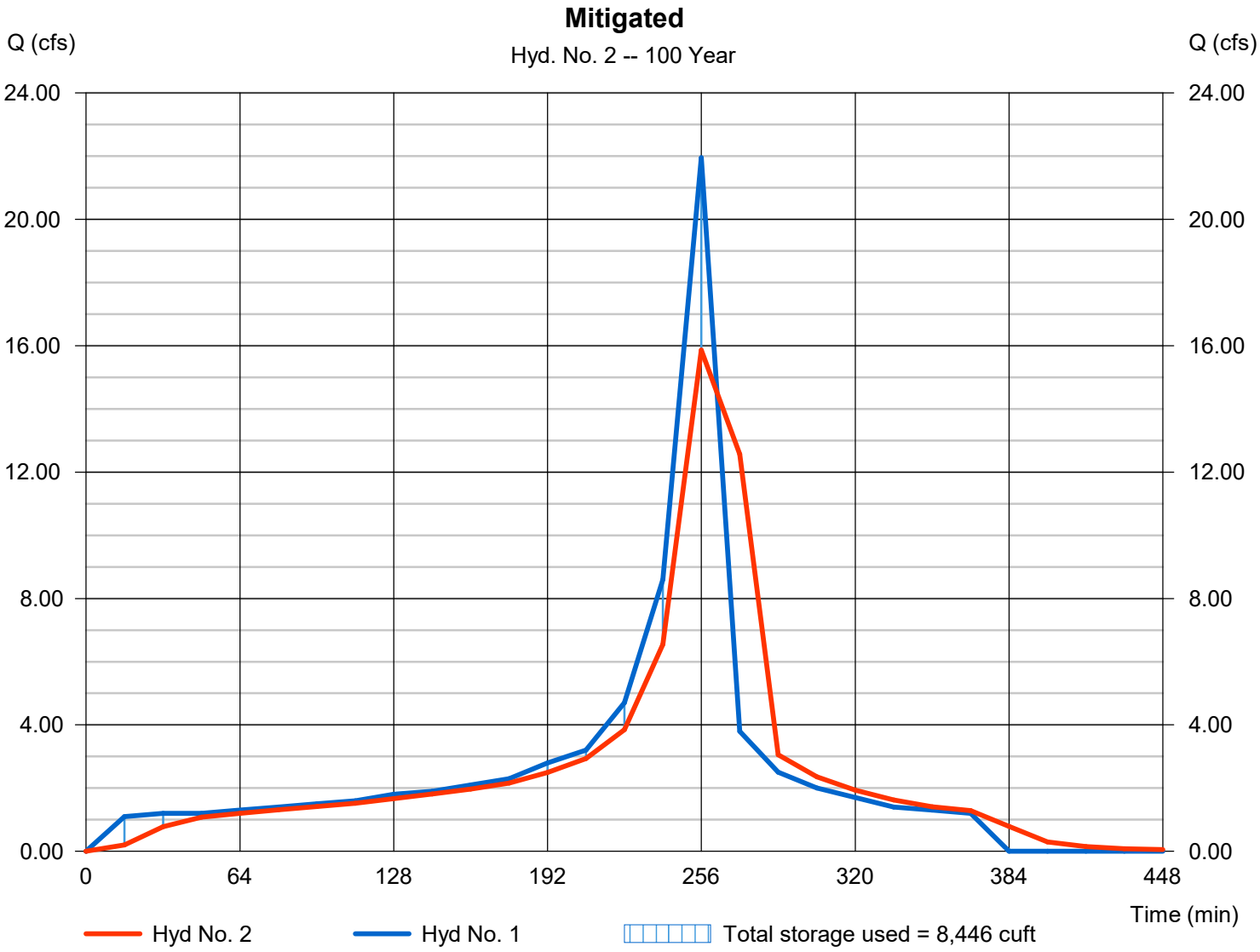
Hydrograph Report

Hyd. No. 2

Mitigated

Hydrograph type	=	Reservoir	Peak discharge	=	15.86 cfs
Storm frequency	=	100 yrs	Time to peak	=	256 min
Time interval	=	16 min	Hyd. volume	=	69,644 cuft
Inflow hyd. No.	=	1 - Unmitigated Inflow	Max. Elevation	=	20.51 ft
Reservoir name	=	Detention	Max. Storage	=	8,446 cuft

Storage Indication method used.



Pond No. 2 - Detention

Pond Data

Contours -User-defined contour areas. Average end area method used for volume calculation. Begining Elevation = 19.70 ft

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	19.70	12,740	0	0
0.10	19.80	13,045	1,289	1,289
0.20	19.90	13,350	1,320	2,609
0.30	20.00	13,660	1,351	3,960
0.40	20.10	13,965	1,381	5,341
0.50	20.20	14,273	1,412	6,753
0.60	20.30	14,581	1,443	8,195
0.70	20.40	14,890	1,474	9,669
0.80	20.50	15,200	1,505	11,173
0.90	20.60	15,510	1,536	12,709
1.00	20.70	15,820	1,567	14,275
1.10	20.80	16,132	1,598	15,873
1.20	20.90	16,444	1,629	17,502
1.30	21.00	16,756	1,660	19,162
1.40	21.10	17,069	1,691	20,853
1.50	21.20	17,382	1,723	22,576

Culvert / Orifice Structures

	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 24.00	0.00	0.00	0.00
Span (in)	= 24.00	0.00	0.00	0.00
No. Barrels	= 1	0	0	0
Invert El. (ft)	= 16.35	0.00	0.00	0.00
Length (ft)	= 10.00	0.00	0.00	0.00
Slope (%)	= 0.50	0.00	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 9.83	0.00	0.00	0.00
Crest El. (ft)	= 19.70	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= 1	---	---	---
Multi-Stage	= Yes	No	No	No

Exfil.(in/hr) = 0.000 (by Wet area)

TW Elev. (ft) = 0.00

Note: Culvert/Orifice outflows are analyzed under inlet (ic) and outlet (oc) control. Weir risers checked for orifice conditions (ic) and submergence (s).

Stage / Storage / Discharge Table

Stage ft	Storage cuft	Elevation ft	Clv A cfs	Clv B cfs	Clv C cfs	PrfRsr cfs	Wr A cfs	Wr B cfs	Wr C cfs	Wr D cfs	Exfil cfs	User cfs	Total cfs
0.00	0	19.70	0.00	---	---	---	0.00	---	---	---	---	---	0.000
0.10	1,289	19.80	23.19 ic	---	---	---	1.04	---	---	---	---	---	1.035
0.20	2,609	19.90	23.19 ic	---	---	---	2.93	---	---	---	---	---	2.928
0.30	3,960	20.00	23.19 ic	---	---	---	5.38	---	---	---	---	---	5.379
0.40	5,341	20.10	23.19 ic	---	---	---	8.28	---	---	---	---	---	8.281
0.50	6,753	20.20	23.19 ic	---	---	---	11.57	---	---	---	---	---	11.57
0.60	8,195	20.30	23.19 ic	---	---	---	15.21	---	---	---	---	---	15.21
0.70	9,669	20.40	23.19 ic	---	---	---	19.17	---	---	---	---	---	19.17
0.80	11,173	20.50	23.35 ic	---	---	---	23.35 s	---	---	---	---	---	23.35
0.90	12,709	20.60	24.92 ic	---	---	---	24.92 s	---	---	---	---	---	24.92
1.00	14,275	20.70	25.94 ic	---	---	---	25.93 s	---	---	---	---	---	25.93
1.10	15,873	20.80	26.74 ic	---	---	---	26.74 s	---	---	---	---	---	26.74
1.20	17,502	20.90	27.42 ic	---	---	---	27.42 s	---	---	---	---	---	27.42
1.30	19,162	21.00	28.02 ic	---	---	---	28.02 s	---	---	---	---	---	28.02
1.40	20,853	21.10	28.57 ic	---	---	---	28.56 s	---	---	---	---	---	28.56
1.50	22,576	21.20	29.07 ic	---	---	---	29.07 s	---	---	---	---	---	29.07

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Hydraulic Calculation for Public 18" RCP SD in Hollister

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.00300	ft/ft
Diameter	1.50	ft
Discharge	5.87	ft³/s

Results

Normal Depth	1.26	ft
Flow Area	1.58	ft²
Wetted Perimeter	3.47	ft
Hydraulic Radius	0.46	ft
Top Width	1.10	ft
Critical Depth	0.94	ft
Percent Full	83.9	%
Critical Slope	0.00616	ft/ft
Velocity	3.71	ft/s
Velocity Head	0.21	ft
Specific Energy	1.47	ft
Froude Number	0.55	
Maximum Discharge	6.19	ft³/s
Discharge Full	5.75	ft³/s
Slope Full	0.00312	ft/ft
Flow Type	SubCritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	83.92	%
Downstream Velocity	Infinity	ft/s

Hydraulic Calculation for Public 18" RCP SD in Hollister

GVF Output Data

Upstream Velocity	Infinity	ft/s
Normal Depth	1.26	ft
Critical Depth	0.94	ft
Channel Slope	0.00300	ft/ft
Critical Slope	0.00616	ft/ft

Cross Section for Public 18" RCP SD in Hollister

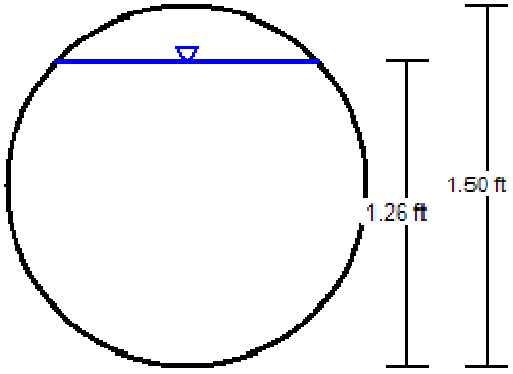
Project Description

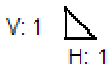
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.00300	ft/ft
Normal Depth	1.26	ft
Diameter	1.50	ft
Discharge	5.87	ft³/s

Cross Section Image





APPENDIX 7

AS-BUILT REFERENCES

ATTACHMENT 1
SITE PLAN



ATTACHMENT 2
CALTRANS AS-BUILT

DIST.	COUNTY	ROUTE	POST MILES—TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
11	SD	5	4.4/9.0	42	437

DISTRICT DESIGN ENGINEER Henry R. Obrecht REGISTERED CIVIL ENGINEER No. 15768
DATE APPROVED July 27, 1970

14.9' IE-OUT (NAVD 88)

16.2' IE-IN (NAVD 88)

L0
Δ = 04° 36' 28"
R = 10,000'
T = 402.32'
L = 804.21'

275+59.38 EC L0

Otay Riv Br
Br. No. 57-246R/L

Otay Riv Overflow Br
Br. No. 57-263R/L

R/W

S Dike

Graded "V" Gutter 2:1 SS

N06°54'00"W 285

2693.09'

Profile Grade

Ground Line

-0.38%

NOTE
Profile Grade is 15' Lt & Rt of ℄ L0

U.S.C & G.S. Datum

"AS BUILT"
CONTRACT 122404
YEAR COMPLETED 8-1-73

DRAINAGE DETAILS AND CONTOUR GRADING PLANS

Project Engineer	Date	Design Engineer	Date	Approval Recommended By	Date

AS BUILT PLANS
Contract No. 11-122404
Date Completed 07-70
Document No. 11-000260

FORM WH23A-PF

4

I HEREBY CERTIFY THAT THIS IS A TRUE AND ACCURATE COPY OF THE ABOVE DOCUMENT TAKEN UNDER MY DIRECTION AND CONTROL ON THIS DATE IN SACRAMENTO, CALIFORNIA PURSUANT TO AUTHORIZATION BY THE DIRECTOR OF TRANSPORTATION.

DATE 8-26-74 SIGNATURE [signature] TITLE [illegible]

SCALE 1"=50' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=20' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=10' or 1"=100' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM
SCALE 1"=40' MICROFILM - STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS - MICROFILM

ATTACHMENT 3 :
FLOOD INSURANCE
STUDY

CHARLES AVENUE

LOURET AVENUE

INTERSTATE HIGHWAY 5

INTERSTATE HIGHWAY 5

14.9' 10 YEAR FLOOD ELEVATION @ OUTLET

15.1' 10-YEAR FLOOD ELEVATION @ INLET

200'

ELEVATION IN FEET (NAVD)

30

20

10

0

2600 2800 3000 3200 3400 3600 3800 4000 4200 4400 4600 4800 5000 5200

STREAM DISTANCE IN FEET ABOVE CROSS SECTION A

D E F G

LEGEND

- 0.2% ANNUAL CHANCE FLOOD
- 1% ANNUAL CHANCE FLOOD
- 2% ANNUAL CHANCE FLOOD
- 10% ANNUAL CHANCE FLOOD
- STREAM BED
- CROSS SECTION LOCATION

FLOOD PROFILES

OTAY RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN DIEGO COUNTY, CA
(AND INCORPORATED AREAS)

259P

APPENDIX 8

FEMA FLOOD 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.

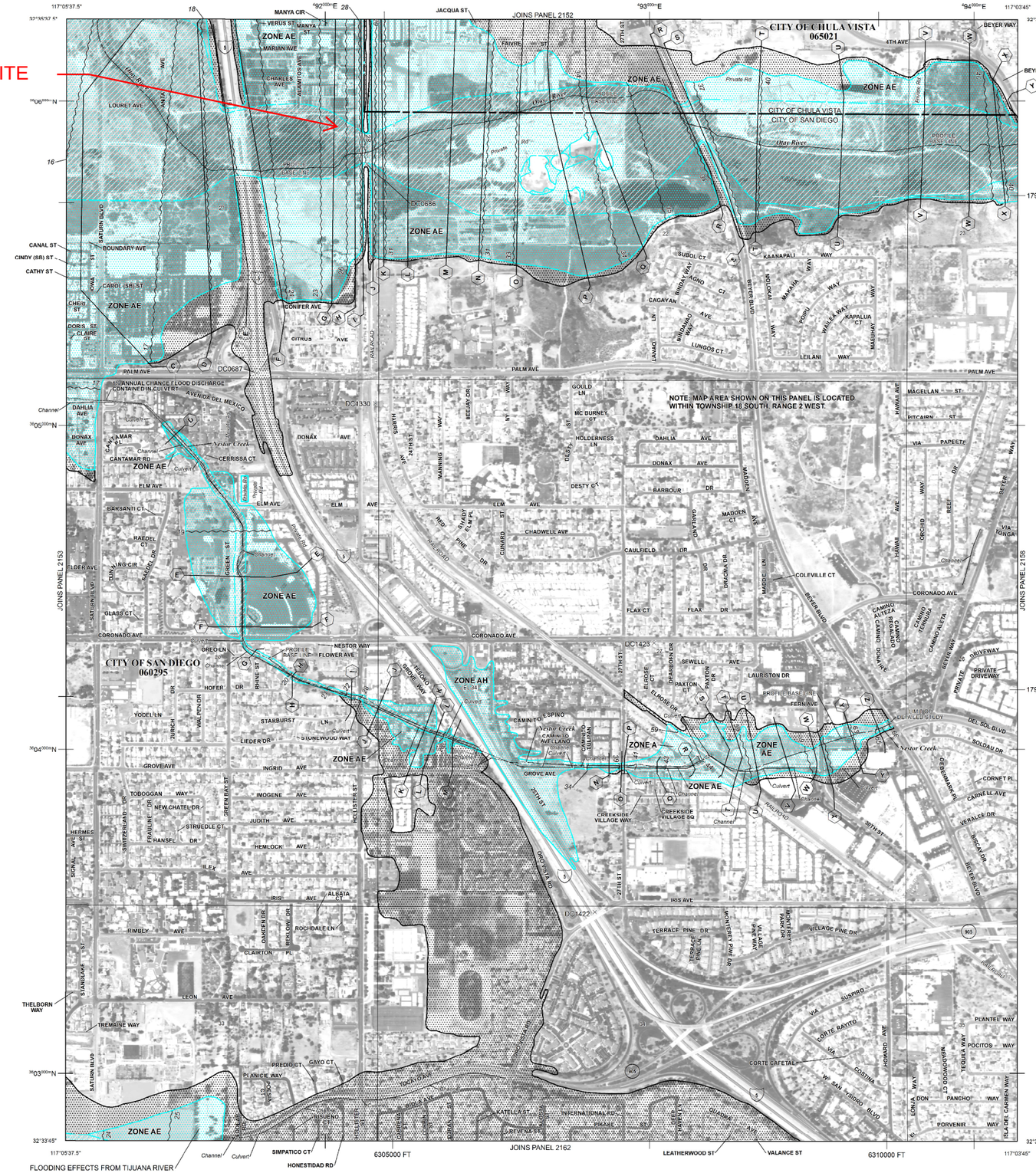
This map reflects more detailed and up-to-date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, 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.

For information and questions about this map, available products associated with this FIRM including historic versions of this FIRM, how to order products or the National Flood Insurance Program in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Map Service Center website at http://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website. Users may determine the current map date for each FIRM panel by visiting the FEMA Map Service Center website or by calling the FEMA Map Information eXchange.

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.



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 (EL 987) Base Flood Elevation line and value; elevation in feet*

* Referenced to the North American Vertical Datum of 1988

Cross section line

Transect line

97°07'30", 32°22'30" Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere

4275000mE 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.
April 5, 2016 - to remove Provisionally Accredited Levee note

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 1,000 FEET
150 0 150 200 METERS

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 2154H

FIRM

FLOOD INSURANCE RATE MAP

SAN DIEGO COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 2154 OF 2375

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
CHULA VISTA, CITY OF	065021	2154	H
SAN DIEGO, CITY OF	060295	2154	H

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 06073C2154H

MAP REVISED APRIL 5, 2016

Federal Emergency Management Agency

APPENDIX 9

FEMA FLOOD STUDIES

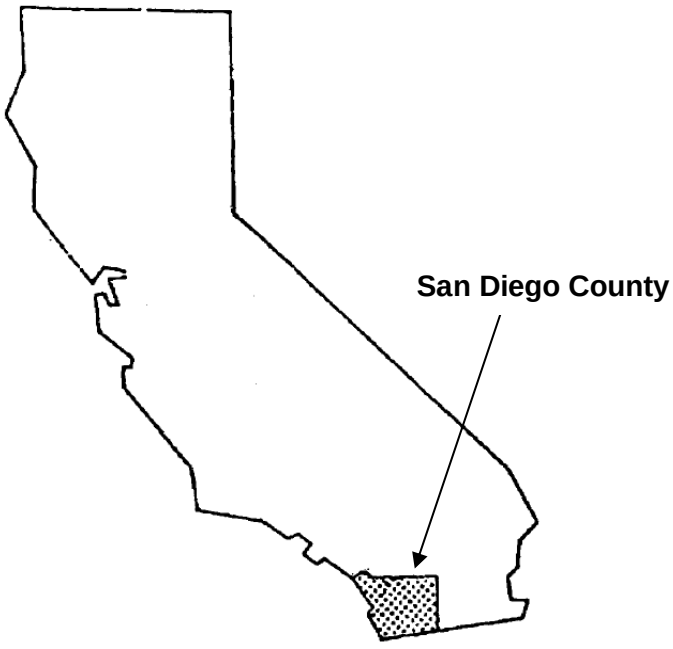
FLOOD INSURANCE STUDY



SAN DIEGO COUNTY, CALIFORNIA
AND INCORPORATED AREAS

VOLUME 1 OF 11

Community Name	Community Number
SAN DIEGO COUNTY, UNINCORPORATED AREAS	060284
CARLSBAD, CITY OF	060285
CHULA VISTA, CITY OF	065021
CORONADO, CITY OF	060287
DEL MAR, CITY OF	060288
EL CAJON, CITY OF	060289
ENCINITAS, CITY OF	060726
ESCONDIDO, CITY OF	060290
IMPERIAL BEACH, CITY OF	060291
LA MESA, CITY OF	060292
LEMON GROVE, CITY OF	060723
NATIONAL CITY, CITY OF	060293
OCEANSIDE, CITY OF	060294
POWAY, CITY OF	060702
SAN DIEGO, CITY OF	060295
SAN MARCOS, CITY OF	060296
SANTEE, CITY OF	060703
SOLANA BEACH, CITY OF	060725
VISTA, CITY OF	060297



REVISED
4/5/2016

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
06073CV001D

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TABLE 8: SUMMARY OF PEAK DISCHARGES

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cubic feet per second) 10% Annual-Chance	2% Annual-Chance	1% Annual-Chance	0.2% Annual-Chance
At 19th Street	--	--	--	864[4]	--
At Elm Avenue	2.45	--	--	796[4]	--
At Coronado Avenue	2.33	--	--	698[4]	--
At Hollister Street	1.99	--	--	496[4]	--
At 25th Street/Interstate 5	1.71	--	--	456[4]	--
At San Diego and Arizona Eastern Railroad	1.40	555	860	1,015	2,295
North Avenue Tributary					
Approximately 1,730 feet upstream of North Broadway	0.5	--	--	440	--
North Branch Poway Creek					
At Sycamore Canyon Road	4.5	650	2,000	3,000	7,200
North Tributary to Santa Maria					
At Mouth	1.6	100	600	1,100	2,900
Olive Creek					
At Mouth	1.0	--	--	1,370	--

-- Data Not Available

[4] Decrease Due to Construction of "Lot 6 Detention Basin" Upstream of Railroad

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Otay River								
A	0	2,533	4,688	4.7	13.2	13.2	13.2	0.0
B	1,390	2,110	9,474	2.3	15.2	15.2	16.1	0.9
C	2,490	2,300	4,084	5.4	16.3	16.3	16.8	0.5
D	3,720	1,662	7,917	2.8	18.2	18.2	18.9	0.7
E	4,040	642	1,928	11.4	19.3	19.3	19.4	0.1
F	4,270	722	3,819	5.8	20.6	20.6	20.6	0.0
G	5,100	641	2,883	7.6	24.0	24.0	24.0	0.0
H	5,350	360	1,767	12.4	25.7	25.7	25.7	0.0
I	5,390	320	2,711	8.1	28.0	28.0	28.0	0.0
J	5,500	304	2,359	9.3	28.9	28.9	28.9	0.0
K	5,600	440	4,010	5.5	30.8	30.8	30.8	0.0
L	5,880	740	4,511	4.9	30.8	30.8	30.9	0.1
M	6,280	1,020	7,451	2.9	30.9	30.9	31.5	0.6
N	6,610	1,225	7,933	2.8	30.9	30.9	31.7	0.8
O	7,012	1,243	4,824	4.6	32.8	32.8	32.9	0.1
P	7,330	1,035	3,833	5.7	33.3	33.3	33.8	0.5
Q	7,670	1,204	6,208	3.5	34.3	34.3	35.3	1.0
R	8,780	451	3,132	7.0	36.4	36.4	37.3	0.9
S	8,875	432	2,553	8.6	36.6	36.6	37.6	1.0
T	9,525	1,060	7,231	3.0	39.7	39.7	39.9	0.2
U	10,375	1,110	9,424	2.3	40.1	40.1	40.3	0.2
V	11,275	935	8,841	2.5	40.3	40.3	40.5	0.2
W	11,825	917	8,300	2.6	40.3	40.3	40.6	0.3
X	12,085	670	6,494	3.4	40.4	40.4	40.7	0.3
Y	12,395	403	1,798	12.2	42.9	42.9	42.9	0.0
Z	12,579	476	3,279	6.8	45.4	45.4	45.4	0.0

[1] Feet above Cross Section A

3.3 Vertical Datum

All FIS reports and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised FIS reports and FIRMs was the National Geodetic Vertical Datum of 1929 (NGVD). With the finalization of the North American Vertical Datum of 1988 (NAVD), many FIS reports and FIRMs are being prepared using NAVD as the referenced vertical datum.

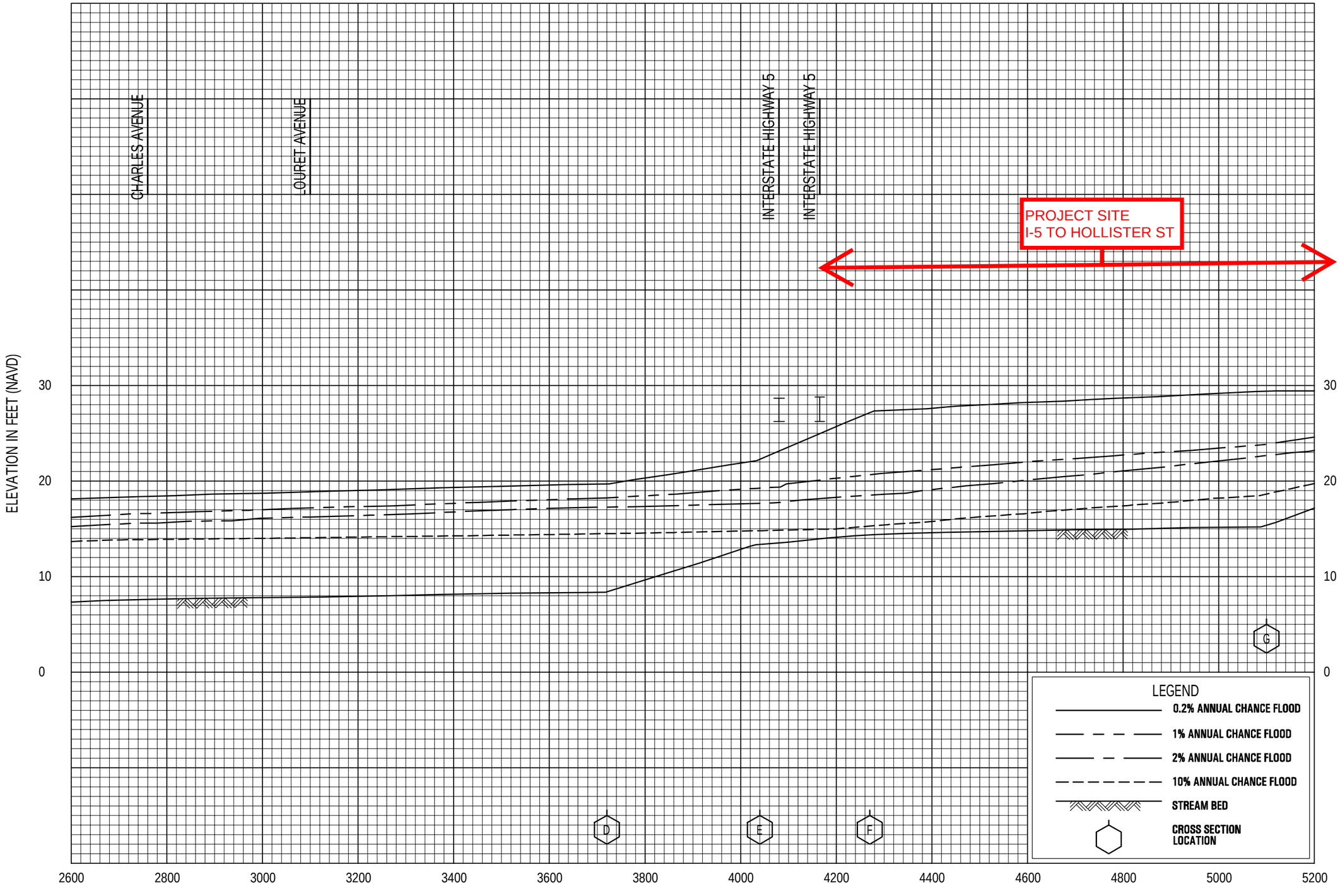
All flood elevations shown in this FIS report and on the FIRM are referenced to NAVD, with exception of two panels: 06073C2151F and 06073C2152F. These panels were not updated with this revision and are referenced to NGVD. Flooding sources on the non-updated FIRMs include Nestor Creek, Otay River, San Diego Bay, Telegraph Canyon Creek, and Tijuana River. The profile panels and floodway data tables that contain information corresponding with the non-updated panels have been included in NGVD, in addition to all of the data being presented in NAVD. Structure and ground elevations in the community must, therefore, be referenced to NAVD. It is important to note that adjacent communities may be referenced to NGVD. This may result in differences in Base (1-percent-annual-chance) Flood Elevations (BFEs) across the corporate limits between the communities. The conversion factor for each flooding source studied by detailed methods is shown below in Table 12 "Flooding Source Conversion Factor."

TABLE 12: FLOODING SOURCE DATUM SHIFT VALUES

Stream Name	Elevation (feet NAVD above NGVD)
Adobe Creek	+2.2
Agua Hedionda Creek	+2.2
Agua Hedionda Creek (At City of Carlsbad)	+2.2
Agua Hedionda Creek (At City of Vista)	+2.3
Alvarado Creek	+2.1
Beaver Hollow Creek	+2.2
Beeler Creek	+2.1
Broadway Creek	+2.1
Buena Creek	+2.3
Buena Vista Creek	+2.3
Buena Vista Creek Tributary 1	+2.3
Buena Vista Creek Tributary 3	+2.3
Calavera Creek	+2.2
Carmel Valley Creek	+2.1
Carroll Canyon Creek	+2.1
Coleman Creek	+2.5
County Ditch Creek	+2.1

TABLE 12: FLOODING SOURCE DATUM SHIFT VALUES

Stream Name	Elevation (feet NAVD above NGVD)
Moosa Creek (North Branch)	+2.3
Moosa Creek (South Branch)	+2.3
Murphy Canyon Creek	+2.1
Murray Canyon Creek	+2.1
Nestor Creek	+2.1
North Avenue Tributary	+2.3
North Branch Poway Creek	+2.1
North Tributary to Santa Maria Creek	+2.2
Olive Creek	+2.4
Otay River	+2.2
Pala Mesa Creek	+2.2
Paradise Creek	+2.1
Paradise Creek – Valley Road Branch	+2.1
Pilgrim Creek	+2.3
Poggi Canyon Creek	+2.2
Pomerado Creek	+2.1
Poway Creek	+2.1
Rainbow Creek (Main Branch)	+2.3
Rainbow Creek (West Branch)	+2.3
Rattlesnake Creek	+2.1
Rattlesnake Creek Split Flow at Heritage Hills	+2.1
Rattlesnake Creek Split Flow at Midland Road	+2.1
Reidy Creek	+2.3
Reidy Creek Split Flow	+2.3
Rice Canyon Creek	+2.1
Rincon Avenue Tributary	+2.3
Rose Canyon Creek	+2.1
Samagutuma Creek	+2.4
San Clemente Canyon Creek	+2.1
San Diego Bay	+2.2
San Diego River	+2.1
San Dieguito River	+2.1
San Elijo Creek	+2.2
San Luis Rey River	+2.3
San Marcos Creek	+2.3
San Marcos Creek (Below Lake San Marcos)	+2.3
San Marcos Creek Highway 78 Split Flow	+2.3



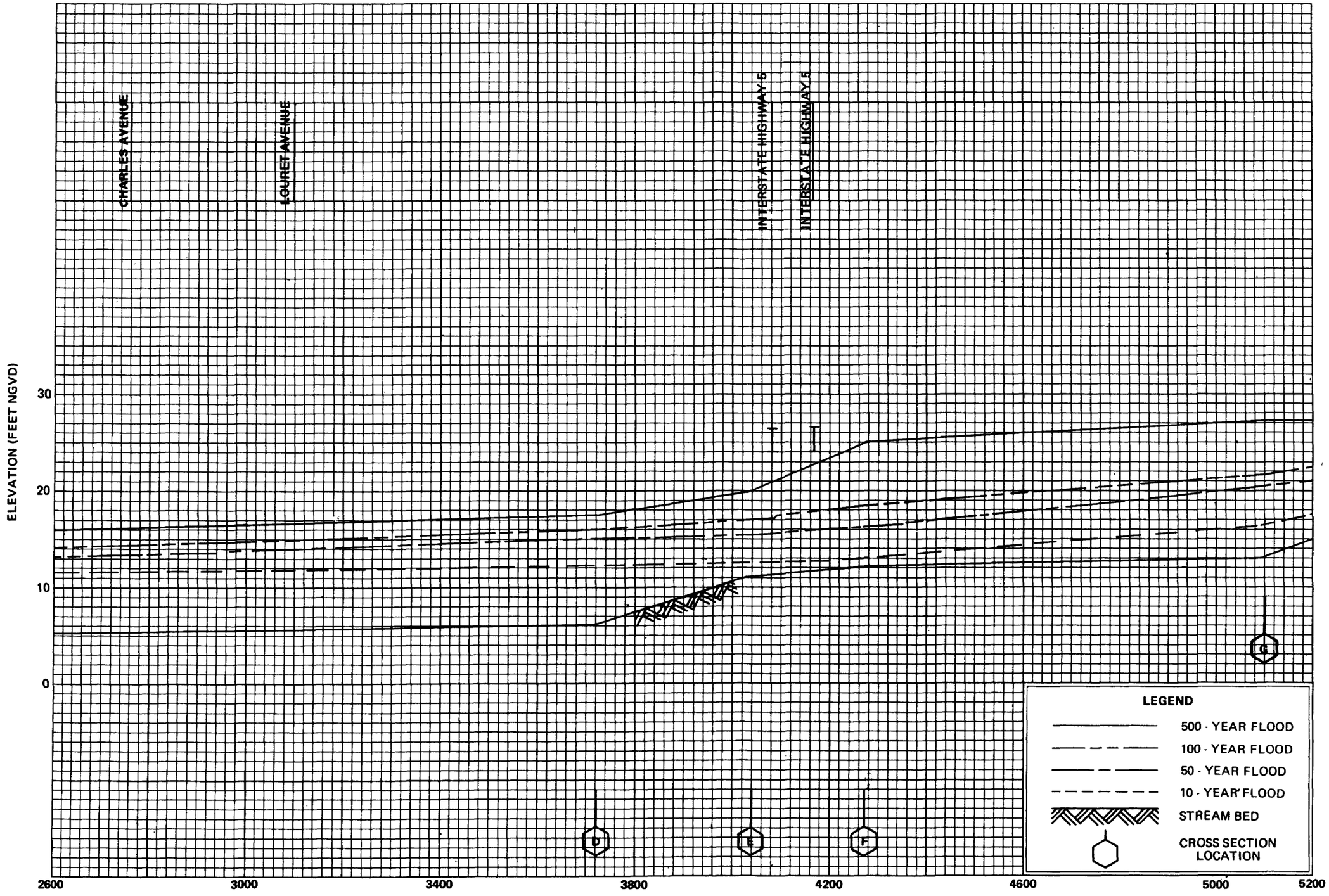
FLOOD PROFILES

OTAY RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN DIEGO COUNTY, CA
(AND INCORPORATED AREAS)

259AP



FLOOD PROFILES

OTAY RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN DIEGO COUNTY, CA AND INCORPORATED AREAS

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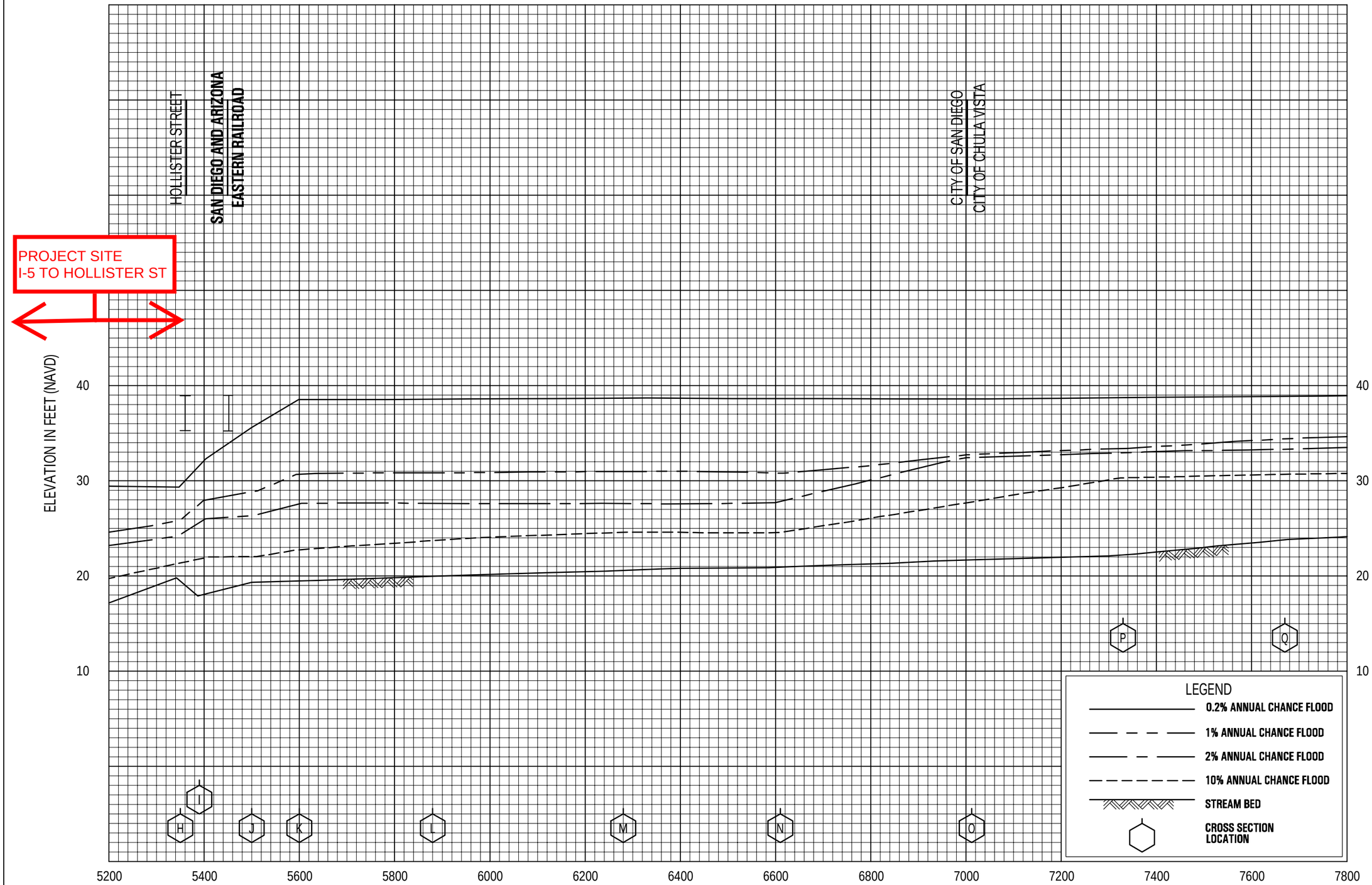

FLOOD PROFILES

OTAY RIVER

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SAN DIEGO COUNTY, CA
(AND INCORPORATED AREAS)

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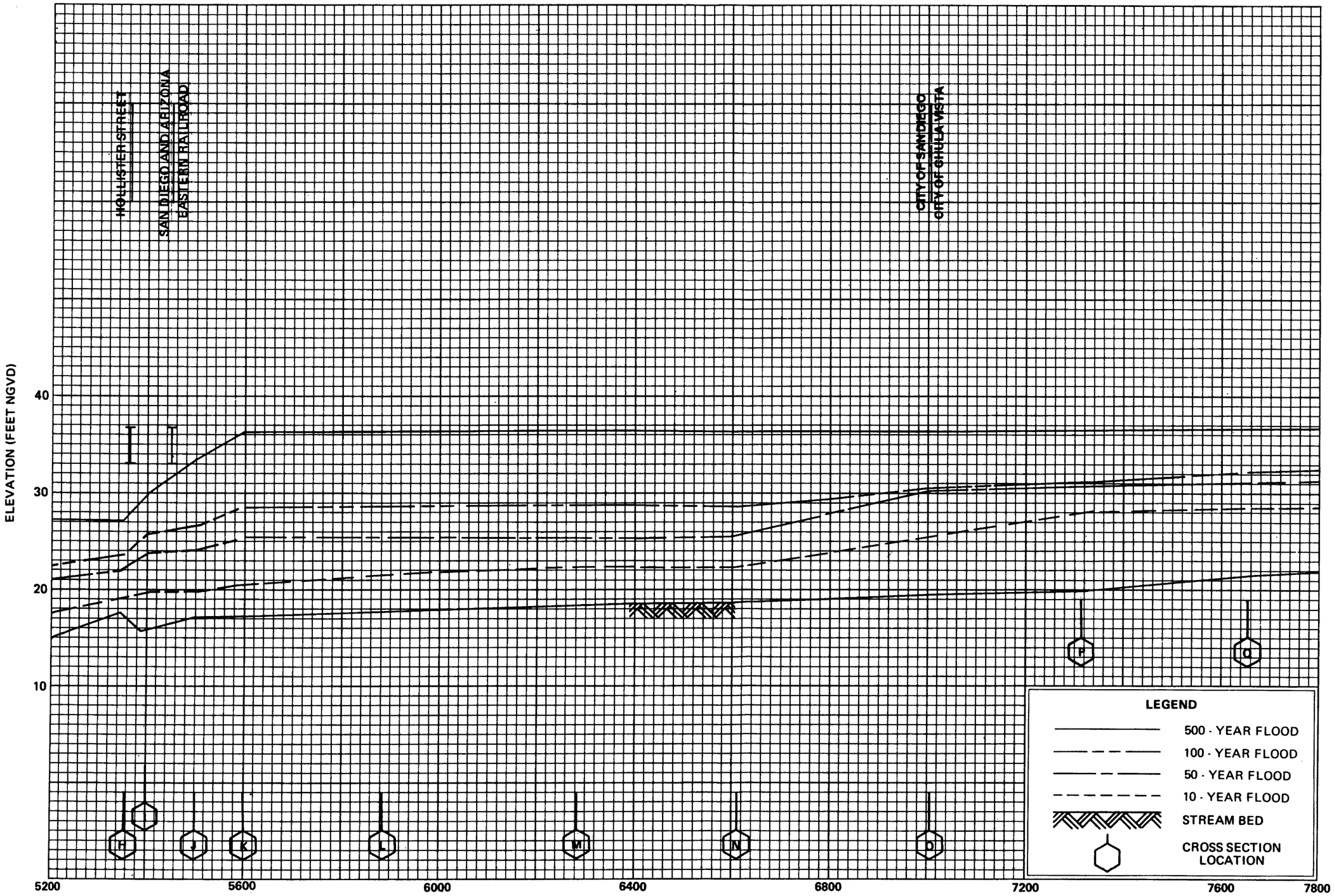

FLOOD PROFILES

OTAY RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

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0250976

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Case Number/ Study ID:060284-19780223

Description:1d. Type 15 Study

Revision Status:

Flooding Source(s):Otay River

Effective Date:10/20/1981

Contents:17. Misc. ref.: Other reference materials 24. Final FIS/FIRM/FHBM

Notes:060284-19780223_FIS

Scanned by:

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QC Staff:

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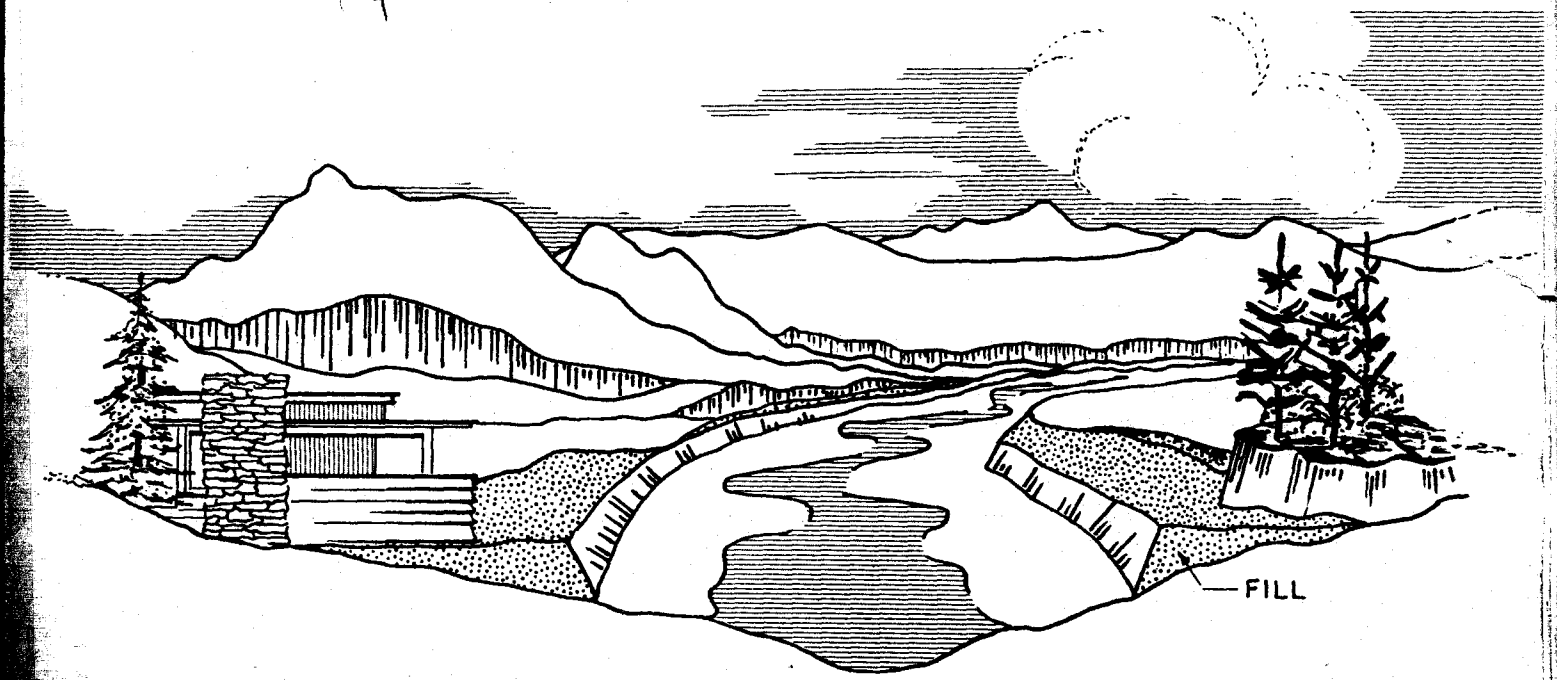
CA081106

FLOODWAY INFORMATION STUDY

OTAY RIVER

SAN DIEGO COUNTY, CALIFORNIA

CA
74



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DECEMBER 1974

FLOOD INSURANCE STUDY

OTAY RIVER

SAN DIEGO COUNTY, CALIFORNIA

PREPARED FOR THE

DEPARTMENT OF WATER RESOURCES

STATE OF CALIFORNIA

BY THE

DEPARTMENT OF SANITATION AND FLOOD CONTROL

SAN DIEGO COUNTY

COMPLETED Joseph C Hill DATE 23 Feb 78

CA 081106

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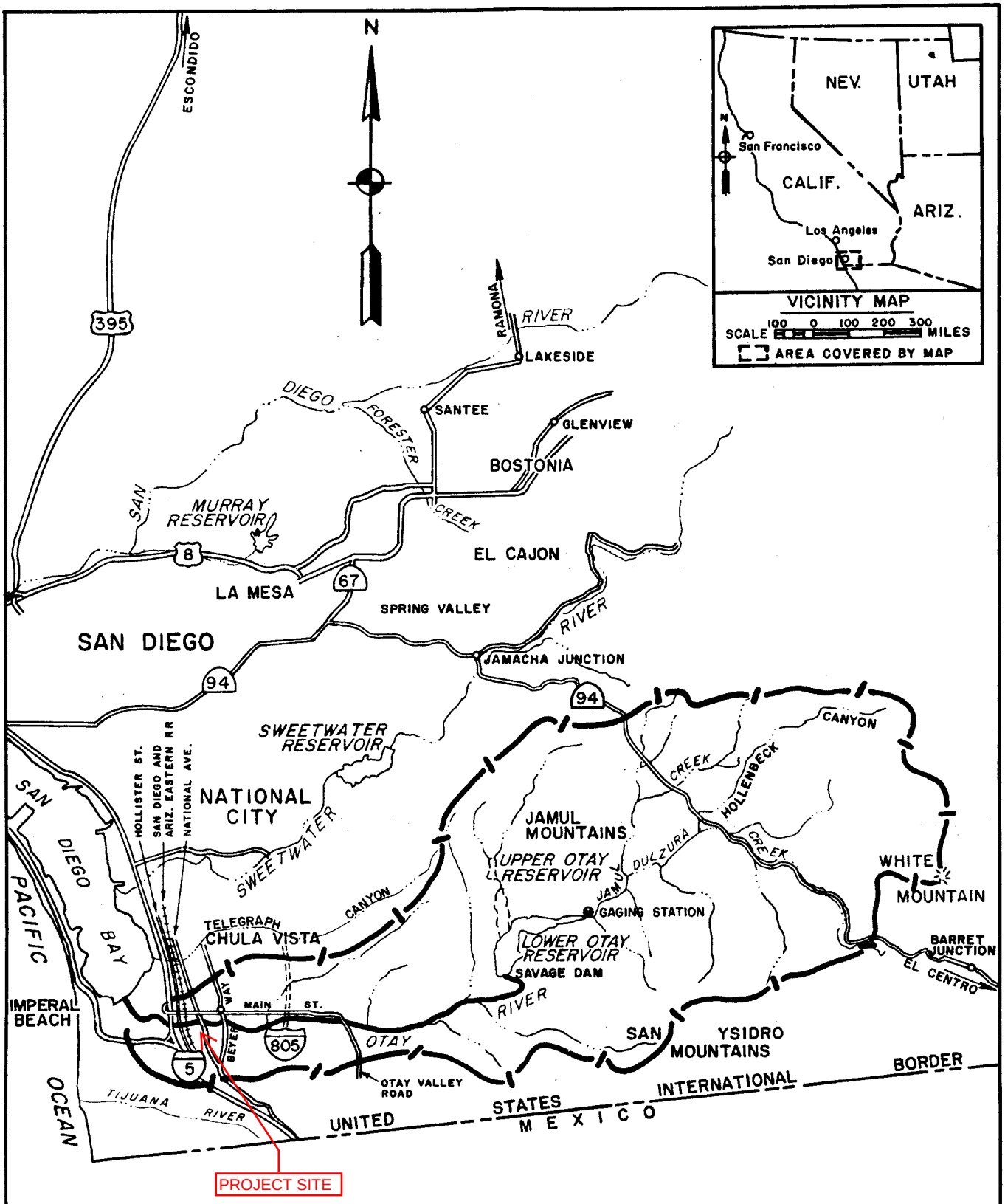


DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

FLOODWAY INFORMATION
OTAY RIVER
SAN DIEGO BAY TO SAVAGE DAM
SAN DIEGO COUNTY, CALIFORNIA

GENERAL MAP

DECEMBER 1974

LEGEND

REACH COVERED BY THIS REPORT.

BOUNDARY OF PROJECT DRAINAGE AREA.

PLATE 1

TABLE 1

Bridges and Culverts Within Study Area

Identification	Location (a)	Elevation (b) Streambed	Low Chord (c)	Roadway (d)	10 Yr (e)	IRF (e)
Interstate 5 Bridge	2.59	11.7	23.5	26.5	13.7	21.8
Hollister Street (4 - 2' CMP)	2.82	15.5	17.5	19.0	20.1	22.3
San Diego and Arizona Eastern Railroad	2.84	16.2	33.0	36.5	20.8	24.0
National Ave. Bridge	3.50	24.7	38.0	40.5	31.1	34.2
Beyer Way Bridge	4.19	32.6	52.5	58.1	37.4	40.0
Interstate 805 Bridge	5.80	74.2	116.0	127.0	79.2	84.6
Otay Valley Rd (2 - 2' CMP)	7.89	132.2	135.2	136.2	138.7	142.4

(a) Miles upstream from mouth
(b) All elevations are in feet, mean sea level datum
(c) Elevation of bottom of bridge structure or top of culvert
(d) Average elevation
(e) Computed water surface elevation based on estimated flow and existing channel and structures.

Although specific flood forecasts are not made for the Otay River drainage basin, daily weather forecasts applicable to the Otay area are issued by the National Weather Service office in San Diego. When weather conditions warrant, storm and probable flood warnings from the San Diego County area are issued by the National Weather Service River District office in San Diego. Local news media and law enforcement agencies disseminate these warnings to the public.

Flood fighting and emergency evacuation plans – There are no specific flood fighting or emergency evacuation plans for the Otay River area. If the need arises, State and local law enforcement agencies, local fire departments and civil defense groups, and street and highway maintenance crews could assist in the rescue of stranded persons and perform other flood fighting activities. The California Department of Water Resources, through its Flood Operation Center, coordinates flood fighting activities throughout the State and is authorized to receive requests from local public agencies for assistance during floods. During emergencies, the San Diego County Civil Defense and Disaster Office coordinates activities of local law enforcement agencies, and of fire, health, and sanitation departments.

The Corps of Engineers responds to requests from the State Disaster Office for assistance in flood fighting and rescue work when flood emergencies are beyond the capabilities of State and local governmental agencies.

FUTURE FLOODS

Although floods of the same magnitude as those that have occurred in the past could recur in the future, discussion of the future floods in this report is limited primarily to those that have been designated as the intermediate regional and 10-year frequency floods. The 10-year frequency flood could occur on the average of once every 10 years, and has a 10 percent chance of being equalled or exceeded in any year. Most storm drains and culverts in San Diego County are designed to pass the 10-year flood without damage to these structure or to adjacent property.

The intermediate regional flood is one that could occur about once every 100 years on the average, and has a 1 percent chance of being equalled or exceeded in any year. Since there are no streamflow records for the study reach, it was necessary to analyze precipitation and streamflow records of other stream basins having hydrologic, meteorologic, and physiographic characteristics similar to those of the Otay River basin. Studies were made to transpose the information, thus derived, to the study basin and to compute peak flows for the 10-year-frequency and intermediate regional floods. Peak flows developed for the 10-year-flood and intermediate regional flood at selected points in the study area are shown in table 2.

TABLE 2

Peak Flows for 10-Year-Frequency and
Intermediate Regional Floods

Location on Otay River	Total Drainage Area (sq. miles)	10-Year Flood* (cfs)	IRF (cfs)
At I-5 Bridge	139.2	3,700	22,000
Beyer Way Bridge	135.9	3,280	22,000
Otay Valley Road Crossing	122.7	2,850	22,000
Downstream of Savage Dam	98.6	2,000	22,000

*Flow values determined by San Diego County.

The future floods discussed herein are of the general winter local storm type. During floods, debris collecting on bridges and culverts could decrease their carrying capacity and cause greater water depths (backwater effect) upstream from these structures. The occurrence and amount of debris are indeterminate factors; however, a limited amount of debris was considered in preparing the profiles of the intermediate regional and the 10-year-frequency floods. Similarly, the maps of flooded areas show the backwater effect of obstructive bridges and culverts, and reflect increased water surface elevations that could be caused by limited amounts of debris collecting against the structures.

Hazards of Large Floods

The amount and extent of damage caused by any flood depends on the topography of the area flooded, depth, and duration of flooding, velocity of flow, and developments on the flood plain. An intermediate regional flood on the Otay River would result in inundation of most of the riverine lands.

Deep floodwater flowing at a high velocity and carrying floating debris would create conditions hazardous to persons and vehicles attempting to cross flooded areas. In general, floodwater 3 or more feet deep and flowing at a velocity of 3 or more feet per second could sweep a person off his feet, thus creating definite dangers of injury or drowing. Rapidly rising and swiftly flowing floodwater may trap persons in homes that are ultimately destroyed or in vehicles that are ultimately submerged. Decaying flood-deposited garbage or other organic materials could create health hazards. Isolation of areas by floodwater could create hazards in terms of medical, fire, or law enforcement emergencies.

Flooded Areas and Flood Damages

The areas along the Otay River that would be flooded by the intermediate regional flood are shown on plates 2 and 3. Also, larger scaled maps (1" = 200') which are on file with the San Diego County Department of Sanitation and Flood Control show the area that would be flooded by the intermediate regional flood and a designated floodway. Coordinates of the south and west margin of these maps are referenced on plates 2 and 3 and conform to the California Rectangular Grid (Zone VI).

Due to the wider flood plain, greater depth of flooding, higher velocity flow, and longer duration of flooding during an intermediate regional flood, damage would be more severe than during the 10-year-frequency flood. Streets, bridges, culverts, and public utilities would be severely damaged by high velocity floodflows. Extensive deposits of silt and debris would occur in many parts of the flooded areas. Plates 4 through 10 show water surface profiles of the intermediate regional and 10-year-frequency floods. Depth of flow in the channel can be estimated from these profiles.

Inundation of structures by the intermediate regional flood would be mostly limited to areas downstream of National Avenue. Flooding of cultivated fields and some industrial and residential properties just upstream of the San Diego and Arizona Eastern Railroad would occur. Flooding of a trailer park, some residences, cultivated fields, and a sewage treatment plant would also occur downstream of the I-5 bridge. Erosion of the dikes forming the salt evaporators would occur in areas where they are overtopped by floodwaters. All of the sand and gravel operations located in the Otay River channel and flood plain would be flooded. Future development of the flood plain which does not take into account the flood hazard would result in greater damages during future floods.

Obstructions – Several bridges and roads cross the Otay flood plain. As can be seen in table 1 and on plates 4 through 9, none of the bridges will be overtopped by the intermediate regional flood. However, stability analysis of these bridges under scour action is beyond the scope of this study, and it is possible that some of these bridges would collapse during a major flood. The two roads which cross the flood plain, namely Hollister Street and Otay Valley Road would be washed out during a major flood. Numerous depressions, dirt road crossings, and mounds of earth in the channel would impede floodflows during the earlier period of a flood, but their effects during the peak stages of major floods would probably be negligable.

Velocities of flow – The slope of the streambed, the shape of the cross sections, and the roughness of the areas in the channel and overbank are major factors that govern the velocity of floodflow. During a flood, velocities of flow would change with time due to bank erosion, sediment transport, and deposition of debris as the discharge increases and decreases. The average velocities of flow for the entire reach studied during peak flows of the intermediate regional flood in the channel and overbank are 9 and 5 feet per second respectively. However, the velocity at any given point along the study reach can vary from 2 to 21 feet per second in the channel and from 1 to 11 feet per second in the overbank.

Photographs, future flood heights – The levels that the intermediate regional flood is expected to reach at various locations on the Otay River flood plain are indicated in figures 8 through 10.

NOTE:

RED NUMBERS INDICATED IN GRID RECTANGLES CORRESPOND WITH THE SAN DIEGO COUNTY MAP NUMBERING SYSTEM. THE COUNTY MAPS ARE ON FILE WITH THE SAN DIEGO COUNTY DEPARTMENT OF SANITATION AND FLOOD CONTROL, 5555 OVERLAND AVE., SAN DIEGO, CALIFORNIA, 92123. THE COUNTY MAPS DELINIATE THE INTERMEDIATE REGIONAL FLOOD PLAIN AND ITS PRELIMINARY FLOODWAY AT A SCALE OF 1" = 200'

Harbor Side

DOWNSTREAM LIMIT OF STUDY

South San Diego

154-1737

154-1743

154-1749

154-1755

154-1761

150-1737

150-1743

150-1749

I-805 BRIDGE UNDER CONSTRUCTION

PROJECT SITE
= 2.8 MILES FROM MOUTH

LEGEND

RIVER MILES

STREAMS

AREA FLOODED BY INTERMEDIATE REGIONAL FLOOD

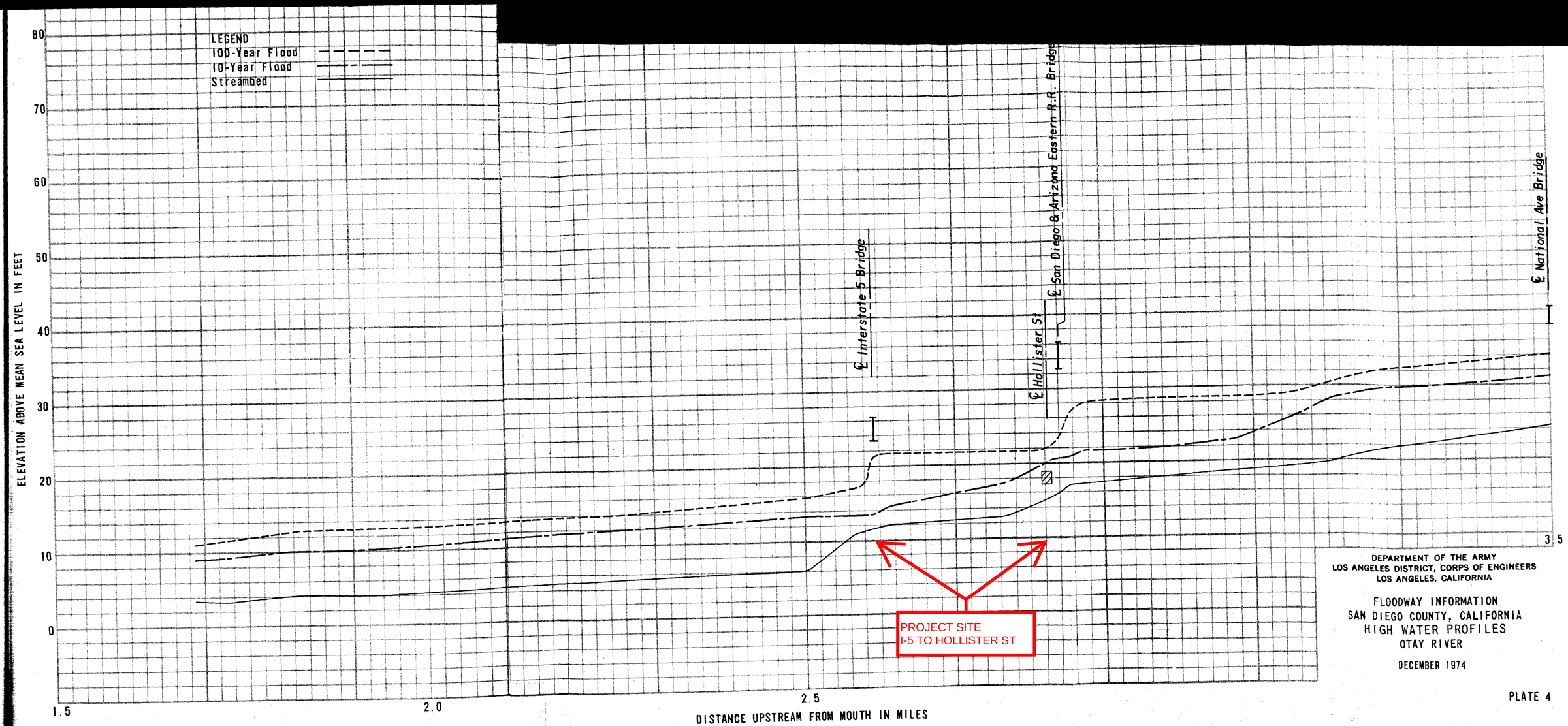
150-1737 SAN DIEGO COUNTY TOPOGRAPHIC MAP INDEX NUMBER

SCALE 1000 0 1000 2000 3000 FEET

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

FLOODWAY INFORMATION
OTAY RIVER
SAN DIEGO COUNTY, CALIFORNIA
INDEX MAP - FLOODED AREA
DECEMBER 1974

PLATE 2



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LOS ANGELES, CALIFORNIA

FLOODWAY INFORMATION
SAN DIEGO COUNTY, CALIFORNIA
HIGH WATER PROFILES
OTAY RIVER

DECEMBER 1974

PLATE 4

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.

GEOTECHNICAL INVESTIGATION

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

RED TAIL ACQUISITIONS, LLC
IRVINE, CALIFORNIA

APRIL 24, 2019
PROJECT NO. G2129-52-03

Project No. G2129-52-03
April 24, 2019

Red Tail Acquisitions, LLC
2082 Michelson Drive, 4th Floor
Irvine, California 92612

Attention: Mr. Tim Kihm

Subject: GEOTECHNICAL INVESTIGATION
BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

Dear Mr. Kihm:

In accordance with your request and authorization of our Proposal No. LG-16466 dated April 10, 2017, we herein submit the results of our geotechnical investigation for the subject project. We performed our investigation to evaluate the underlying soil/geologic conditions and potential geologic hazards. This report can assist in the design of the proposed buildings and associated improvements.

The accompanying report presents the results of our study and conclusions and recommendations pertaining to geotechnical aspects of the proposed project. The site is suitable for the proposed buildings and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project.

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

Very truly yours,

GEOCON INCORPORATED

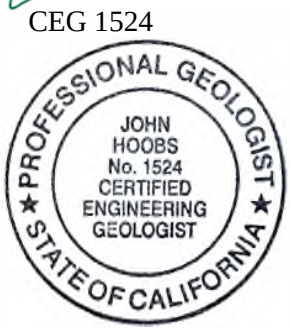
Lilian E. Rodriguez
RCE 83227



Shawn Foy Weedon
GE 2714



John Hoobs
CEG 1524



LER:SFW:JH:kcd

(email) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed residential development in the Otay Mesa-Nestor Community Planning Area in San Diego, California (see Vicinity Map, Figure 1). The purpose of the geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect development of the property including faulting, liquefaction and seismic shaking based on the 2016 CBC seismic design criteria. In addition, we provided recommendations for remedial grading, shallow foundations, concrete slab-on-grade, concrete flatwork, pavement, and retaining walls.

We reviewed the following plans and reports in preparation of this report:

1. *Report of Preliminary Geotechnical Investigation, Proposed Bella Mar RV Park, 408 Hollister Street, San Diego, California,* prepared by Christian Wheeler Engineering, dated December 22, 2005 (Project No. 2050832.01).

2. *Preliminary Geotechnical Investigation, 408 Hollister Street, San Diego, California,* prepared by Geocon Incorporated, dated February 25, 2019.

3. *Tentative Parcel Map, Bella Mar, Conceptual Grading and Utilities, San Diego, California,* prepared by Geocon Incorporated, dated May 7, 2013 (Project No. G1580-11-01).

The scope of this investigation included reviewing readily available published and unpublished geologic literature (see List of References); performing engineering analyses; and preparing this report. We also advanced 2 exploratory borings to a maximum depth of about 51 feet, performed 6 percolation/infiltration tests, sampled soil and performed laboratory testing. Appendix A presents the exploratory boring logs and details of the field investigation for the current study. The details of the laboratory tests and a summary of the test results are shown in Appendix B and on the boring logs in Appendix A. Appendix C presents the boring and trench logs and the laboratory test results from the previous geotechnical investigation (Christian Wheeler Engineering, 2005). Appendix D presents the results of our liquefaction analysis, and Appendix E presents a summary of our storm water management investigation.

2. SITE AND PROJECT DESCRIPTION

The subject property is located east of Interstate 5, west of Hollister Street and about 725 feet north of Conifer Avenue in the Otay Mesa – Nestor Community Planning Area in San Diego, California. The site slopes gently to the northwest with existing elevations ranging from approximately 16 feet to 22 feet above Mean Sea Level (MSL). The property is covered with seasonal shrubs and grasses, and appears to have been previously used for agriculture purposes. Several structures were located on the northeast corner of the property that appear to have been demolished between 2006 and 2009.

Remnants of former asphalt concrete pavement remain on the northeast portion of the site. An approximately 10 foot high slope descends along the northern property line with inclinations of about 2:1 (horizontal to vertical) to the Otay River.

We understand the planned development will consist of the construction of approximately 16 residential buildings on two parcels with associated driveways, retaining walls, utilities and landscaping. We understand maximum cuts and fills of approximately 5 and 8 feet, respectively, will be required to achieve planned on-site and off-site grades.

The locations, site descriptions and proposed development herein are based on our site reconnaissance, review of published geologic literature, field investigations, and discussions with project personnel. If development plans differ from those described herein, Geocon Incorporated should be contacted for review of the plans and possible revisions to this report.

3. GEOLOGIC SETTING

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

The site is located on the western portion of the coastal plain at the western portion of the Otay River drainage and the southern end of the San Diego Bay. Localized shallow undocumented fill and topsoil overlie alluvial soils deposits likely placed during the formation of the San Diego Bay and deposition from the Otay River. The site has Late Pleistocene-age Old Paralic Deposits (Unit 6 Nestor Terrace) and the Pliocene-age San Diego Formation underlying the alluvial soils at depths of approximately 35 feet.

4. SOIL AND GEOLOGIC CONDITIONS

Our field investigation and previous field investigations performed by Christian Wheeler Engineering (CWE) indicates the site is underlain by three surficial soil consisting of undocumented fill (Qudf), topsoil (unmapped) and alluvium (Qal). The Old Paralic Deposits (Qop) and the San Diego Formation (Tsd) exist below the alluvium. The occurrence, distribution, and description of the units encountered is shown on the Geologic Map, Figure 2 and on the boring logs in Appendices A and C. The Geologic Cross-Section, Figure 3, shows the approximate subsurface relationship between the surficial and geologic units. The surficial soils and geologic unit are described herein in order of increasing age.

4.1 Undocumented Fill (Qudf)

Undocumented fill associated with the previous land use was encountered in Boring B-1 by CWE (2005) to a depth of approximately 3 feet. The fill is reported to consist of gray, loose, silty sand with gravel. We expect fill is present on the northeast portion of the property from previously existing structures. Undocumented fill is also present on the western property boundary associated with the construction and support of existing Interstate 5 freeway. These materials are unsuitable in their present condition and will require removal and recompaction in the areas proposed to be graded and/or where settlement sensitive improvements are planned. The undocumented fill can be reused for the proposed compacted fill during grading operations provided it is free of roots and debris.

4.2 Topsoil (unmapped)

Holocene-age topsoil is present as a relatively thin veneer locally overlying alluvium materials across the site. The topsoil ranges in thickness from about 1 to 2 feet and can be characterized as soft and very loose to loose, dark brown to grayish brown, silty clay to silty sand. Removal of the topsoil will be necessary in areas to support proposed fill or structures. Due to the relatively thin thickness and discontinuity of these deposits, topsoil is not shown on the Geologic Map.

4.3 Alluvium (Qal)

The Quaternary-age alluvial deposits exist below the fill and topsoil across the site. We encountered alluvium in the current borings to a depth of approximately 34 feet below existing grade. The alluvium typically possesses a “very low” to “low” expansion potential (expansion index of 50 or less). The alluvium within the upper approximately 10 to 20 feet below grade encountered in the current and previous exploratory excavations at the site generally consists of loose to medium dense, moist to saturated, dark brown to reddish brown, silty to clayey sand. This material is compressible and the upper portions will require remedial grading.

The alluvium materials below a depth of approximately 10 to 20 feet to a depth of approximately 35 feet generally consists of very dense, saturated, dark brown sands with some gravels and cobbles.

The underlying dense to very dense alluvium is considered suitable for support of additional fill and/or structural load. We expect up to approximately 10 feet of compressible alluvium will be left in place due to the presence of the groundwater table. The alluvium is suitable for use as compacted fill soil; however, special soil handling will likely be required due to the saturated soil conditions.

4.4 Old Paralic Deposits (Qop)

We encountered the late Pleistocene-age Old Paralic Deposits underlying the alluvial deposits to depths of approximately 49 and 44 feet below existing grade in Borings B-1 and B-2, respectively. The Old Paralic Deposits encountered consist of interbedded layers of dense to very dense, silty sand to very stiff to hard silt. This geologic unit generally has a "low" to "medium" expansion potential (expansion index of 21 to 90) and adequate shear strengths. The Old Paralic Deposits are considered suitable for support of additional fill and/or structural loads.

4.5 San Diego Formation (Tsd)

The San Diego Formation is present underlying the Old Paralic Deposits at an elevation of about -30 to -25 feet MSL. The San Diego Formation encountered generally consists of weakly to well-cemented, micaceous, saturated, light brown, fine- to medium-grained sandstone. The San Diego Formation typically possesses a "very low" to "low" expansion potential (expansion index of 50 or less). The San Diego Formation is considered suitable for support of additional fill and/or structural loads.

5. GROUNDWATER

We encountered groundwater at depths of 13.5 and 16 feet in current Borings B-1 and B-2 (elevations of about 6.5 and 4 feet above Mean Sea Level). In addition, CWE reported that groundwater was encountered at depths ranging from 8 to 13 feet (elevations of about 7 to 12 feet above Mean Sea Level). We also encountered saturated soils at depths of approximately 5 feet in current Borings B-1 and B-2 (elevation of approximately 15 feet above MSL). The project should be designed with a groundwater elevation of 10 feet MSL, and saturated conditions should be expected at an elevations of approximately 15 feet MSL. Due to the relatively close vicinity of the San Diego Bay, the groundwater should be considered brackish. We expect groundwater and saturated soil will be a factor during site development, especially for liquefaction mitigation/ground improvements (if required), grading operations and utility installation. The use of dewatering techniques should be considered to facilitate site grading and improvements, where necessary. It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Groundwater and seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

According to the *Water Quality Control Plan for the San Diego Basin* prepared by the California Regional Water Quality Control Board, the site is located within Hydrologic Unit Basin Number 10.20 (the Otay Valley hydrologic area). The subject site is classified as having beneficial use for municipal, agricultural and industrial uses according to Table 2-5, Beneficial Uses of Ground Waters.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 6 defines the site with *Hazard Category 31: High Liquefaction Potential – shallow groundwater, major drainages, hydraulic fills.* Based on a review of the map, a fault does not traverse or project toward the planned development area.

6.2 Faulting and Seismicity

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

According to the computer program *EZ-FRISK* (Version 7.65), 6 known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. The Rose Canyon Fault zone and the Newport-Inglewood Fault are the closest known active faults, located approximately 4 miles northwest of the site. Earthquakes that might occur on the Newport-Inglewood or Rose Canyon Fault Zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.32g, respectively. Table 6.2.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008 acceleration-attenuation relationships.

TABLE 6.2.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2007 (g)
Newport-Inglewood	4	7.50	0.32	0.22	0.27
Rose Canyon	4	6.90	0.30	0.22	0.26
Palos Verdes	12	7.70	0.27	0.16	0.22
Coronado Bank	12	7.40	0.26	0.16	0.20
Elsinore	45	7.85	0.20	0.09	0.12
Earthquake Valley	50	6.80	0.13	0.06	0.06

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008 in the analysis. Table 6.2.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.2.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2007 (g)
2% in a 50 Year Period	0.59	0.32	0.37
5% in a 50 Year Period	0.45	0.25	0.29
10% in a 50 Year Period	0.35	0.20	0.23

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the 2016 California Building Code (CBC) guidelines currently adopted by the City of San Diego.

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

6.3 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the earth surface. The potential for ground rupture is considered to be negligible due to the absence of active faults at the subject site.

6.4 Liquefaction Potential and Seismically Induced Settlement

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface, and soil densities are less than about 70 percent of the maximum dry densities. If the four previous criteria are met, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations.

The County of San Diego Hazard Mitigation Plan (2017) maps the site in an area with a high liquefaction risk. In addition, the *City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Sheet 6* defines the site with a geologic hazard Category 31: *Liquefaction: High Potential – Shallow Groundwater, major drainages, hydraulic fills*. The current standard of practice, as outlined in the *Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California* requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

We performed liquefaction analysis of our CPT soundings using the program CLiq (Version 1.7). This program utilizes the 2001 NCEER method of analysis. In addition, we performed liquefaction analyses using the data from current Borings B-1 and B-2. We used methods following the methodology of NCEER (2001 and 2008) to perform a liquefaction evaluation using the data from the borings. We used a static groundwater depth of 10 feet, a modal magnitude of 6.12 earthquake (attributed to the

Newport-Inglewood Fault), and a peak horizontal site acceleration, PGA_M, of 0.39g calculated from ASCE 7-10 Section 11.8.3. This semi-empirical method is based on correlations with the data collected from the borings and CPT soundings and field performance data.

The liquefaction analyses (included in Appendix D) indicate the soils to depths from generally 10 and 15 feet below the existing grade could be prone to between 0 and 0.9 inches using CPT data. We estimate the differential settlement of ⅔ the total settlement ranging from 0 to ⅔ inches. The estimated liquefaction settlement potential using the data from Borings B-1 and B-2 ranges from approximately 1 and 2 inches (average of 1.5 inches), with differential settlement ranging from approximately ⅔ and 1⅓ inches (average of 1 inch). We used a groundwater depth of approximately 10 feet for our liquefaction analyses based on the groundwater elevations encountered in the previous borings and trenches by CWE and the saturated conditions of the alluvial soils encountered above the groundwater table in the current borings. Our analyses indicate that liquefaction could at depths between 10 to 15 feet in the areas of CPT-1 through CPT-9 and Borings B-1 and 2 for the levels of ground shaking assumed. We do not expect liquefaction to occur within the underlying dense, gravelly alluvium, Old Paralic Deposits and San Diego Formation. Recommendations presented in this report are intended to reduce the effects of seismically-induced settlement on the proposed structures.

Sand boils occur where liquefiable soil is extruded upward through the soil deposit to the ground surface. Providing an increase in overburden pressure and a compacted fill mat can mitigate surface manifestation. Research presented by Ishihara (1985) indicates that the presence of a non-liquefiable surface layer typically results in the effects of at-depth liquefaction from reaching the surface. Modifications to Ishihara's chart have been made to include higher ground accelerations (Ishihara's 1985 chart was based on a 0.25g ground acceleration) by Youd and Garris (1995). Based on Youd's modified curves and the thickness of the non-liquefiable soil layer (layer above the assumed groundwater table), the potential for surface manifestation is possible unless ground improvements are performed.

Lateral spreading occurs when liquefiable soil is in the immediate vicinity of a free face such as a slope. Factors controlling lateral displacement include earthquake magnitude, distance from the earthquake epicenter, thickness of liquefiable soil layer, grain size characteristics, fines content of the soil and SPT blow counts. Bartlett and Youd (1995) have concluded that lateral spreading is restricted to sediments with corrected SPT blow counts of 15 or less for earthquake magnitudes less than or equal to 8.0. The potential of lateral spreading in the liquefiable soil below the groundwater table is not considered an adverse impact to the proposed development due to the limited amount of liquefaction potential and the distance between the Otay River face of slope located to the north of the site and the proposed buildings.

The mitigation of potential hazards due to liquefaction can be accomplished by the densification of the potentially liquefiable soil through ground improvements or the use of foundation systems that still provide acceptable structural support should liquefaction occur. Soil densification can be accomplished by compaction grouting, vibrocompaction, soil mixing, and deep dynamic compaction (among others). Soil densification is generally used to increase the density and provide liquefaction mitigation of sensitive soil to relatively shallow depths over large areas. Deep foundation systems may be used to transmit structural loads to bearing depths below the liquefiable zones and may consist of driven piles or drilled piles. Deep foundations are designed to mitigate damage to the structures supported on the piles; however, they do not generally reduce the potential for damage to underground utilities and peripheral site improvements. The effects of differential settlement between ridged structures and attached settlement-sensitive surface improvements can be mitigated by designing the utilities to accommodate differential movement at the connections.

6.5 Storm Surge, Tsunamis and Seiches

Storm surges are large ocean waves that sweep across coastal areas when storms make landfall. Storm surges can cause inundation, severe erosion and backwater flooding along the water front. The site is located approximately ½ miles from the San Diego Bay and is at an elevation of about 15 feet or greater above MSL. Based on historic and predicated wave heights and runout lengths, we opine that the proposed site elevation is sufficient to mitigate the risk; therefore, the potential of storm surges affecting the site is considered low.

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first-order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). Historically, tsunami wave heights have ranged up to 3.7 feet in the San Diego area. According to the County of San Diego Hazard Mitigation Plan (2010), the largest tsunami effect recorded in San Diego since 1950 was May 22, 1960 which had maximum run-up amplitudes of 2.1 feet (0.7 meters). Wave heights and run-up elevations from tsunamis along the San Diego Coast have historically fallen within the normal range of the tides. The California Geologic Survey (CGS) *Tsunami Inundation Map for Emergency Planning Imperial Beach Quadrangle* maps tsunami inundation areas. The subject site is not included within a tsunami inundation area.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is located adjacent to the Otay River at an elevation of about 15 feet or greater above Mean Sea Level (MSL). Based on historic and predicated wave heights and runout lengths, it is our opinion that the proposed site elevation is sufficient to mitigate the risk; therefore, we consider the potential for seiches to impact the site low.

6.6 Landslides

We did not observe evidence of previous or incipient slope instability at the site during our study as the property is relatively flat. Published geologic mapping indicates landslides are not present on or adjacent to the site. Therefore, in our professional opinion, the potential for a landslide is not a significant concern for this project.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 We did not encounter soil or geologic conditions during our exploration that would preclude the proposed development, provided the recommendations presented herein are followed and implemented during the design and construction of the planned development. We will provide supplemental recommendations if we observe variable or undesirable conditions during construction, or if the proposed construction will differ from that anticipated herein.

7.1.2 The site may be subject to geologic hazards, including moderate to strong seismic shaking, liquefaction, seismically induced settlement and consolidation settlement. We included recommendations for the mitigation of these geologic hazards herein.

7.1.3 Our field investigation and the previous investigation performed by CWE indicates the site is underlain by undocumented fill and/or topsoil overlying alluvium, Old Paralic Deposits and the San Diego Formation. The undocumented fill, topsoil and upper portion of the alluvium is not considered suitable for the support of additional fill and/or settlement-sensitive building structures in its current state and will require remedial grading. The Old Paralic Deposits and Santiago Formation are considered suitable for the support of compacted fill and settlement-sensitive structures.

7.1.4 We encountered groundwater at depths of 13.5 and 16 feet in current Borings B-1 and B-2 (elevations of about 6.5 and 4 feet MSL). In addition, CWE reported that groundwater was encountered at depths ranging from 8 to 13 feet (elevations of about 7 to 12 feet MSL). We also encountered saturated alluvial soils at depths of approximately 5 feet in current Borings B-1 and B-2 (elevation of about 15 feet MSL). Groundwater and saturated soils will likely have a significant influence on construction of deep utilities and subterranean structures (if proposed) and during remedial grading. Dewatering will likely be required for excavations below the fluctuating groundwater elevation and preliminary recommendations are provided herein. The project should be designed with a groundwater elevation of 10 feet MSL, and saturated conditions should be estimated at an elevation of approximately 15 feet MSL.

7.1.5 Excavation of the undocumented fill, topsoil and alluvium should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. We do not expect Old Paralic Deposits or the San Diego Formation will be encountered during the grading operations.

7.1.6 The upper portion of the alluvium is considered compressible and has a potential for liquefaction and seismically induced settlement. Removal and recompaction of the upper portions of these materials will be required in areas intended to support structural improvements. Because of the presence of groundwater, complete removal of the alluvium will likely not be possible. Remedial grading recommendations are provided hereinafter for areas where groundwater is encountered. We expect up to approximately 10 to 15 feet of compressible and liquefaction prone alluvium will be left in place during grading operations due to groundwater considerations. We expect the alluvium left in place and new compacted fills will settle up to 1¾ inches. We estimate the differential settlement would be 1 inch in 40 feet. We expect the settlement would occur within roughly 90 to 120 days after fill placement.

7.1.7 The proposed structures can be founded on mat foundations or post-tensioned shallow foundations designed to resist total and differential settlement. Recommendations for foundation systems are presented herein.

7.1.8 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas. Recommendations for site drainage are provided herein.

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

7.1.10 Based on our review of the project plans, we opine the planned development can be constructed in accordance with our recommendations provided herein. We do not expect the planned development will destabilize or result in settlement of adjacent properties.

7.1.11 Canyon subdrains will not be required on this project.

7.2 Soil Characteristics

7.2.1 The soil encountered in the field investigation is considered to be “expansive” (expansion index [EI] of greater than 20) as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 7.2 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a “very low” to “low” expansion potential (EI of 50 or less) in accordance with ASTM D 4829.

TABLE 7.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

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

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

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

7.3 Grading

7.3.1 Grading should be performed in accordance with the recommendations provided in this report, the Recommended Grading Specifications contained in Appendix F and the City of San Diego Grading Ordinance. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during the fill placement.

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

7.3.3 Site preparation should begin with the removal of deleterious material, debris, and vegetation. The depth of vegetation removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during

stripping and/or site demolition should be exported from the site. Asphalt and concrete should not be mixed with the fill soil unless approved by the Geotechnical Engineer.

7.3.4 Abandoned foundations and buried utilities (if encountered) should be removed and the resultant depressions and/or trenches should be backfilled with properly compacted material as part of the remedial grading.

7.3.5 The upper 2 to 3 feet of surficial materials (undocumented fill, topsoil and alluvium) should be removed and replaced with compacted fill within the limits of grading. Deeper removals may be required if soft materials are encountered. Geocon Incorporated should be on-site during the removal process/grading operations to evaluate removal depths.

7.3.6 The bottom of swimming pool/spa areas, if proposed, should not be supported by different types of materials. A layer of common bearing material is needed to provide uniform support for the pool/spa. The pool/spa excavation should be overexcavated to a depth of approximately 2 to 3 feet above groundwater (elevation of approximately 9 feet above MSL) or 3 feet below the bottom of the swimming pool/spa, whichever is shallower, and recompacted up to rough grade. We expect stabilization with rock and/or reinforcement will be required for the pool grading operations.

7.3.7 Some areas of overly wet and saturated soil will be encountered during removals due to the variable groundwater elevation and existing saturated soil. The saturated soil would require considerable drying and mixing effort prior to placement as compacted fill. Special equipment such as swamp cats, excavators and top loading operations may be required to excavate wet alluvium. Stabilization of the soil would include scarifying and air-drying, removing, mixing with drier soils, and replacement with drier soil, use of stabilization fabric (e.g. Tensar TX7, Mirafi HP370 or other approved fabric), or chemical treating (i.e. cement treatment).

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

7.3.9 The site should then be brought to final subgrade elevations with structural fill compacted in layers. In general, soil native to the site is suitable for use as fill if relatively free from vegetation, debris and other deleterious material. Layers of fill should be no thicker than

will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content, as determined in accordance with ASTM D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

7.3.10 Finish elevations across the majority of the site that will have surficial soils left in place will experience significant settlements in the first 90 days. Therefore, fine pad grading should be performed after the 90-day primary consolidation period has completed such that finish grade elevations are maintained and construction of the planned structures can begin.

7.4 Temporary Excavations

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

7.4.2 Temporary slopes should be made in conformance with OSHA requirements and as directed by the assigned competent person in the field (contractor). Stable undocumented fill, topsoil and alluvium can be considered a Type C soil, and properly compacted fill should be considered a Type B soil (Type C if seepage is encountered) in accordance with OSHA requirements. In general, special shoring requirements will not be necessary if temporary excavations will be less than 4 feet in height. Temporary excavations greater than 4 feet in height, however, should be sloped back at an appropriate inclination. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

7.5 Seismic Design Criteria – California Building Code

7.5.1 We used the Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) web application *Seismic Design Maps* (https://seismicmaps.org/) to evaluate site-specific seismic design parameters in accordance with the 2016 CBC/ASCE 7-10. Table 7.7.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section

1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The Site Class can be designated as a Site Class F due to the potential of liquefaction in the upper 15 feet; however, the building structure and improvements can be designed using a Site Class E because we expect the planned structures will possess a structural period of less than 0.5 seconds. A site specific seismic analysis will be required if the structural periods of the planned structures are greater than 0.5 seconds. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The Site Class designation could change based on the results of additional field explorations at the site. The values presented in Table 7.7.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.7.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	E	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.015g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.383g	Figure 1613.3.1(2)
Site Coefficient, F_A	0.900	Table 1613.3.3(1)
Site Coefficient, F_V	2.467	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.913g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.946g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.609g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.630g	Section 1613.3.4 (Eqn 16-40)

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

TABLE 7.7.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.430g	Figure 22-7
Site Coefficient, F_{PGA}	0.900	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.387g	Section 11.8.3 (Eqn 11.8-1)

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

7.6 Post-Tensioned Foundation Recommendations

7.6.1 The proposed buildings can be supported on a post-tensioned concrete slab and foundation systems founded in properly compacted fill subsequent to approximately 90 to 120 days after fill placement, or subsequent to soil mitigation. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2016 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented in Table 7.9.1. The parameters presented in Table 7.9.1 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 7.9.1
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) DC10.5 Design Parameters	Value
Thornthwaite Index	-20
Equilibrium Suction	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3
Edge Lift, y_M (inches)	0.61
Center Lift Moisture Variation Distance, e_M (feet)	9.0
Center Lift, y_M (inches)	0.30

7.6.2 The parameters presented in Table 7.9.1 are based on soil characteristics only (EI of 50 of less). We can provide final post-tensioned foundation system design parameters for each building or lot after finish pad grades have been achieved and we perform laboratory testing of the subgrade soil.

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

7.6.4 If the structural engineer proposes a post-tensioned foundation design method other than PTI, DC 10.5:

- The deflection criteria presented in Table 7.9.1 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

7.6.5 Foundation systems for the buildings that possess a foundation Category I and a "very low" expansion potential (expansion index of 20 or less) can be designed using the method described in Section 1808 of the 2016 CBC. If post-tensioned foundations are planned, an alternative, commonly accepted design method (other than PTI) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of 1 inch. Geocon Incorporated should be contacted to review the plans and provide additional information, if necessary.

7.6.6 If an alternate design method is contemplated, Geocon Incorporated should be contacted to evaluate if additional expansion index testing should be performed to identify the lots that possess a "very low" expansion potential (expansion index of 20 or less).

7.6.7 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift from tensioning, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

7.6.8 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the

footings/grade beams and the slab during the construction of the post-tension foundation system unless designed by the structural engineer.

7.6.9 The foundations may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. The estimated maximum total and differential settlement for the planned structures due to foundation loads is ½ inch.

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

7.7 Mat Foundations

7.7.1 The planned buildings can be supported on a mat foundation. A mat foundation consists of a thick, rigid concrete mat that allows the entire footprint of the structure to carry building loads. In addition, the mat can tolerate significantly greater differential movements such as those associated with expansive soils or differential settlement.

7.7.2 The allowable bearing capacity can be taken as 500 pounds per square foot (psf). The modulus of subgrade reaction for design of the mat can range from 75 to 100 pounds per cubic inch (pci) for the compacted fill. These values should be modified as necessary using standard equations for mat size as required by the structural engineer. We expect total and differential settlements to be 1½ and ¾ inches in 40 feet, respectively, under static loads for these buildings.

7.7.3 The modulus of subgrade reaction can range from 100 to 150 pounds per cubic inch (pci) for the proposed compacted fill. These values should be modified as necessary using standard equations for mat size as required by the structural engineer. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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

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

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

7.7.6 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

7.8 Settlement Considerations

7.8.1 The undocumented fill, topsoil and upper portion of the alluvial deposits are compressible when subjected to increased vertical stress (placement of fill soils and building loads) and will require special foundation design, grading and construction phasing measures to reduce the potential for adverse settlement. We expect up to approximately 10 to 15 feet of compressible alluvium will be left in place subsequent to grading operations. We recommend that construction of buildings and improvements in areas where compressible alluvium is left in place be delayed until periodic settlement monitoring indicates that primary consolidation of the underlying saturated alluvial deposits is essentially complete. This will require finish grade elevations be achieved to fully load the compressible surficial soils left in place and create the required settlements. Based on the current plans, we expect fills of approximately 5 to 7 feet will be required to achieve grade within the building areas.

7.8.2 Estimates of potential settlement are generally based on the thickness of compressible alluvium left-in-place, the thickness of additional fill placed to achieve finish grade, and the

compressibility characteristics of the alluvial materials. The rate of settlement is generally based on the compressibility characteristics of the materials, and the drainage path thickness to allow for pore water pressure dissipation.

7.8.3 Laboratory test results, engineering analyses, and our experience indicate that up to 1¾ inches of total consolidation settlement could occur below the planned buildings after finish grades have been established. This is assuming the upper 3 feet of surficial soils are removed and replaced with properly compacted fill soils and up to approximately 15 feet of undocumented fill, topsoil and compressible alluvium will be left in-place.

7.8.4 The magnitude of the total settlement and the associated time rate of consolidation will not be uniform throughout the site due to the variable thickness and compressibility characteristics of the underlying compressible materials and the variable thickness of additional fills to achieve finish grade.

7.8.5 As discussed, we expect the consolidation settlement due to the placement of the planned new fill to take 90 to 120 days. This time could be reduced through the use of surcharge fill loading or the installation of wick drains, if possible. The use of surcharge fill may be advantageous for the proposed buildings supported on shallow foundations, while surcharging the portions of the site planned for non-structural improvements may not be necessary. Surcharge fill is additional fill material placed on above finish grade (in building pad areas) to decrease the settlement time for consolidation prior to construction of improvements. The use of surcharge fill requires double handling of the surcharge soil and is generally 5 to 10 feet thick plus the thickness of the proposed fill extending slightly outside the building footprint. Surcharging does not generally provide mitigation of liquefaction potential but does reduce the potential for surface manifestation.

7.8.6 Surficial soil with a potential for settlement will be left in-place beneath the planned improvements. Therefore, settlement monitoring using plate and surface settlement monuments will be required as discussed herein to evaluate when the settlement has stabilized and further improvements may proceed. We will evaluate the number, locations and type of settlement monuments during grading operations based on the final limits or removals performed.

7.8.7 Settlement plates should be installed within the bottom of the over-excavated areas where compressible surficial materials will be left in place. At the designated locations, a 4-foot-square, ½-inch-thick steel plate should be placed at 2 feet below the bottom elevation and leveled with a layer of gravel. After surveying the plate (horizontal and vertical), the plate should be covered with 2 feet of gravel and at least 2 feet of fill soil placed before heavy

equipment is allowed to run over the plate location. The steel plate, gravel, and equipment for installation should be provided by the grading contractor. Figure 4 presents a typical plate settlement monument detail.

7.8.8 Subsequent to placement of planned fill, Geocon will drill and "tag" the steel plate. The civil engineer will then survey the plate to determine settlement which occurred during grading. At that time a surface settlement monument will be installed at finished grade at the location of the plate monument to monitor settlement movement of the underlying fill and surficial materials thereafter. Figure 5 presents a typical surface settlement monument detail.

7.8.9 The project surveyor should record the movements of the plate and surface settlement monuments every two weeks until data indicates that the rate of primary fill and left in place surficial material soil compression is essentially non-detrimental (settlement monument data with a relatively level plateau) to proposed improvements. When we receive two to three data points of settlement values that show a relatively level settlement slope on the graphs, the construction of the buildings and surrounding utility improvements can begin. Based on our experience, we expect the monuments will require monitoring for roughly 90 to 120 days. At that time, we expect development can begin for settlement-sensitive underground utilities with less than one percent gradient along with construction of the buildings. Underground utilities with a gradient of one percent or greater will not have a waiting period and can start construction after finish grade is achieved. Underground wet utilities should not be installed until finish grade is achieved, as excessive settlements will occur with the placement of compacted fills. We will evaluate the location of the settlement monuments subsequent to the grading operations.

7.8.10 The planned buildings should be designed with the appropriate settlement. Table 7.10.1 provides a summary of the estimated static and seismic settlements for the project.

TABLE 7.10.2
SUMMARY OF ESTIMATED SETTLEMENT DUE TO STATIC AND SEISMIC LOADING

Building Foundation Type	Estimated Total Seismic Settlement (inches)	Estimated Total Static Settlement Due to Placement of Planned Fill (inches)	Estimated Static Settlement Due to Building Loads (inches)
Post-Tension Foundation	1	1¾	½
Mat Foundation	1	1¾	½

7.9 Mitigation of Liquefaction and Settlement

7.9.1 Based on our analysis discussed herein, we estimate a potential for up to 1 inch of total liquefaction settlement using CPT data.. In addition, we estimate up to approximately 1¾ inches of static settlement can occur within the existing compressible soils that will be left in-place subsequent to grading. Mitigation of liquefiable and compressible soil may be necessary for settlement-sensitive structures. Several alternatives are generally available for mitigation including deep foundations, ground improvements and structural mitigation.

7.9.2 The proposed buildings can be supported on a mat foundation or post-tensioned foundation provided the estimated seismic and static settlements due to liquefaction discussed herein are within the settlement tolerance for the foundation type selected. The recommendations for a mat foundation system and shallow post-tensioned foundation are presented herein.

7.9.3 Ground improvement techniques mitigate liquefaction by densifying existing soil through the use of stone columns, deep dynamic compaction, compaction grouting, soil mixing or other densification method. We do not recommend that deep dynamic compaction be used for densification due to the proximity of adjacent facilities and the limited influence depth of the method in fine grained materials. In addition, compaction grouting may not be economical due to the expected depth and the area of the required improvements.

7.9.4 Ground improvement using stone columns may be used from a geotechnical engineering standpoint that consist of densifying existing soils with a vibrating probe and placing crushed rock.. However, the amount of fine-grained material may limit the effectiveness of the stone columns. The pattern and depth of ground improvements may vary depending on the purposes of mitigation and stratigraphic conditions but the depths of improvement would be approximately 30 feet with spacing of approximately 8 to 10 feet and a diameter of 30 inches. The specialty contractor should determine the final spacing and diameter to obtain the necessary densification as outlined below. Following stone column construction, CPTs should be performed to check if the soil mitigation is successful. The post-production CPTs should show a maximum of 1 inch of total (static and seismic) settlements using generally accepted calculation parameters, unless noted otherwise by the structural engineer

7.9.5 Soil/cement mixing is a soil improvement technique of mechanically blending a cementitious binder into existing unsuitable soils to create load bearing columns. As the soil mixing tool is advanced into the ground, cement-based slurry is pumped through the hollow stem of the shaft and injected into the soil through jets located on the backside of the leading rotating mixing blades. The mixing blades on the tool mix the soil with the slurry. Injection and mixing will continue to design depth. When design depth is reached, the mixing tool is withdrawn, leaving behind stabilized soil mix columns. Soil mix piles are typically designed

and installed by a specialty geotechnical contractor. Soil mix piles should derive support in the competent Old Paralic Deposit materials found at or below an approximate depth of 15 to 20 feet below the existing ground surface.

7.9.6 Following soil modification construction, the upper 3 to 4 feet of the upper soils will be disturbed and should be removed and recompacted as discussed herein.

7.9.7 The mitigation should extend at least 15 feet laterally outside the edge of planned building structures, where practical. Mitigation within non-structure areas will be limited to that improvement obtained by nearby remediation within structure areas. We can provide additional recommendations for the ground improvement techniques when the improvement has been selected.

7.9.8 The mitigation should extend at least 15 feet laterally outside the edge of planned building structures, where practical. Mitigation within non-structure areas will be limited to that improvement obtained by nearby remediation within structure areas. We can provide additional recommendations or the ground improvement techniques when the improvement has been selected.

7.10 Exterior Concrete Flatwork

7.10.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein assuming the subgrade materials possess an expansion index of 90 or less. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh or No. 3 reinforcing bars spaced 18 inches center-to-center in both directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

7.10.2 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to

the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

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

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

7.10.5 The subgrade soil for flatwork not subjected to vehicle loads should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. Flatwork that will experience vehicular loading (e.g. driveways) should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. We should check the subgrade soil prior to concrete and steel placement.

7.11 Retaining Walls

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

7.11.2 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

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

7.11.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2016 CBC or Section 11.6 of ASCE 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 13H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.39g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.3. Figure 6 presents a retaining wall loading diagram.

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

7.11.6 In general, wall foundations having a minimum depth and width of 1 foot may be designed for an allowable soil bearing pressure of 2,000 psf. The allowable soil bearing pressure may be increased by an additional 300 psf for each additional foot of depth and width, to a maximum allowable bearing capacity of 3,000 psf. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

7.11.7 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load.

Figure 7 presents a typical retaining wall drainage detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.11.8 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls (such as mechanically stabilized earth [MSE] walls, soil nail walls, or soldier pile walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

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

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

7.12 Lateral Loading

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

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

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

7.13 Pool/Spa Recommendations

7.13.1 If a swimming pool/spa is proposed, it should be reinforced and designed by a structural engineer. The existing soil possesses a "very low" to "low" expansion potential (expansion index [EI] of 50 or less). The corresponding lateral pressures used for the design should be equivalent to a fluid pressure of at least 40 pcf, if drained. The swimming pool/spa also should be designed for possible surcharge loading if such nearby loading is a lateral distance from the top of the pool equal to the depth of the pool and hydrostatic loading if the walls are not drained. The walls should be designed with an equivalent to a fluid pressure of at least 100 pcf if not drained.

7.13.2 The bottom of the swimming pool/spa should not be supported by different types of materials. A layer of common bearing material is needed to provide uniform support for the pool/spa. The pool/spa excavation should be overexcavated to a depth of approximately 2 to 3 feet above groundwater (elevation of approximately 9 feet above MSL) or 4 feet below the bottom of the swimming pool/spa, whichever is shallower, and recompacted up to rough grade. We expect stabilization with rock and/or reinforcement will be required for the pool grading operations. The bottom of the excavation should be scarified, moisture conditioned as necessary, and compacted in accordance with the "Grading" section of this report.

7.13.3 Surface drainage around the pool/spa should be designed to prevent water from ponding and seeping into the ground. Surface water should be collected and conducted through non-erosive devices to the street, storm drain or other approved water course or disposal area. Leakage from the proposed pool/spa could create an artificial groundwater condition that will likely create instability problems. Therefore, all plumbing and the pool/spa should be leak free.

7.13.4 The deck for the swimming pool/spa should be cast separately of the swimming pool/spa, and water stops should be provided between the bond beam and the deck. Jointing for concrete flatwork should be provided in accordance with the recommendations of the American Concrete Institute. The joints should be sealed with an approved flexible sealant to reduce the potential for introduction of surface water into the underlying soil.

7.13.5 The flatwork can be underlain by a minimum of 12 inches of clean sand compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above

optimum moisture content. This would help with reducing the potential for expansion of the underlying soil and lifting of the concrete flatwork.

7.13.6 Consideration should be given to installing a subdrain system for the pool area. The subgrade surface should be graded to slope a minimum of 1 percent away from the pool. An impermeable liner (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent PVC liner) could be placed over the subgrade soil. The liner, if installed, should overlap by at least 12 inches and sealed in accordance with manufacturer's recommendations.

7.13.7 To mitigate the potential for moisture infiltration into the subgrade soils beneath the pool deck, we recommend the construction of a deepened footing along the outside edge of the pool deck flatwork.

7.13.8 A subdrain consisting of 4-inch diameter perforated PVC pipe should be installed inside the deepened footing and sloped to drain into an approved outlet. The pipe should be surrounded by ¾ inch open-graded gravel and wrapped with filter fabric.

7.13.9 Our recommendations will not eliminate the potential for pavement and hardscape distress due to expansive soil. Concrete pavements are lightweight when compared to swell pressures exerted by highly expansive soil. Therefore, movement of the concrete could still occur if the clayey subgrade soils fluctuate in moisture content due to desiccation or water infiltration.

7.14 Preliminary Pavement Recommendations

7.14.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have assumed an R-Value of 25 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 7.15.1 presents the preliminary flexible pavement sections.

TABLE 7.15.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete Thickness (inches)		
			3	3½	4
			Class 2 Aggregate Base (inches)		
Parking stalls for automobiles and light-duty vehicles	5.0	25	7	5	4
Driveways for automobiles and light-duty vehicles	5.5	25	8	7	6
Medium truck traffic areas	6.0	25	---	9	8
Driveways for heavy truck traffic	7.0	25	---	---	11

7.14.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

7.14.3 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 7.15.2.

TABLE 7.15.2
RIGID PAVEMENT DESIGN PARAMETERS

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

7.14.4 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.15.3.

TABLE 7.15.3
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A)	5.5
Driveways (TC=C)	7.0

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

7.14.6 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., 5.5-inch and 7.0-inch-thick slabs would have an 7.5- and 9.0-inch-thick edge, respectively). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

7.14.7 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 12 feet for 5.5-inch-thick and 15 feet for the 6.0-inch and thicker slabs and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report. The depth of the crack-control joints should be at least ¼ of the slab thickness when using a conventional saw, or at least 1 inch when using early-entry saws on slabs 9 inches or less in thickness, as determined by the referenced ACI report discussed in the pavement section herein. Cuts at least ¼ inch wide are required for sealed joints, and a ⅜ inch wide cut is commonly recommended. A narrow joint width of $^1/_{10}$- to $^1/_8$-inch wide is common for unsealed joints.

7.14.8 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-

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

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

7.15 Site Drainage and Moisture Protection

7.15.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

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

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

7.15.4 We understand storm water management devices are planned for the proposed development and biofiltration basins are planned along the west and east perimeters of the site. Appendix E presents recommendations regarding storm water management.

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

7.16 Grading and Foundation Plan Review

7.16.1 Geocon Incorporated should review the grading and building foundation plans for the project prior to final design submittal to evaluate if additional analyses and/or recommendations are required.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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



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N

NO SCALE

VICINITY MAP

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LR / CW		DSK/GTYPD

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

DATE 04 - 24 - 2019	PROJECT NO. G2129 - 52 - 03	FIG. 1

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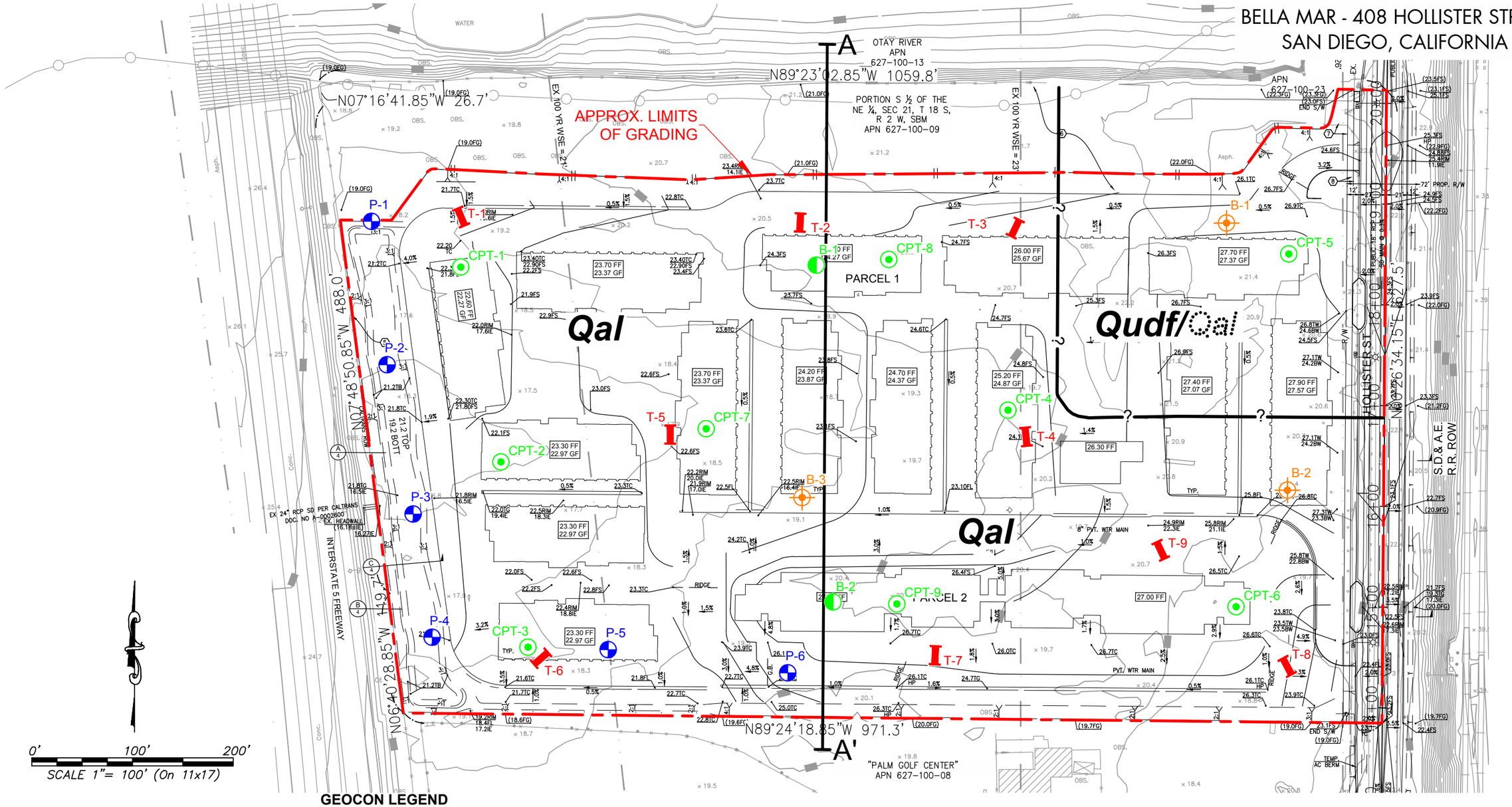
BELLA MAR - 408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA



GEOCON LEGEND

Qudf .....UNDOCUMENTED FILL

Qal .....ALLUVIUM (Dotted Where Buried)

B-2 .....APPROX. LOCATION OF GEOTECHNICAL BORING (Current Study)

CPT-9 .....APPROX. LOCATION OF CONE PENETROMETER TEST (Current Study)

P-6 .....APPROX. LOCATION OF INFILTRATION TEST (Geocon, 2017)

B-3 .....APPROX. LOCATION OF GEOTECHNICAL BORING (CWE, 2005)

T-9 .....APPROX. LOCATION OF TRENCH (CWE, 2005)

A A' .....APPROX. LOCATION OF CROSS-SECTION

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

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FIGURE 2

DATE 04 - 24 - 2019

GEOLOGIC MAP

Plotted:07/02/2019 9:20AM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2129-52-03 Bella Mar 408 Hollister Street\SHEETS\G2129-52-03 SitePLAN.dwg

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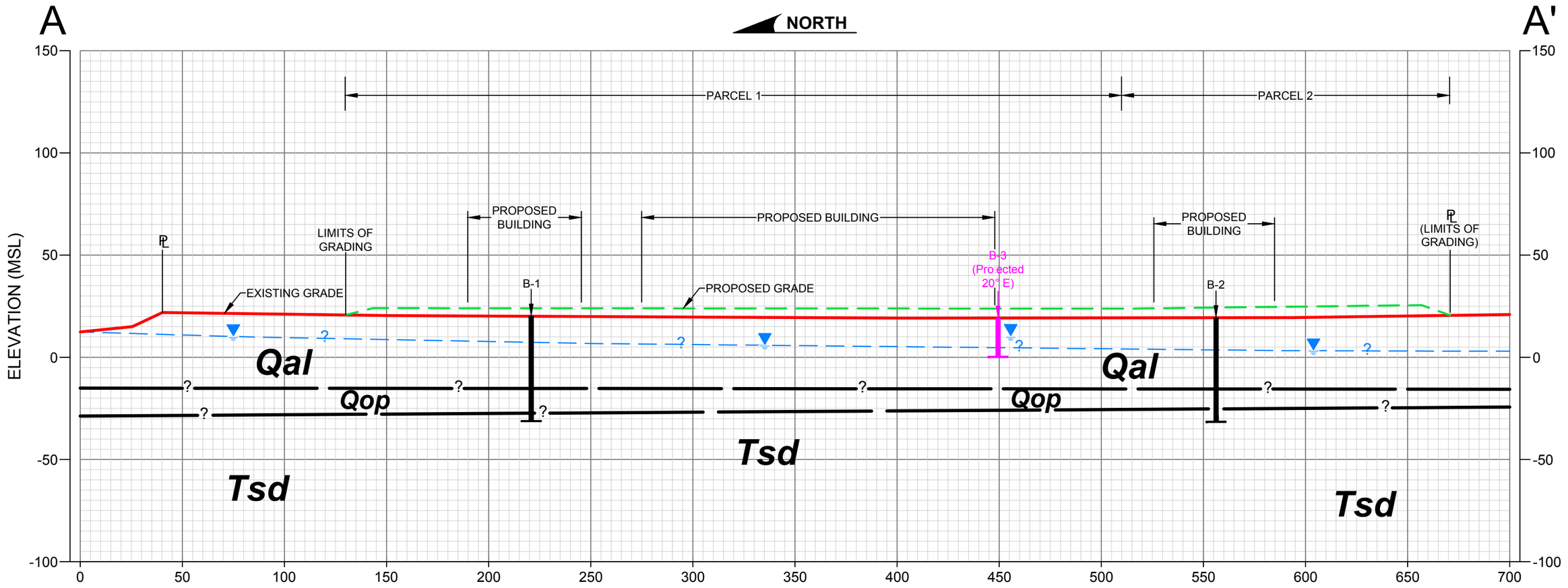
SAN DIEGO, CALIFORNIA



DISTANCE (FEET)

GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 50' (Vert. = Hori□.)

GEOCON LEGEND

Qal .....ALLUVIUM

Qop .....OLD PARALIC DEPOSITS

Tsd .....SAN DIEGO FORMATION

B-2 .....APPROX. LOCATION OF BORING (Current Study)

B-3 .....APPROX. LOCATION OF BORING (CWE, 2005)

.....ESTIMATED GROUNDWATER ELEVATION (Queried Where Uncertain)

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

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FIGURE 3
DATE 04 - 24 - 2019

GEOLOGIC CROSS-SECTION

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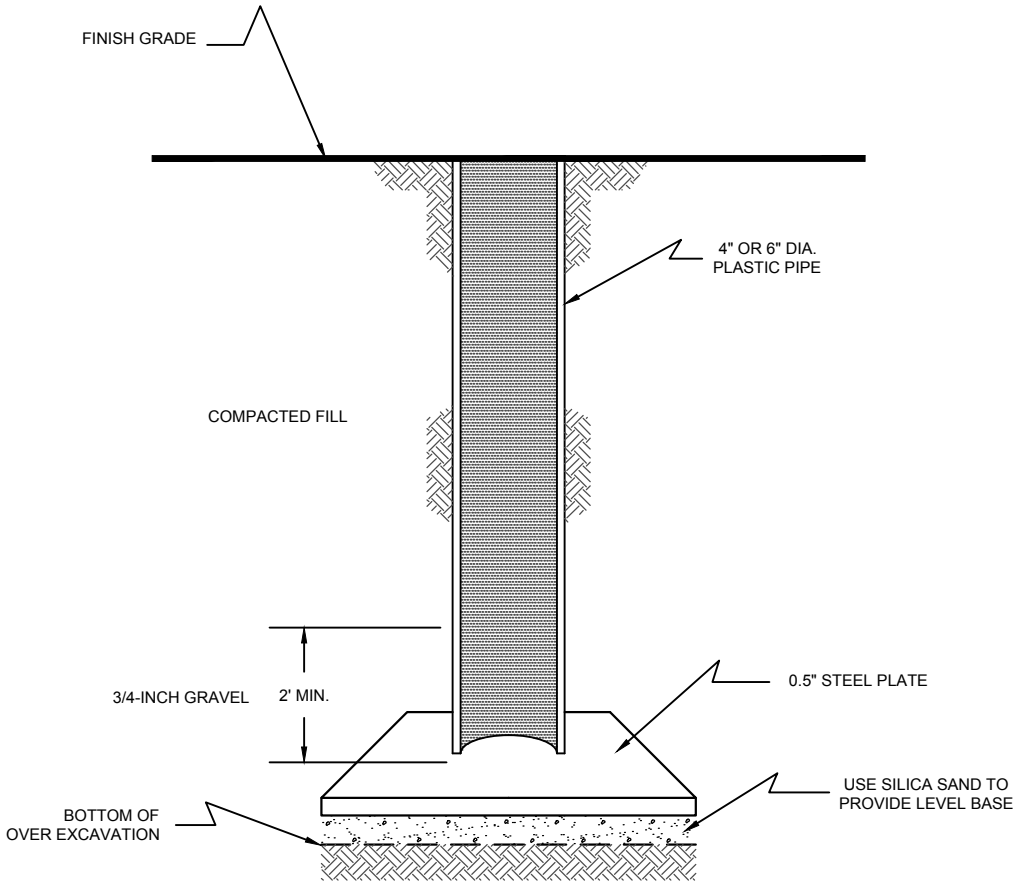

NOTES:

1.....STEEL PLATE SHOULD BE 4-FEET SQUARED. LOCATION OF SETTLEMENT PLATES SHALL BE SURVEYED AND VISIBLE (RED FLAG) TO EQUIPMENT OPERATORS.

2.....CONTRACTOR SHALL INSTALL SETTLEMENT PLATES. THERE SHOULD BE 2 FEET OF GRAVEL OVER THE SETTLEMENT PLATE. THERE SHOULD BE 2 FEET OF FILL OVER THE GRAVEL BEFORE HEAVY EQUIPMENT CAN OPERATE OVER THE PLATE.

3.....MULTIPLE READING CAN BE PERFORMED ON THE SETTLEMENT PLATE. A 4-INCH DIAMETER PVC PIPE SHOULD BE INSTALLED IF MULTIPLE READINGS ARE REQUIRED.

4.....THE EXCAVATION SHOULD BE PROPERLY BACKFILLED AFTER THE SETTLEMENT MONITORING PROGRAM IS COMPLETE.

NO SCALE

PLATE SETTLEMENT MONUMENT

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DATE 04 - 24 - 2019	PROJECT NO. G2129 - 52 - 03	FIG. 4

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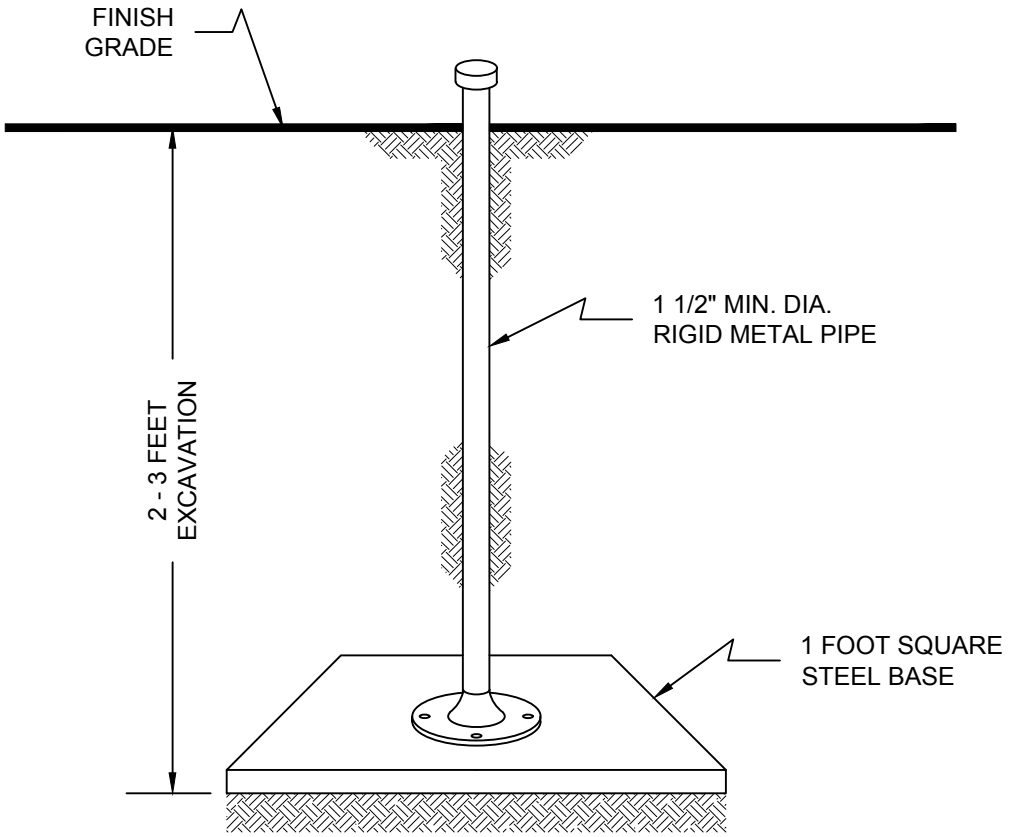

NOTES:

1.....LOCATION OF SETTLEMENT MONUMENTS SHALL BE CLEARLY MARKED AND READILY VISIBLE (RED FLAG) TO EQUIPMENT OPERATORS.

2.....CONTRACTOR SHALL MAINTAIN 10-FOOT HORI□ONTAL CLEARANCE FOR HEAVY EQUIPMENT WITHIN 5 FEET (VERTICAL) OF PLATE BASE. FILL WITHIN CLEARANCE AREA SHALL BE HAND COMPACTED TO PROJECT SPECIFICATIONS OR COMPACTED BY ALTERNATIVE APPROVED SOILS ENGINEER.

3.....IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE OR EXTENSION RESULTING FROM EQUIPMENT OPERATING WITHIN PRESCRIBED CLEARANCE AREA, CONTRACTORS SHALL IMMEDIATELY NOTIFY SOILS ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.

NO SCALE

SURFACE SETTLEMENT MONUMENT

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STLMNT2

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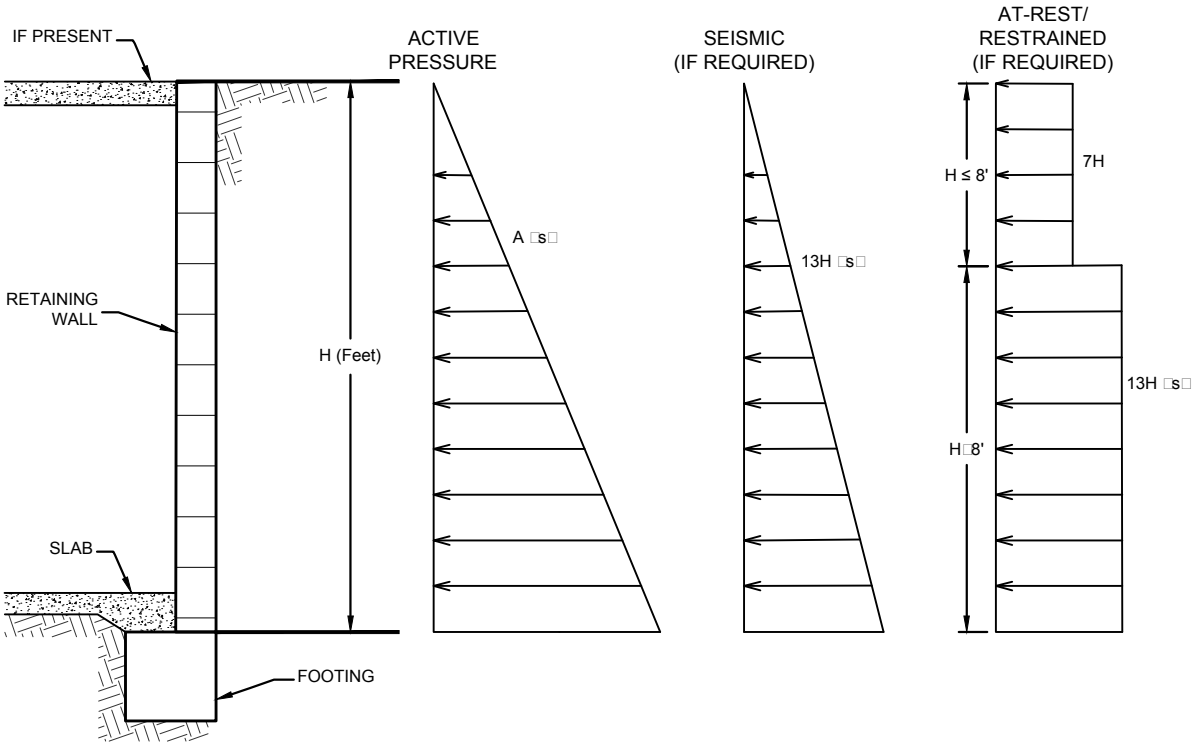

ACTIVE PRESSURE, A (□s□)		
EXPANSION INDEX, EI	LEVEL BACKFILL	2:1 SLOPING BACKFILL
EI ≤ 50	35	50
EI ≤ 90	40	55

NOTES:

1..... A SURCHARGE OF 2 FEET OF SOIL (250 PSF VERTICAL LOAD) SHOULD BE ADDED TO THE DESIGN OF THE WALL WHERE TRAFFIC LOADS ARE WITHIN A HORI□ONTAL DISTANCE EQUAL TO ⅔ THE WALL HEIGHT. OTHER SURCHARGES SHOULD BE APPLIED, AS APPLICABLE.

2..... EXPANSION INDEX GREATER THAN 50/90 SHOULD NOT BE USED FOR WALL BACKFILL PER REPORT.

3..... RETAINING WALLS SHOULD BE PROPERLY DRAINED AND WATER PROOFED.

4..... THE PROJECT STRUCTURAL ENGINEER SHOULD EVALUATE THE WALLS LOADING COMBINATIONS.

NO SCALE

RETAINING WALL LOADING DIAGRAM

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DATE 04 - 24 - 2019	PROJECT NO. G2129 - 52 - 03	FIG. 6

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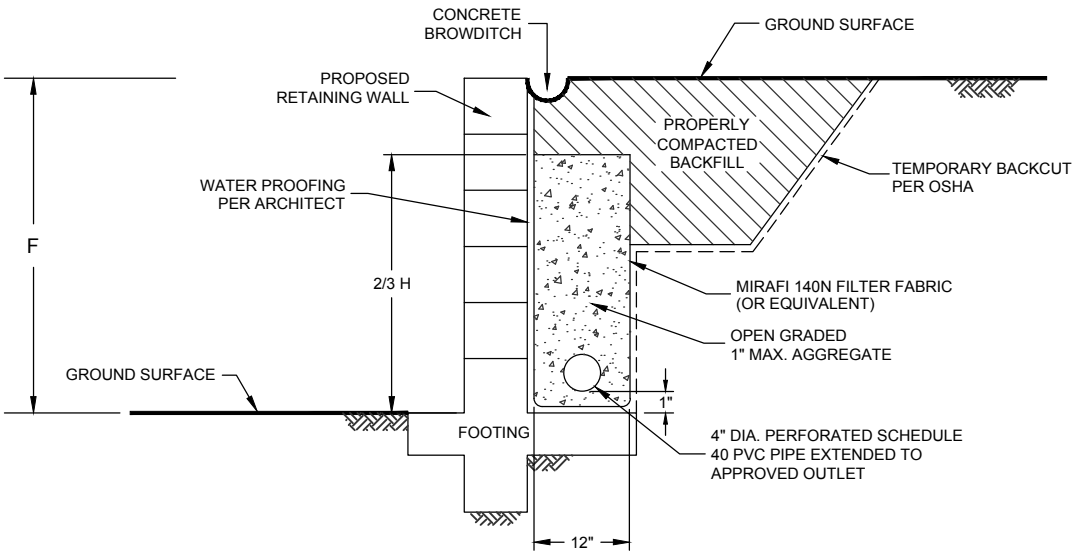

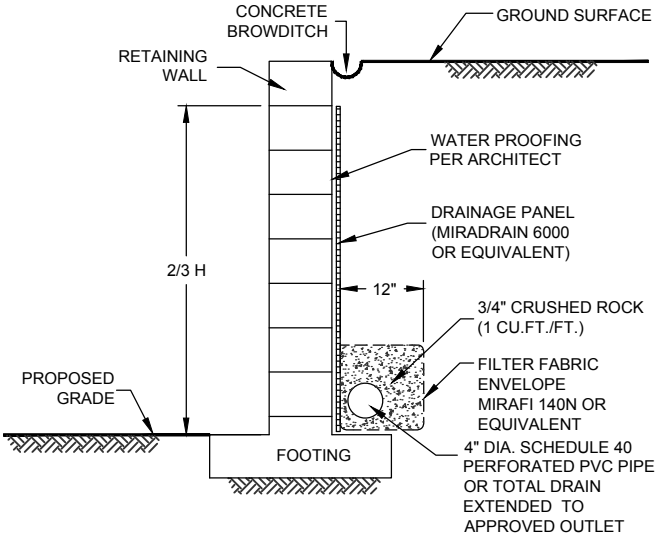

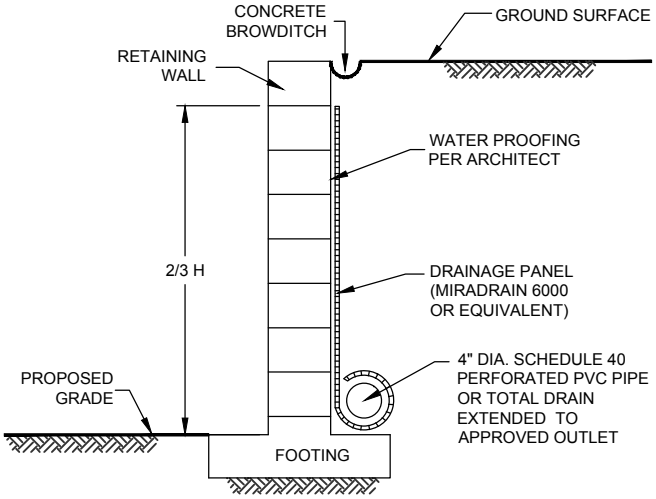

NOTE :

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

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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DATE 04 - 24 - 2019	PROJECT NO. G2129 - 52 - 03	FIG. 7

Plotted:04/24/2019 9:30AM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2129-52-03 Bella Mar 408 Hollister Street\DETAILS\Typical Retaining Wall Drainage Detail (RWDD7A).dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on April 26, 2017 and March 11 and 14, 2019 that consisted of excavating 2 exploratory borings, conducting 6 infiltration tests, and 9 CPTs. The borings and CPTs extended to a maximum depth of approximately 51 feet. The locations of the exploratory borings and CPTs are shown on the Geologic Map, Figure 2 (map pocket). The boring logs and CPTs are presented in this Appendix. We located the borings and CPTs in the field using a measuring tape and existing reference points; therefore, actual locations may deviate slightly.

The geotechnical borings were drilled to a depth of approximately 51 feet below existing grade using a CME 75 drill rig equipped with hollow-stem augers. The infiltration-test borings were excavated with hand digging equipment to depths of approximately 4.5 to 5 feet.

We obtained samples during our subsurface exploration in the borings using a California sampler. The sampler is composed of steel and is driven to obtain ring samples. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 3 inches. Up to 18 rings are placed inside the sampler that is 2.4 inches in diameter and 1 inch in height. We obtained ring samples at appropriate intervals, placed them in moisture-tight containers, and transported them to the laboratory for testing. The type of sample is noted on the exploratory boring logs.

The samplers were driven 12 inches. The sampler is connected to A rods and driven into the bottom of the excavation using a 140-pound hammer with a 30-inch drop. Blow counts are recorded for every 6 inches the sampler is driven. The penetration resistances shown on the boring logs are shown in terms of blows per foot. The values indicated on the boring logs are the sum of the last 12 inches of the sampler. If the sampler was not driven for 12 inches, an approximate value is calculated in term of blows per foot or the final 6-inch interval is reported. These values are not to be taken as N-values as adjustments have not been applied. We estimated elevations shown on the boring logs either from a topographic map or by using a benchmark. Each excavation was backfilled as noted on the boring logs.

We visually examined, classified, and logged the soil encountered in the borings in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). The logs depict the soil and geologic conditions observed and the depth at which samples were obtained.

Kehoe Testing & Engineering performed the CPT soundings. The soil conditions encountered during the field investigation were automatically logged in a nearly continuous profile of penetration resistance as each CPT sounding was being conducted. The recorded tip stress, sleeve stress, and pore pressure of the soil is used to develop a stratigraphic interpretation of the soil profile.

PROJECT NO. G2129-52-03

BORING B 1

ELEV. (MSL.) **20'** DATE COMPLETED **03-11-2019**

EQUIPMENT **CME 75** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B1-1	SM	**TOPSOIL** Loose, moist, dark brown, Silty, fine to medium SAND			
2		SC	**ALLUVIUM (Qal)** Loose, moist, dark brown, Clayey, fine to medium SAND			
4		SM	Loose, moist, dark brown, Silty, fine SAND			
6	B1-2		-Becomes saturated	15	105.4	22.7
8						
10	B1-3		-Becomes dark reddish brown	7	105.4	25.0
12						
14						
16	B1-4			8	99.0	30.8
			-Gravel			
18		GP	Very dense, saturated, brown to yellowish brown, fine to coarse Sandy GRAVEL; cobble up to 4 inches in diameter			
20	B1-5			83/10"	121.7	15.8
22						
24						
26	B1-6			82/10"		11.4
28						

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

G2129-52-03.GPJ

SAMPLE SYMBOLS

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

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

PROJECT NO. G2129-52-03

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING B 1** ELEV. (MSL.) **20'** DATE COMPLETED **03-11-2019** EQUIPMENT **CME 75** BY: **K. HAASE** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30				GP				
32				SP	Medium dense, saturated, brown, fine to medium SAND			
34								
36	B1-7			SM	**OLD PARALIC DEPOSITS (Qop)** Dense, saturated, light reddish brown, Silty, fine to coarse SAND	52	105.5	23.0
38								
40	B1-8				-Becomes yellowish brown	66	98.6	25.1
42								
44					-Becomes medium dense, brown			
46	B1-9			MH	Hard, wet, light reddish brown, fine Sandy SILT; micaceous	74		
48								
50	B1-10			SP	**SAN DIEGO FORMATION (Tsd)** Very dense, saturated, light brown, fine to medium SAND	90/11"	106.6	27.3
					BORING TERMINATED AT 51 FEET Groundwater encountered at 13.5 feet Backfilled with 58 cu. ft. of Portland I, II, V cement			

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

G2129-52-03.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2129-52-03

BORING B 2

ELEV. (MSL.) **20'** DATE COMPLETED **03-11-2019**

EQUIPMENT **CME 75** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B2-1	SM	**TOPSOIL** Loose, moist, dark brown, Silty, fine to medium SAND			
2		SC	**ALLUVIUM (Qal)** Loose, moist, dark brown, Clayey, fine to medium SAND			
4		SM	Loose, wet to saturated, brown, Silty, fine SAND; trace gravel			
6	B2-2			16	112.6	20.1
8						
10	B2-3		-No recovery	12		
12						
14						
16	B2-4	SM	Dense, saturated, brown, fine to coarse SAND; some silt, little gravel	89		
	B2-5			33		21.1
18						
20		SW-SM	Very dense, saturated, brown, fine to coarse Silty SAND; some gravel			
	B2-6			83/11"		
22						
24						
26	B2-7		-Becomes dark yellowish brown	90/11"	135.3	9.9
28						

G2129-52-03.GPJ

Figure A-8,
Log of Boring B 2, Page 1 of 2

SAMPLE SYMBOLS

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

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

PROJECT NO. G2129-52-03

BORING B 2

ELEV. (MSL.) **20'** DATE COMPLETED **03-11-2019**

EQUIPMENT **CME 75** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-8			SW-SM	-No recovery	50/1"		
32				SM	Medium dense, moist to wet, light olive brown, Silty, fine SAND; little gravel			
34				SM	**OLD PARALIC DEPOSITS (Qop)** Very dense, wet to saturated, brown, Silty, fine to coarse SAND			
36	B2-9					83		
38								
40	B2-10			ML	Very stiff, brown, moist to wet, fine Sandy SILT	25		
42								
44				CL	**SAN DIEGO FORMATION (Tsd)** Hard, moist to wet, light brown, Sandy CLAY; micaceous			
46	B2-11					58		
48				SP	Very dense, saturated, dark grayish brown, fine to medium SAND; micaceous			
50	B2-12					70/11"	98.2	25.8
					BORING TERMINATED AT 51 FEET Groundwater encountered at 16 feet Backfilled with 58 cu. ft. of Portland I, II, V cement			

Figure A-8,
Log of Boring B 2, Page 2 of 2

G2129-52-03.GPJ

SAMPLE SYMBOLS

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

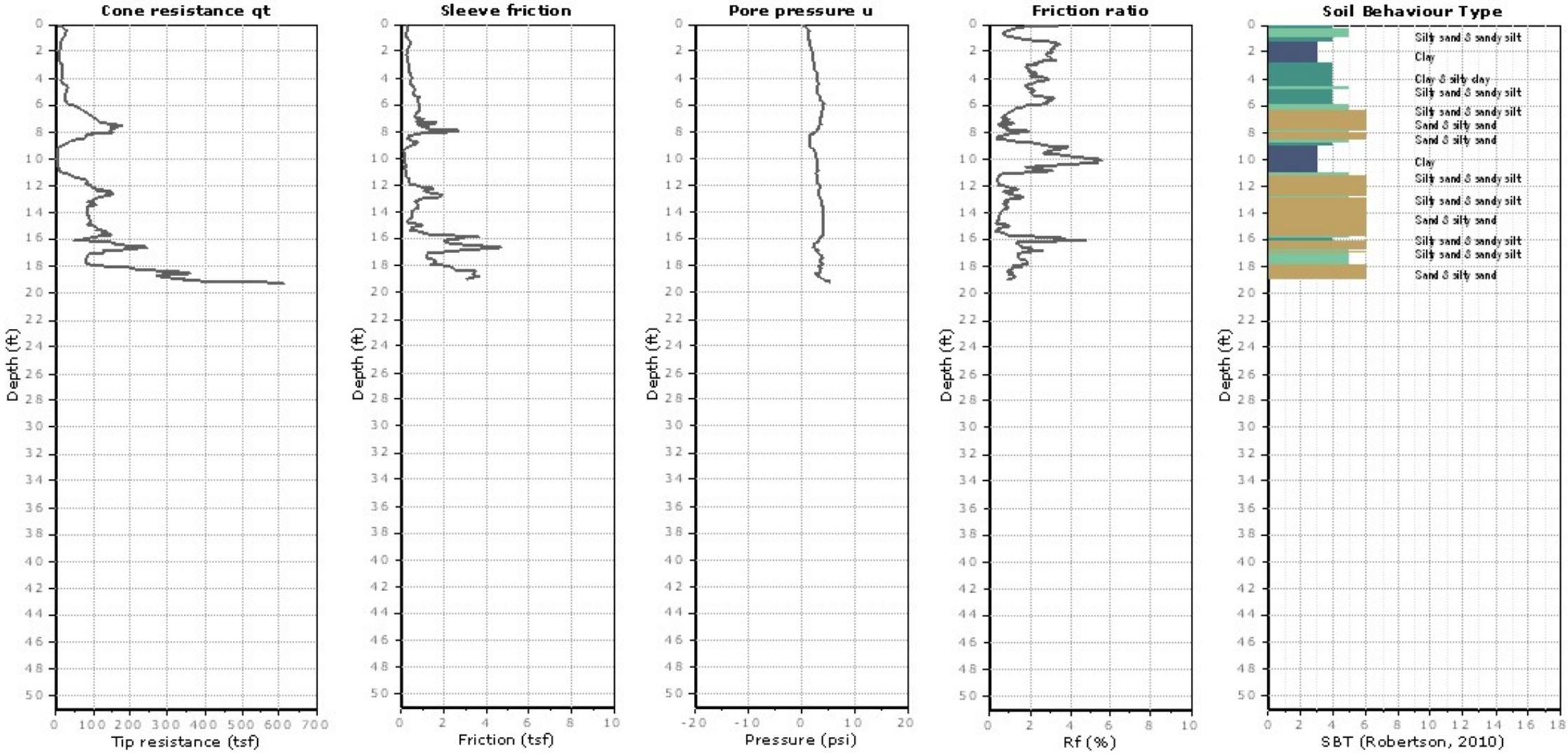
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

CPT-1

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 19.30 ft, Date: 3/14/2019
Cone Type: Vertek



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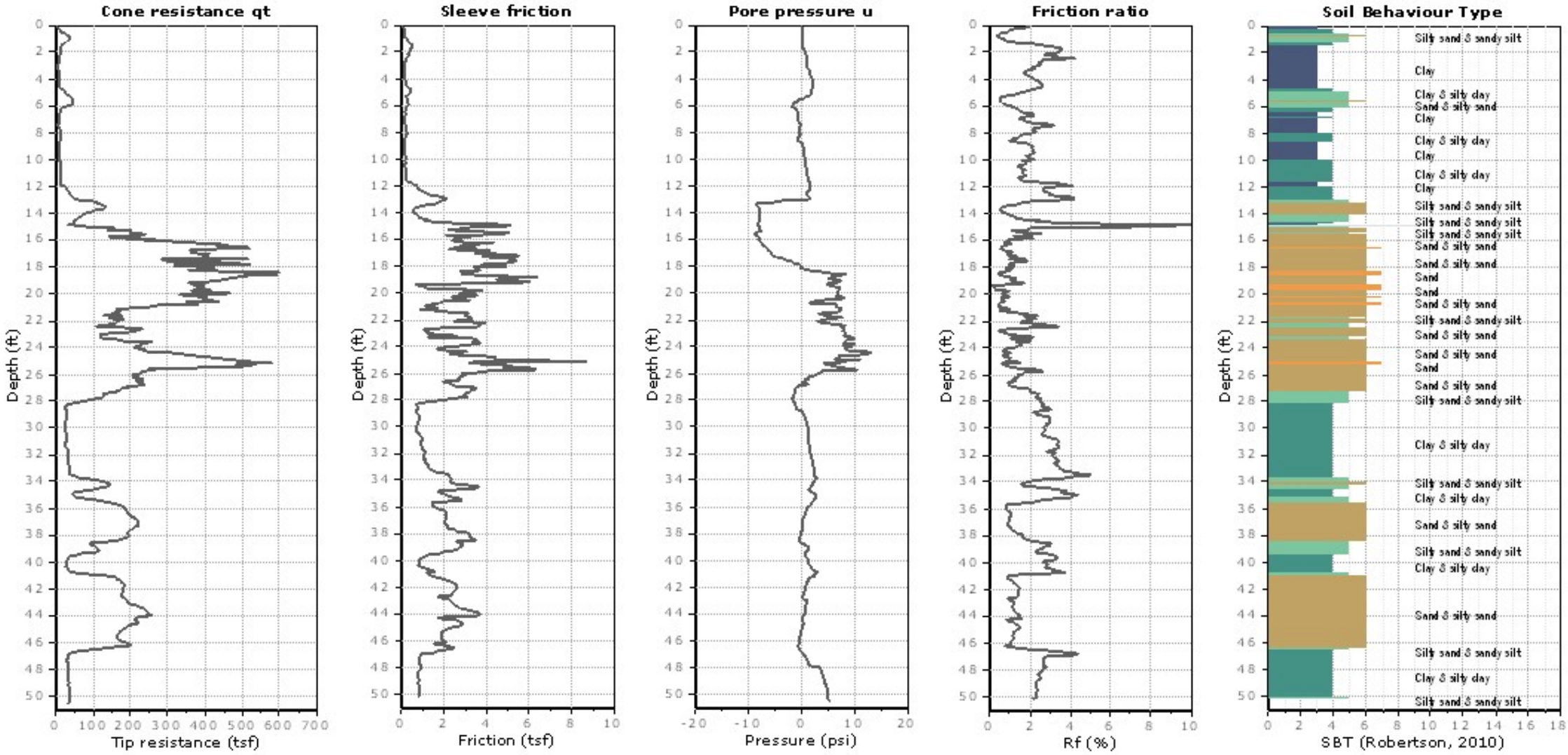
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

CPT-2

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 50.47 ft, Date: 3/14/2019
Cone Type: Vertek



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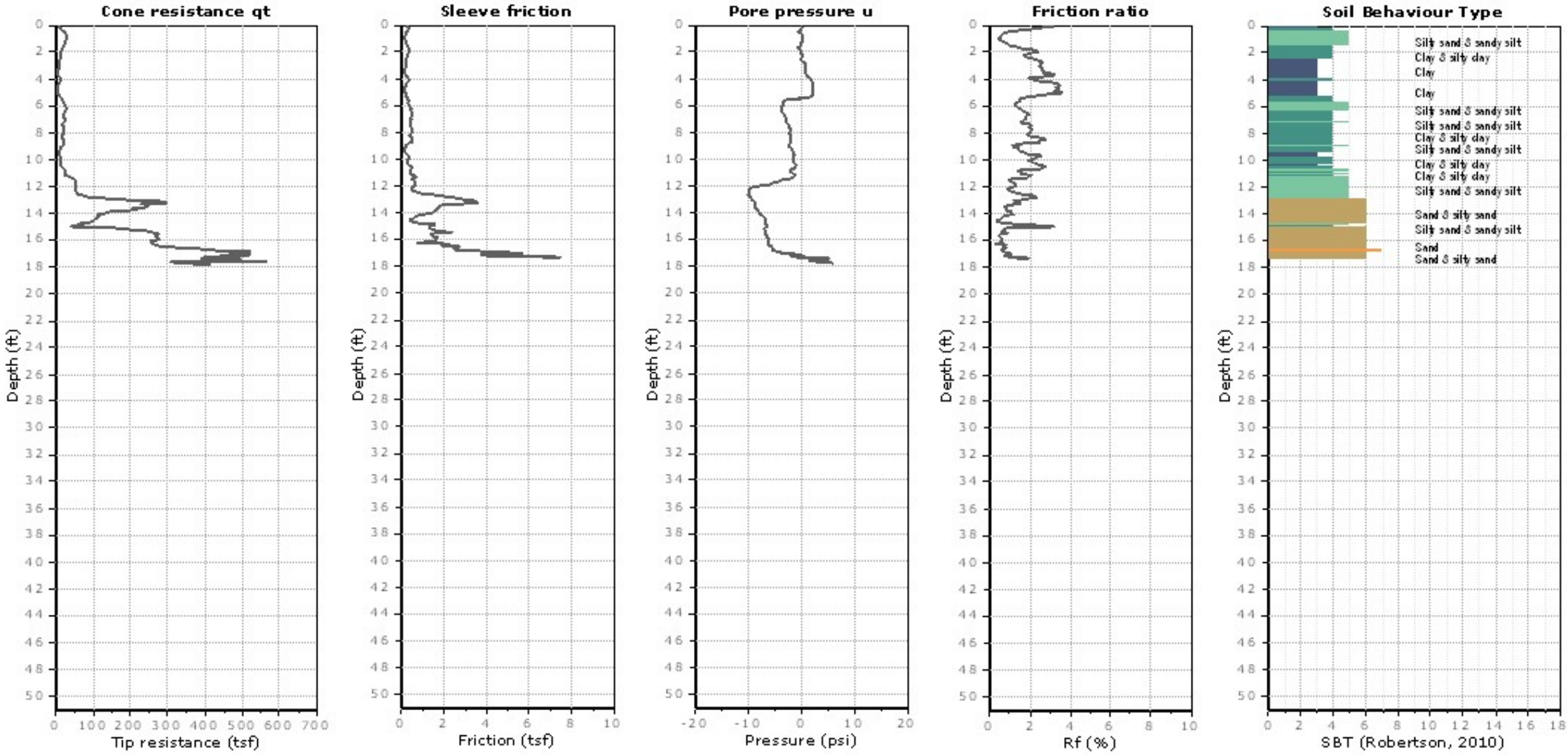
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

CPT-3

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 17.79 ft, Date: 3/14/2019
Cone Type: Vertek



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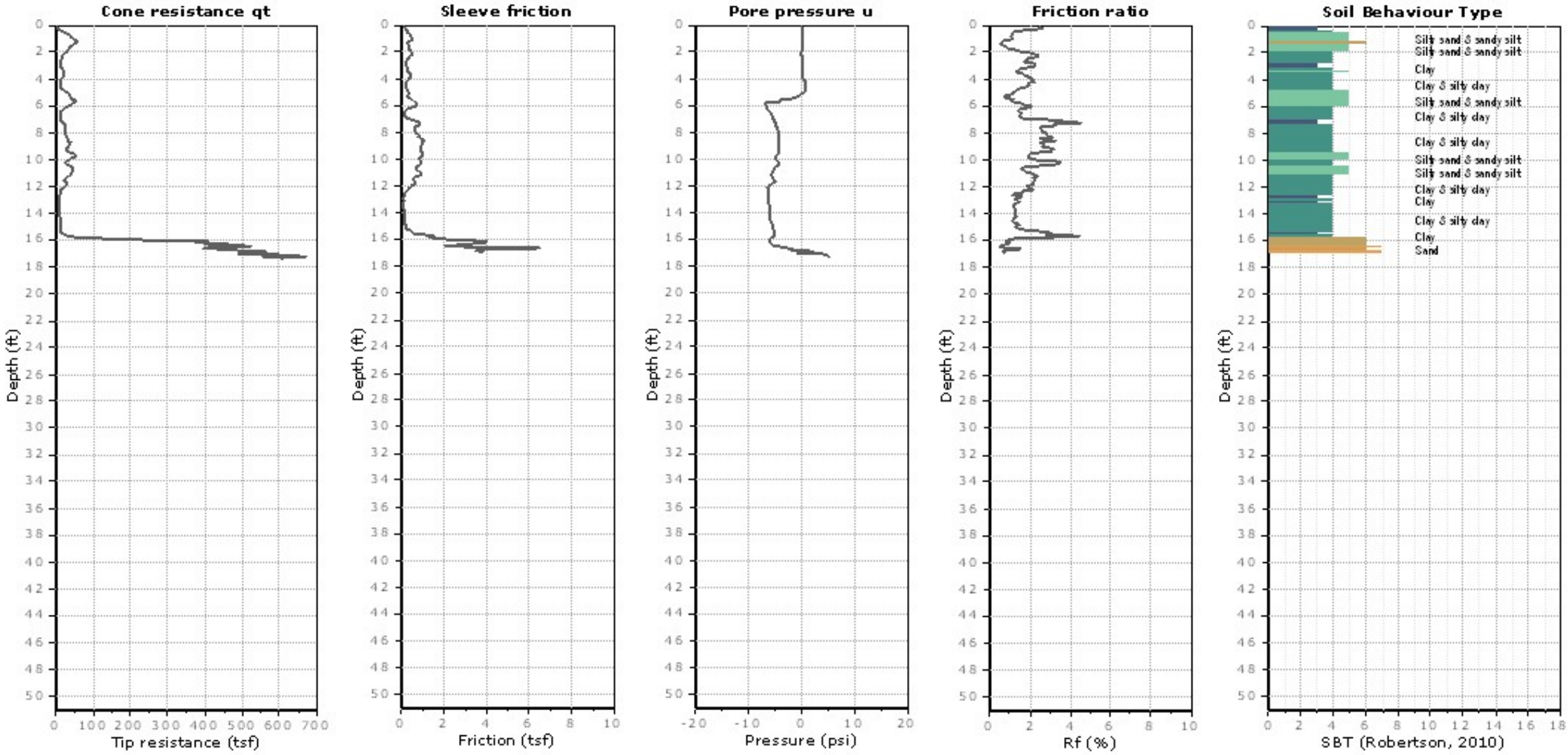
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

CPT-4

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 17.32 ft, Date: 3/14/2019
Cone Type: Vertek



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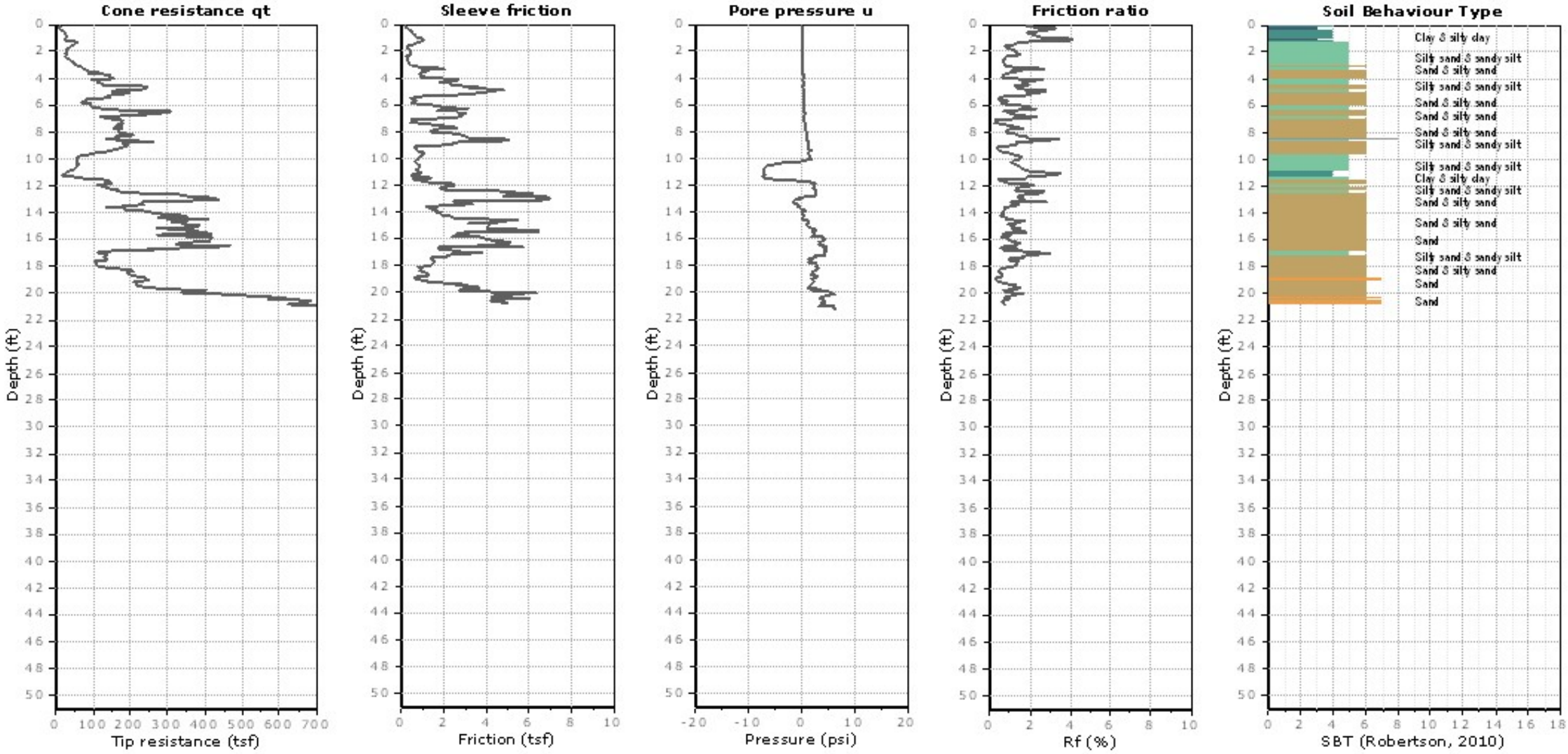
CPT-5

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 21.20 ft, Date: 3/14/2019
Cone Type: Vertek



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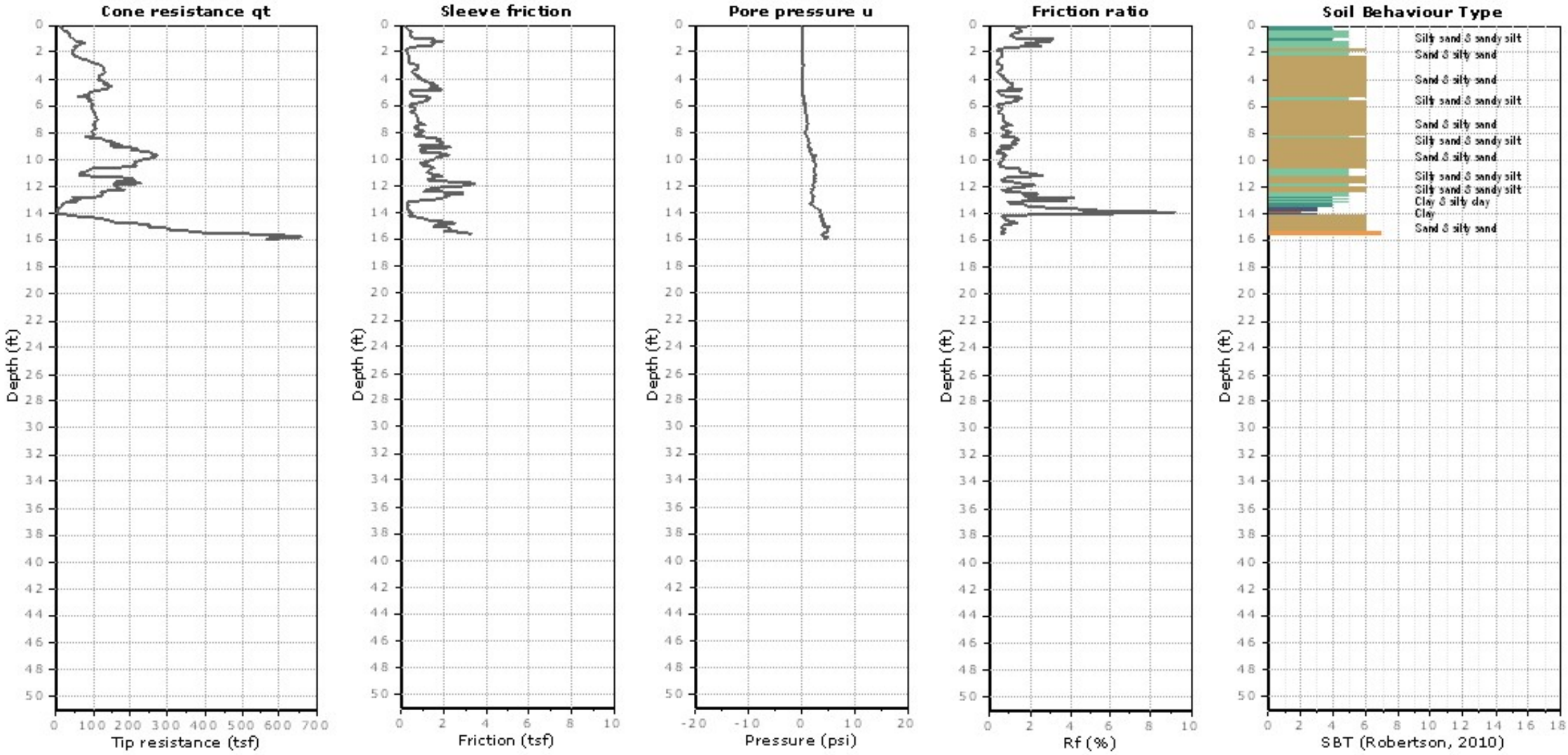
CPT-6

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 15.95 ft, Date: 3/14/2019
Cone Type: Vertek



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steve@kehoetesting.com
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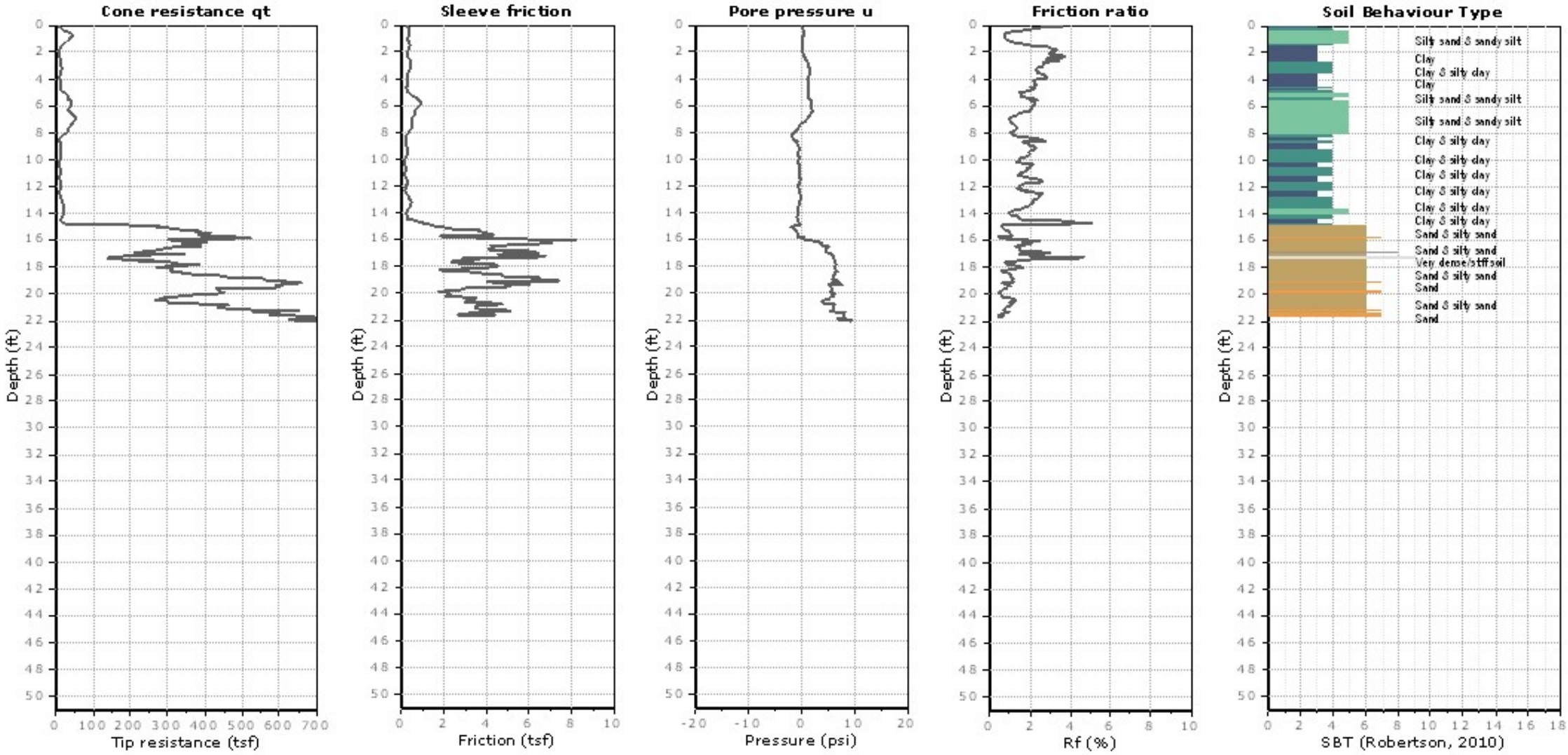
CPT-7

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 22.12 ft, Date: 3/14/2019
Cone Type: Vertek



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steve@kehoetesting.com
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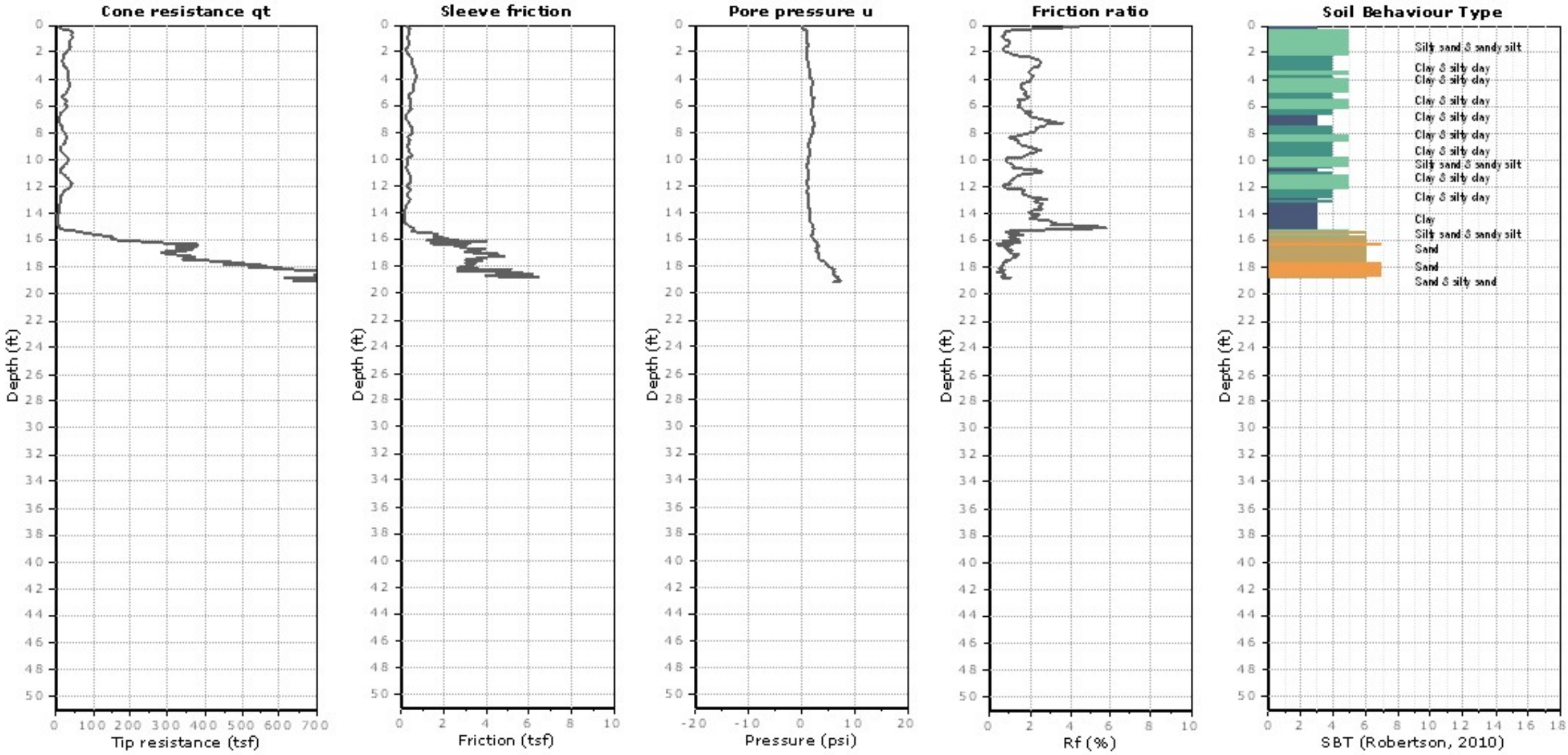
CPT-8

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 19.23 ft, Date: 3/14/2019
Cone Type: Vertek



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steve@kehoetesting.com
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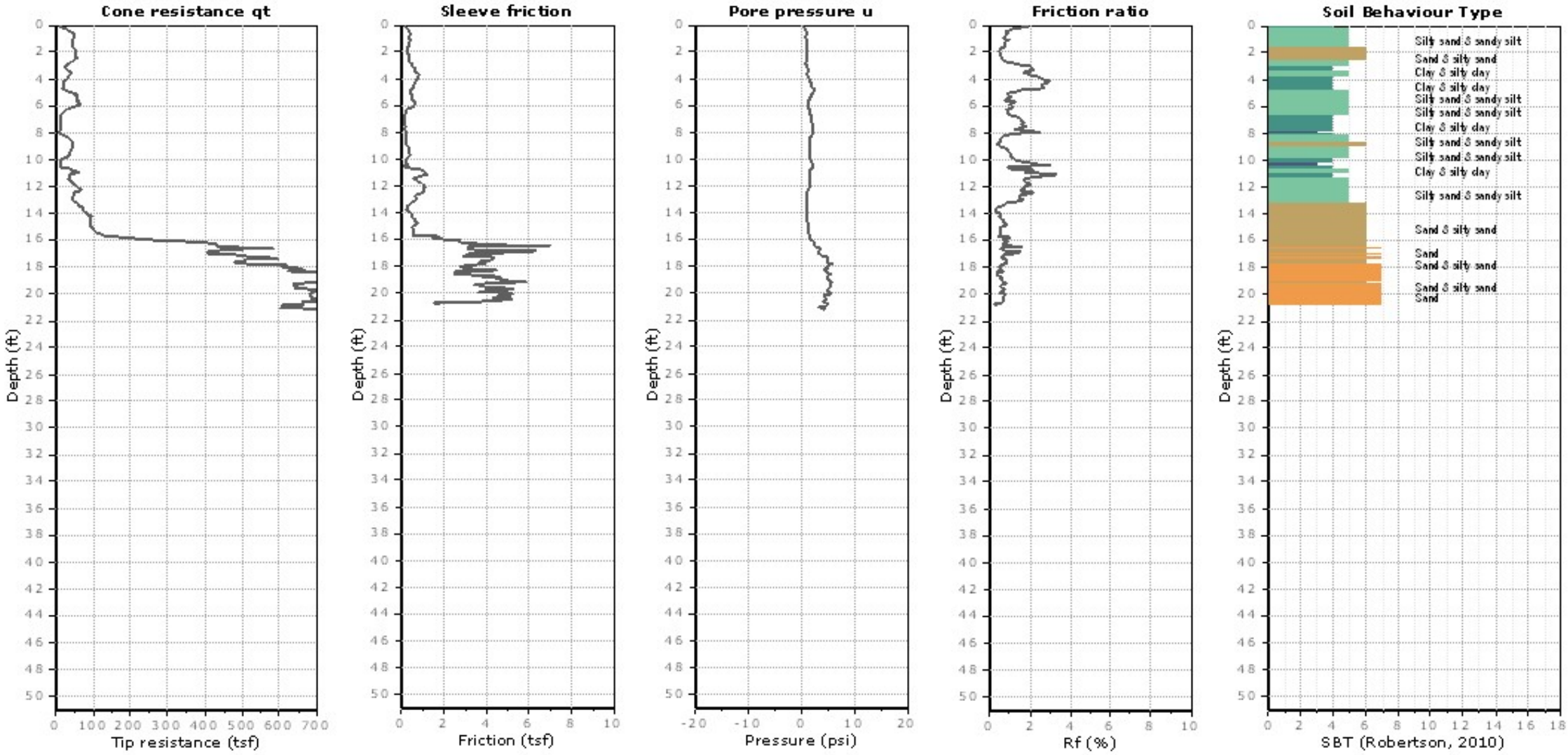
CPT-9

Project: **Geocon**
Location: **408 Hollister St, San Diego, CA**

Total depth: 21.20 ft, Date: 3/14/2019
Cone Type: Vertek



APPENDIX B

APPENDIX B

LABORATORY TESTING

The laboratory test results performed on representative materials are presented on the following tables. We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for in-place dry density and moisture content, maximum density/optimum moisture content, direct shear strength, expansion index, water-soluble sulfate, plasticity index, R-Value, unconfined compressive strength, gradation and consolidation characteristics. The results of our laboratory tests are presented in Tables B-I through B-VI and on Figures B-1 through B-5. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B1-1	Dark brown, Clayey fine to medium SAND; trace gravel	127.8	10.6
B2-1	Dark brown, Clayey fine to medium SAND	126.9	11.3

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

Sample No.	Depth (feet)	Geologic Unit	Dry Density (pcf)	Moisture Content (%)		Peak [Ultimate[1]] Cohesion (psf)	Peak [Ultimate[1]] Angle of Shear Resistance (degrees)
				Initial	Final		
B1-2	5	Qal	105.4	22.7	22.8	400 [400]	31 [31]
B1-8	40	Qal	98.6	25.1	25.2	600 [425]	34 [32]

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

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	2016 CBC Expansion Classification	ASTM Soil Expansion Classification
	Before Test	After Test				
B1-1	10.3	20.1	110.0	39	Expansive	Low
B2-1	9.5	18.2	111.7	24	Expansive	Low

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

Sample No.	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
B1-1	0.034	S0
B2-1	0.024	S0

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

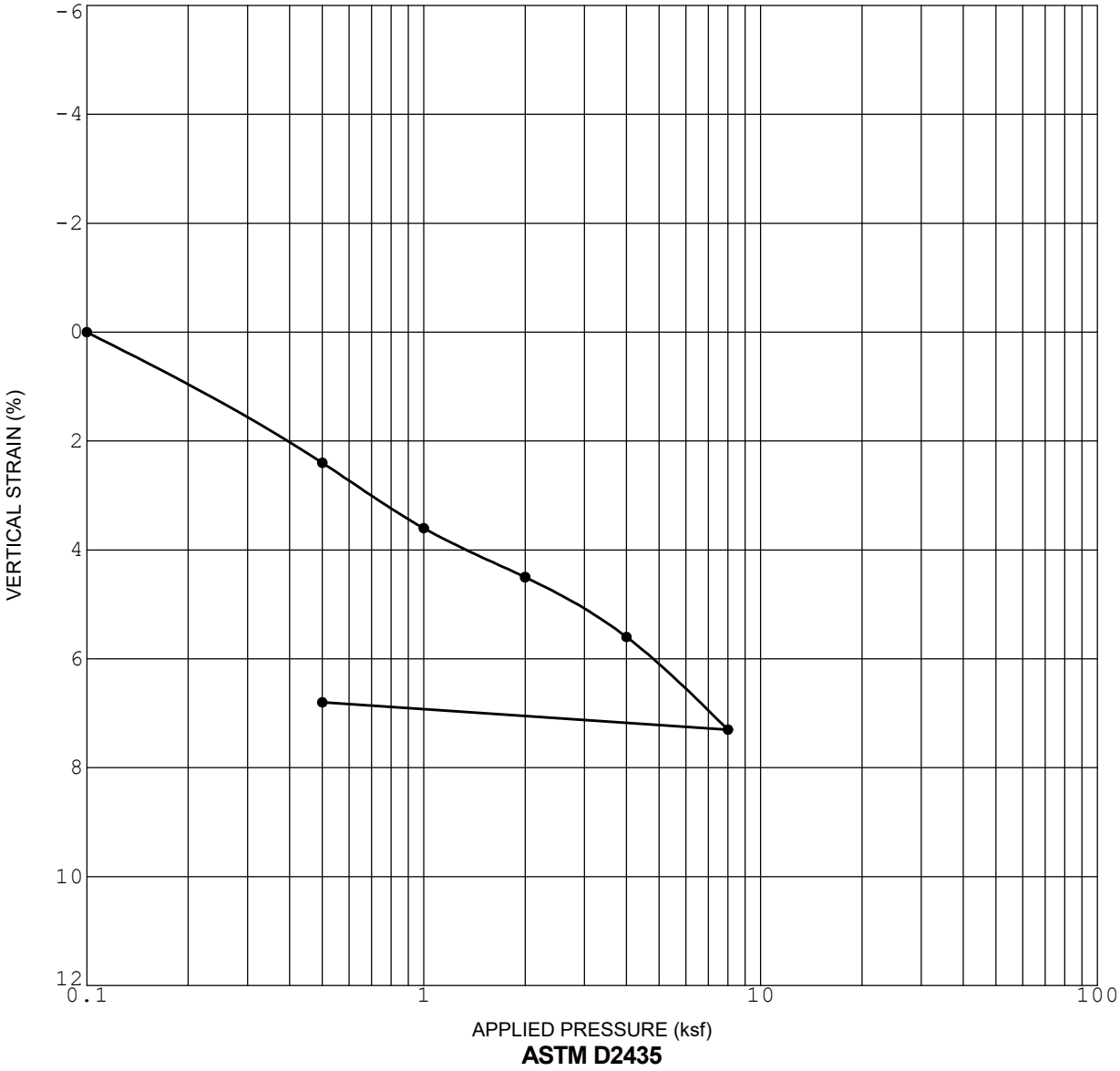
Sample No.	R-Value
B1-1	25
B2-1	40

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

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index
B1-9	35	27	8

PROJECT NO. G2129-52-03

SAMPLE NO. B1-3 @10'



ASTM D2435

Initial Dry Density (pcf)	105.4	**Initial Saturation (%)**	100+
Initial Water Content (%)	25.0	**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

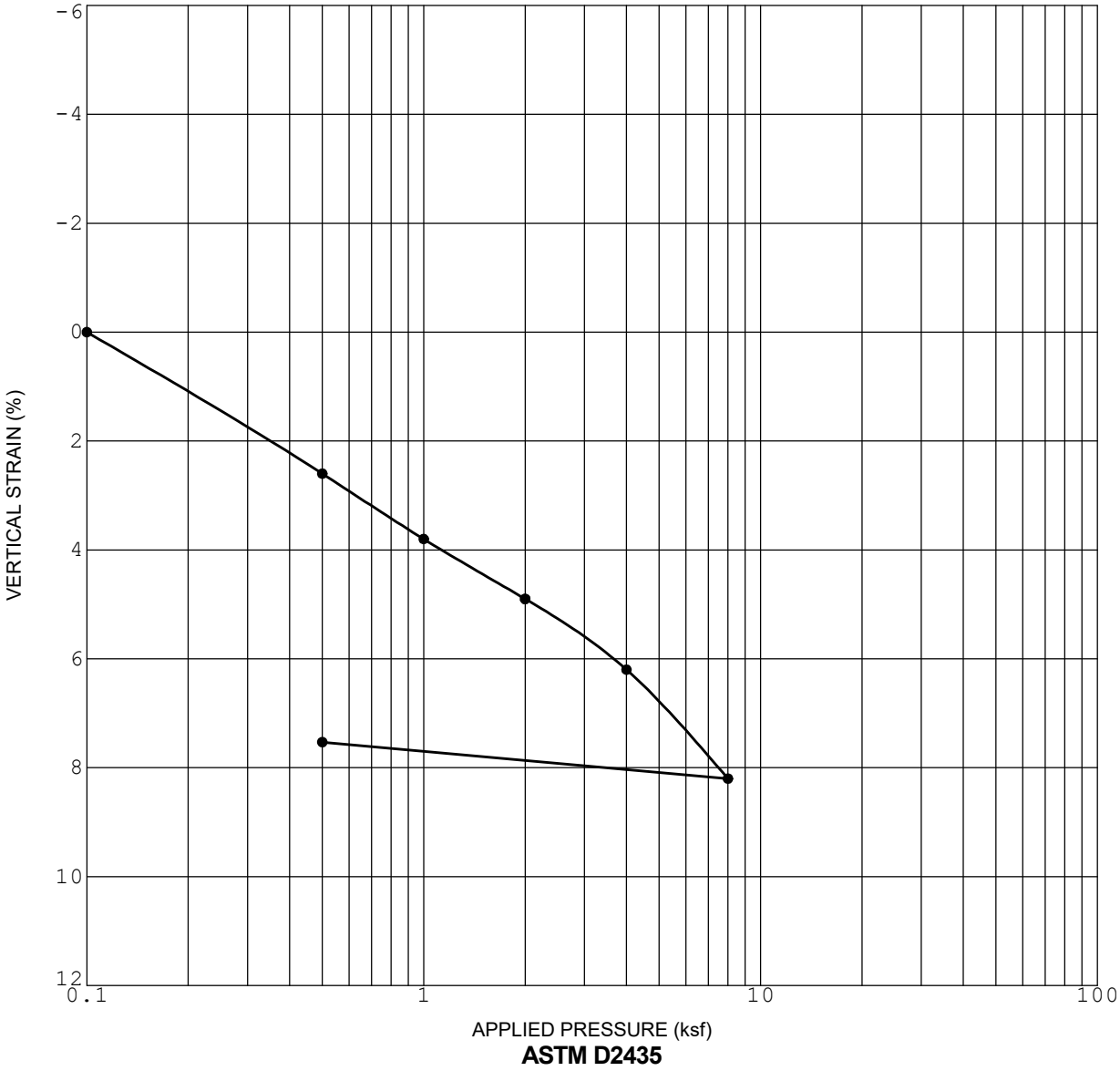
G2129-52-03.GPJ

Figure B-1

GEOCON

PROJECT NO. G2129-52-03

SAMPLE NO. B1-4 @15'



ASTM D2435

Initial Dry Density (pcf)	99.0	**Initial Saturation (%)**	100+
Initial Water Content (%)	30.8	**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

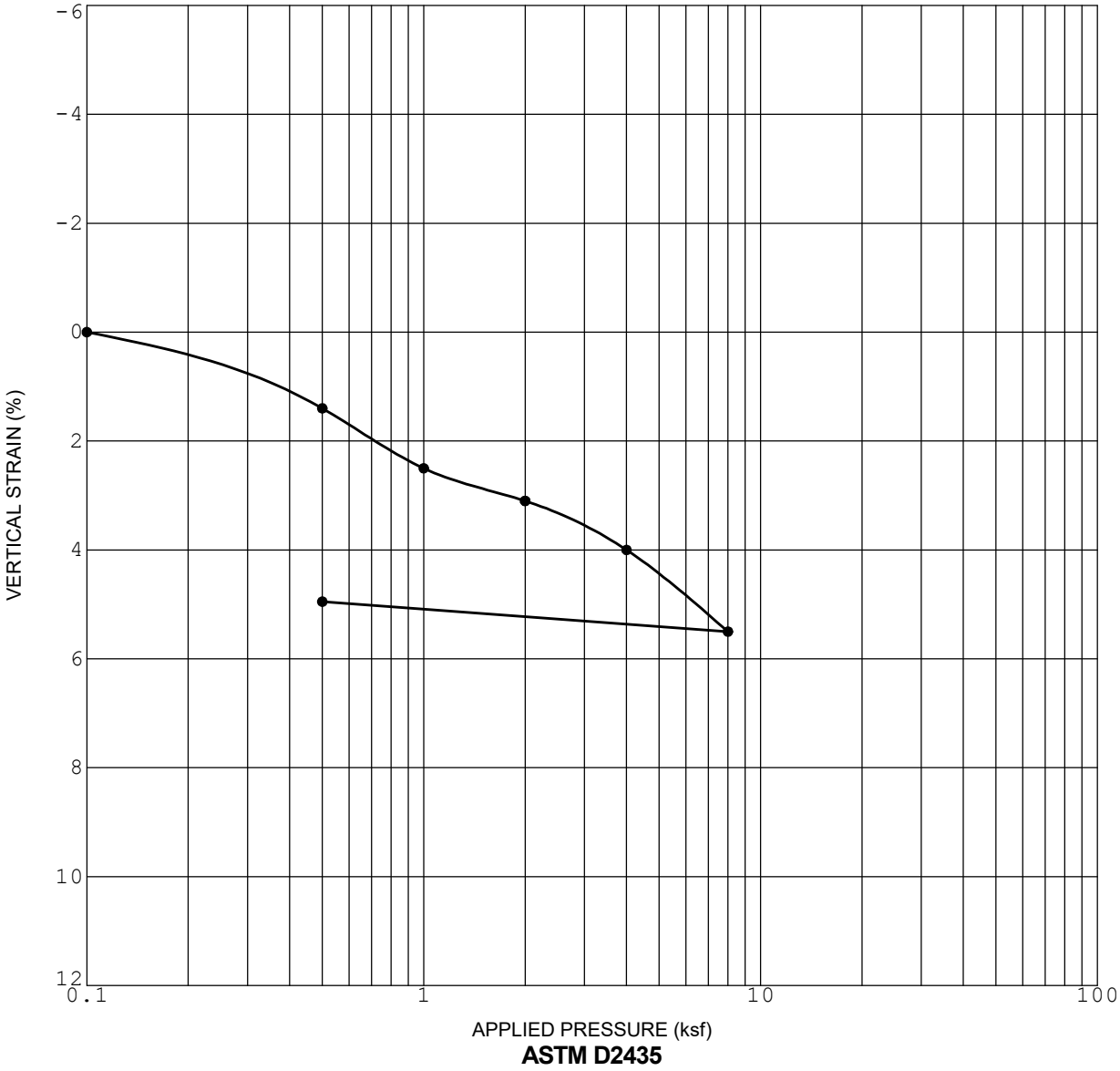
G2129-52-03.GPJ

Figure B-2

GEOCON

PROJECT NO. G2129-52-03

SAMPLE NO. B2-2 @5'



ASTM D2435

Initial Dry Density (pcf)	112.6	**Initial Saturation (%)**	100+
Initial Water Content (%)	20.1	**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

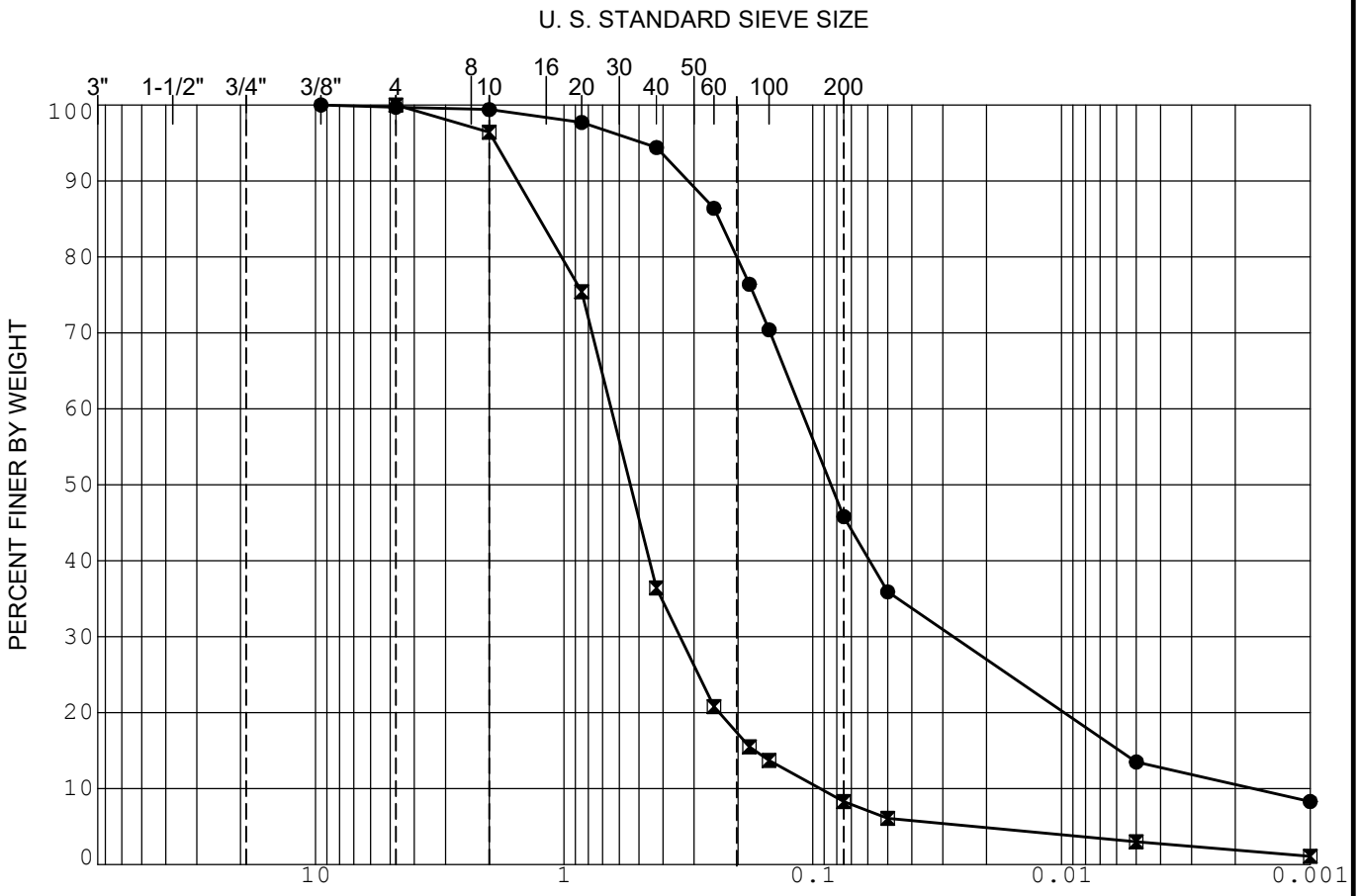
G2129-52-03.GPJ

Figure B-3

GEOCON

PROJECT NO. G2129-52-03

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



ASTM D422

	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B2-2	5.0	SM - Silty SAND				
⊠	B2-6	20.0	SW-SM - Well-graded SAND to Silty SAND				
▲							

GRADATION CURVE

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

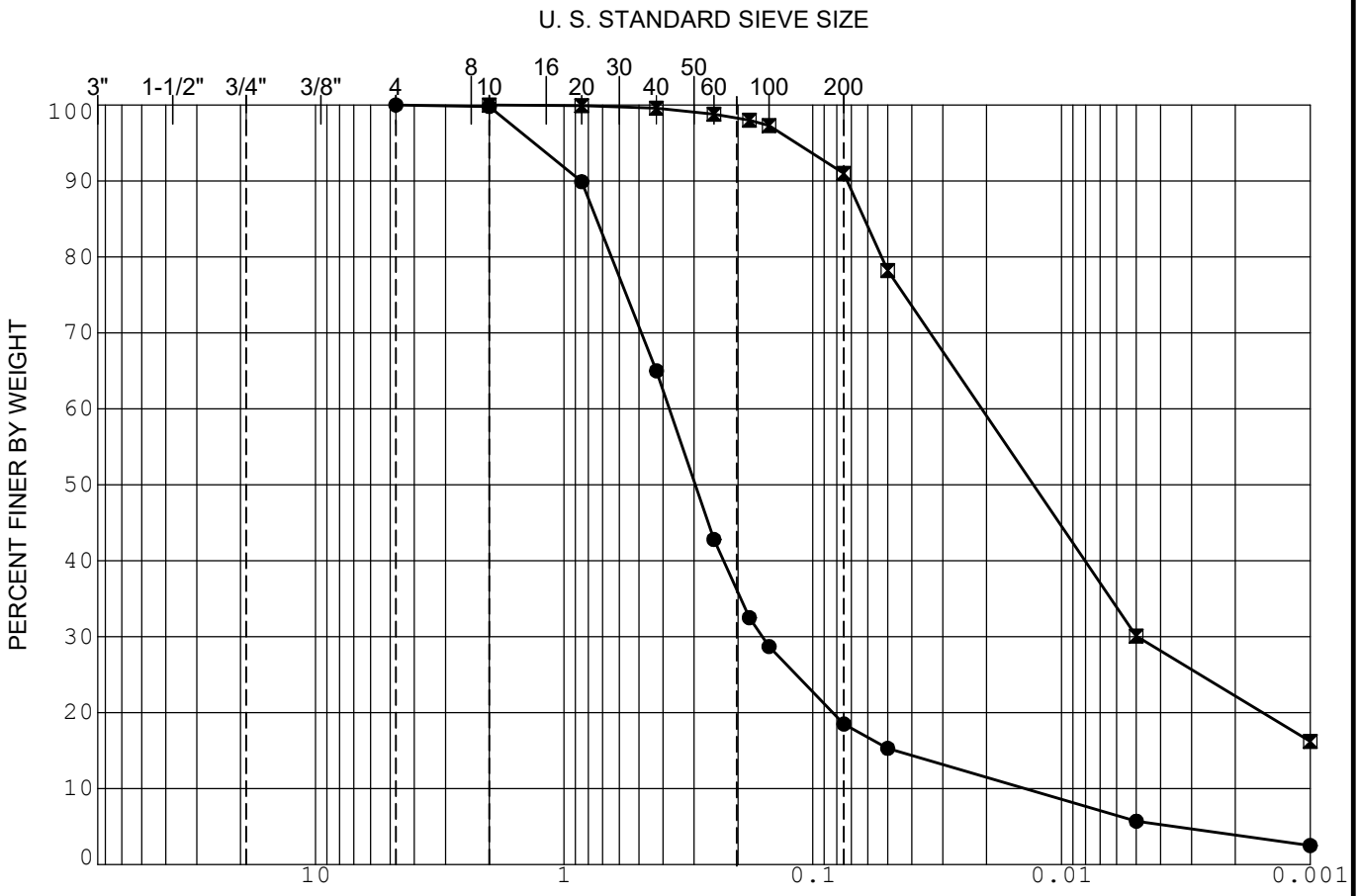
G2129-52-03.GPJ

Figure B-4

GEOCON

PROJECT NO. G2129-52-03

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



ASTM D422

	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B2-9	35.0	SM - Silty SAND				
⊠	B2-11	45.0	CL - Lean CLAY				
▲							

GRADATION CURVE

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

G2129-52-03.GPJ

Figure B-5

GEOCON

APPENDIX C

APPENDIX C

BORING AND TRENCH LOGS, AND LABORATORY TESTING FROM PREVIOUS INVESTIGATION (CHRISTIAN WHEELER)

FOR

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

PROJECT NO. G2129-52-03

LOG OF TEST BORING NUMBER B-1

Date Excavated: 10/3/2005
Equipment: CME 55
Existing Elevation: 13.0 feet
Finish Elevation:

Logged by: AKN
Project Manager: CHC
Depth to Water: 13 feet
Drive Weight: 140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		2" layer of A.C.						
2		**Artificial Fill (Qaf):** Light to medium gray, damp to moist, loose, SILTY SAND (SM), with minor gravels and trace glas and A.C. debris.	Cal		8	2.5	97.8	
4		**Alluvium (Qal):** Light to medium grayish-brown, moist, loose, SILTY SAND (SM), with minor gravels.	SPT*		4			MD, DS, SA, SO_4
6			Cal		5	2.5	98.9	
8		Light to medium grayish-brown, moist, medium dense, POORLY GRADED SAND-SILTY SAND (SP-SM), coarse to very coarse-grained, with gravels and cobbles.	SPT		24			
10								
12	▽	At 11 feet becomes very moist.	Cal		28	11.9	97.3	
14		At 13 feet becomes saturated.	SPT		43			SA
16			Cal*		50/5½"			
18								
20			SPT		55			

Boring continued on Plate No. 3.

* No sample recovery.

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	2

LOG OF TEST BORING NUMBER B-1 (Continued)

Date Excavated:	10/3/2005	Logged by:	AKN
Equipment:	CME 55	Project Manager:	CHC
Existing Elevation:	13.0 feet	Depth to Water:	13 feet
Finish Elevation:		Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
22		Alluvium (Qal): Light to medium grayish-brown, saturated, medium dense, POORLY GRADED SAND-SILTY SAND (SP-SM), coarse to very coarse-grained, with gravels and cobbles.	Cal*		50/5"			
24		Practical refusal at 27 feet on abundant cobbles.						
26								
28								
30								
32								
34								
36								
38								
40								

* No sample recovery.

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	3

LOG OF TEST BORING NUMBER B-2

Date Excavated:	10/3/2005	Logged by:	AKN
Equipment:	CME 55	Project Manager:	CHC
Existing Elevation:	12.0 feet	Depth to Water:	12 feet
Finish Elevation:		Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
2		**Residual Soil:** Light to medium brown, damp, loose, SILTY SAND (SM), with minor gravels.						
4		**Alluvium (Qal):** Light to medium grayish-brown, damp to moist, medium dense, POORLY GRADED SAND (SP), medium-grained, poorly sorted, with minor gravels.	Cal		30	1.0	99.4	SA
6		At 4½ feet becomes moist, well sorted.	Cal		19			
8		Medium grayish-brown, moist, medium dense, SILTY SAND-CLAYEY SAND (SM-SC), mottled.						
10			Cal		28			
12	▽	Light to medium grayish-brown, very moist, medium dense, POORLY GRADED SAND (SP), with gravels and cobbles. At 12 feet becomes saturated.						
14			SPT		13			
16			Cal		19			
18								
20			SPT		50/5"			SA

Boring continued on Plate No. 5.

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	4

LOG OF TEST BORING NUMBER B-2 (Continued)

Date Excavated:	10/3/2005	Logged by:	AKN
Equipment:	CME 55	Project Manager:	CHC
Existing Elevation:	12.0 feet	Depth to Water:	12 feet
Finish Elevation:		Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
22		**Alluvium (Qal):** Light to medium grayish-brown, saturated, medium dense, POORLY GRADED SAND (SP), with gravels and cobbles.	Cal*		50/5"			
24								
26			SPT		50/5½"			
28		Practical refusal at 27 feet on abundant cobbles.						
30								
32								
34								
36								
38								
40								

* No sample recovery.

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	5

LOG OF TEST BORING NUMBER B-3

Date Excavated:	10/3/2005	Logged by:	AKN
Equipment:	CME 55	Project Manager:	CHC
Existing Elevation:	8.0 feet	Depth to Water:	8 feet
Finish Elevation:		Drive Weight:	140 lbs./30"

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		Residual Soil: Light to medium brown, damp, loose, SILTY SAND (SM), with trace gravels.						
2		**Alluvium (Qal):** Medium grayish-brown, moist, medium dense/stiff, SILTY SAND-SANDY SILT (SM-ML), fine-grained, with white mineral precipitate deposits and trace clay.	Cal		21	11.1	117.1	MD, DS, SA, SO_4
4			Cal		27	11.8	116.4	
6								
8		Becomes saturated at 8 feet.						
10			Cal		6	21.4	110.0	
12								
14		Light to medium brownish-gray, saturated, medium dense, POORLY GRADED SAND-SILTY SAND (SP-SM), very coarse-grained, with gravels and cobbles.	Cal		50/6"			
16								
18								
20			Cal		59			

Boring terminated at 20 feet.

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	6

LOG OF TEST TRENCH NUMBER T-1

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	9.0 feet	Depth to Water:	9 feet
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		4" layer of Residual Soil.	CK			7.8	109.4	
2		**Alluvium (Qal):** Light to medium grayish-brown, damp to moist, loose to medium dense, SILTY SAND (SM), with trace clay. At 2 feet becomes moist.	CK			7.8	100.8	MD, DS, SA, SO_4
6		Medium grayish-brown, moist, stiff to very stiff, CLAYEY SILT-SILTY CLAY (ML-CL).	CK			23.9	93.7	HA, EI
8	▽	Light to medium brown, very moist, loose to medium dense, POORLY GRADED GRAVELLY SAND (SP), coarse-grained, friable. Becomes saturated at 9 feet.						
12		Practical refusal at 11 feet.						
14								
16								
18								
20								

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	7

LOG OF TEST TRENCH NUMBER T-2

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	11.0 feet	Depth to Water:	11 feet
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		4" layer of Residual Soil.	CK			3.0	118.4	
2		**Alluvium (Qal):** Light to medium grayish-brown, damp to moist, loose to medium dense, SILTY SAND (SM), with trace clay.						
4		At 2 feet becomes moist.	CK			15.4	88.2	
6								
8			CK			15.7	98.0	
10	▽	Light to medium brown, very moist, loose to medium dense, POORLY GRADED GRAVELLY SAND (SP), medium-grained, friable.						
12		Practical refusal at 12 feet.						
14								
16								
18								
20								

CHRISTIAN WHEELER
Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	8

LOG OF TEST TRENCH NUMBER T-3

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	12.0 feet	Depth to Water:	12 feet
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
2		**Residual Soil:** Light to medium grayish-brown, damp, loose, SILTY SAND (SM), with minor gravels.	CK			7.5	94.1	
4		**Alluvium (Qal):** Light to medium grayish-brown, moist, loose to medium dense, SILTY SAND (SM), porous.	CK			11.7	95.1	
8			CK			13.1	95.8	
10		Light to medium brown, moist, loose to medium dense, POORLY GRADED GRAVELLY SAND-GRAVELLY SILTY SAND (SP-SM), friable.						
12		At 12 feet becomes saturated.						
14		Practical refusal at 13 feet.						
16								
18								
20								

CHRISTIAN WHEELER
Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	9

LOG OF TEST TRENCH NUMBER T-4

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	11.0 feet	Depth to Water:	11 feet
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
2		**Residual Soil:** Light to medium grayish-brown, damp, loose, SILTY SAND (SM), with minor gravels.	CK			1.1	109.7	
4		**Alluvium (Qal):** Light to medium jgrayish-brown, moist, loose to medium dense, SILTY SAND (SM), porous, with trace clay.	CK			16.9	101.1	
6								
8			CK			11.6	102.5	
10		Medium brown, very moist, loose to medium dense, SILTY SAND-SANDY SILT (SM-ML), very fine-grained, with trace clay. At 11 feet becomes saturated.						
12		Practical refusal at 12 feet.						
14								
16								
18								
20								

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	10

LOG OF TEST TRENCH NUMBER T-5

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	9.0 feet	Depth to Water:	9 feet
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		4" layer of Residual Soil.	CK			3.2	110.9	
2		Alluvium (Qal): Light to medium grayish-brown, moist, loose to medium dense, SILTY SAND (SM), porous, with trace clay.						
4			CK			17.2	103.5	
6								
8		Medium brown, very moist, loose/medium stiff, SILTY SAND-SANDY SILT (SM-ML), very fine-grained, with trace clay. At 9 feet becomes saturated.						
10								
12								
14		Practical refusal at 13 feet.						
16								
18								
20								

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	11

LOG OF TEST TRENCH NUMBER T-6

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	11.0 feet	Depth to Water:	11 feet
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		4" layer of Residual Soil.	CK			3.4	104.5	
2		**Alluvium (Qal):** Light to medium grayish-brown, moist, loose to medium dense, SILTY SAND (SM), porous, with trace clay.	CK			24.7	92.0	
10		Medium brown, very moist, loose/medium stiff, SILTY SAND-SANDY SILT (SM-ML), very fine-grained. At 11 feet becomes saturated.	CK			24.4	102.0	
12		Practical refusal at 12 feet.						
14								
16								
18								
20								

CHRISTIAN WHEELER Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	12

LOG OF TEST TRENCH NUMBER T-7

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	10.0 feet	Depth to Water:	10 feet
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		4" layer of Residual Soil.	CK			2.2	100.9	
2		**Alluvium (Qal):** Light to medium grayish-brown, moist, loose to medium dense, SILTY SAND (SM), porous, with trace clay.						
4			CK			13.0	100.5	
6								
8								
10		At 10 feet becomes medium brown, saturated, loose, with minor gravels and cobbles.						
12		Practical refusal at 11 feet.						
14								
16								
18								
20								

CHRISTIAN WHEELER
Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	13

LOG OF TEST TRENCH NUMBER T-8

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	10.0 feet	Depth to Water:	N/A
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0		**Residual Soil:** Light to medium grayish-brown, damp, loose to medium dense, SILTY SAND (SM).						
2		**Alluvium (Qal):** Light to medium brown, damp to moist, loose to medium dense, SILTY SAND-POORLY GRADED SAND (SM-SP), friable, with minor cobbles throughout.						
4		At 2½ feet becomes moist.						
6								
8		Practical refusal at 8 feet.						
10								
12								
14								
16								
18								
20								

CHRISTIAN WHEELER
Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	14

LOG OF TEST TRENCH NUMBER T-9

Date Excavated:	9/29/2005	Logged by:	AKN
Equipment:	580L with 18" bucket	Project Manager:	CHC
Existing Elevation:	12.0 feet	Depth to Water:	N/A
Finish Elevation:		Drive Weight:	N/A

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		6" layer of Residual Soil.	CK					
2		Alluvium (Qal): Light to medium grayish-brown, damp to moist, loose to medium dense, SILTY SAND (SM), slightly porous, with trace gravels.						
4		Light to medium brown, moist, loose to medium dense, SILTY SAND-POORLY GRADED SAND (SM-SP), with minor cobbles throughout.						
6								
8		Practical refusal at 8 feet.						
10								
12								
14								
16								
18								
20								

CHRISTIAN WHEELER
Engineering

PROPOSED BELLA MAR R.V. PARK
408 Hollister Road, San Diego, California

BY:	HF	DATE:	December 2005
JOB NO.:	2050832	PLATE NO.:	15

LABORATORY TEST RESULTS

PROPOSED BELLA MAR RV PARK
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D1557)

Sample Location	**Boring B-1 @ 3'-8½'**	**Boring B-3 @ 1'-5'**	**Trench T-1 @ 0-4½'**
Sample Description	Grayish-brown, SP-SM	Grayish-brown, ML	Grayish-brown, SM
Maximum Density	116.0 pcf	122.9 pcf	121.8 pcf
Optimum Moisture	10.7 %	11.2 %	11.8 %

DIRECT SHEAR (ASTM D3080)

Sample Location	**Boring B-1 @ 3'-8½'**	**Boring B-3 @ 1'-5'**	**Trench T-1 @ 0-4½'**
Sample Type	Remolded to 90 %	Remolded to 90 %	Remolded to 90 %
Friction Angle	27°	26°	28°
Cohesion	125 psf	100 psf	250 psf

GRAIN SIZE DISTRIBUTION (ASTM D422)

Sample Location	**Boring B-1 @ 3'-8½'**	**Boring B-1 @ 14'-15'**	**Boring B-2 @ 2'-5'**	**Boring B-2 @ 19'-20'**
Sieve Size	*Percent Passing*	*Percent Passing*	*Percent Passing*	*Percent Passing*
2"			100	
1½"	100	100	99	100
1"	99	91	99	83
¾"	98	87	93	77
½"	96	83	85	66
⅜"	94	79	82	58
#4	91	71	75	46
#8	87	62	71	36
#16	82	49	65	29
#30	65	34	53	23
#50	30	19	26	16
#100	14	9	13	10
#200	8	5	8	7

Sample Location	**Boring B-3 @ 1'-5'**	**Trench T-1 @ 0-4½'**	**Trench T-1 @ 4½'-8'**
Sieve Size	*Percent Passing*	*Percent Passing*	*Percent Passing*
#4			
#8	100	100	
#16	99	99	100
#30	97	97	99
#50	92	86	97
#100	73	61	85
#200	54	43	61
0.05 mm	--	--	50
0.005 mm	--	--	18
0.001 mm	--	--	15

LABORATORY TEST RESULTS (Continued)

EXPANSION INDEX (ASTM D4829)

Sample Location	**Trench T-1 @ 4½'-8'**
Initial Moisture	10.3 %
Initial Dry Density	97.1 pcf
Final Moisture	29.0 %
Expansion Index	42 (low)

SOLUBLE SULFATE CONTENT (CALTEST 417)

Sample Location	**Boring B-1 @ 3'-8½'**	**Boring B-3 @ 1'-5'**	**Trench T-1 @ 0-4½'**
Sulfate Content	0.005 %	0.024 %	0.050 %

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APPENDIX D

APPENDIX D

LIQUEFACTION ANALYSIS

FOR

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

PROJECT NO. G2129-52-03

Geocon Incorporated
6960 Flanders Drive
San Diego, California 92121-2974
www.geoconinc.com

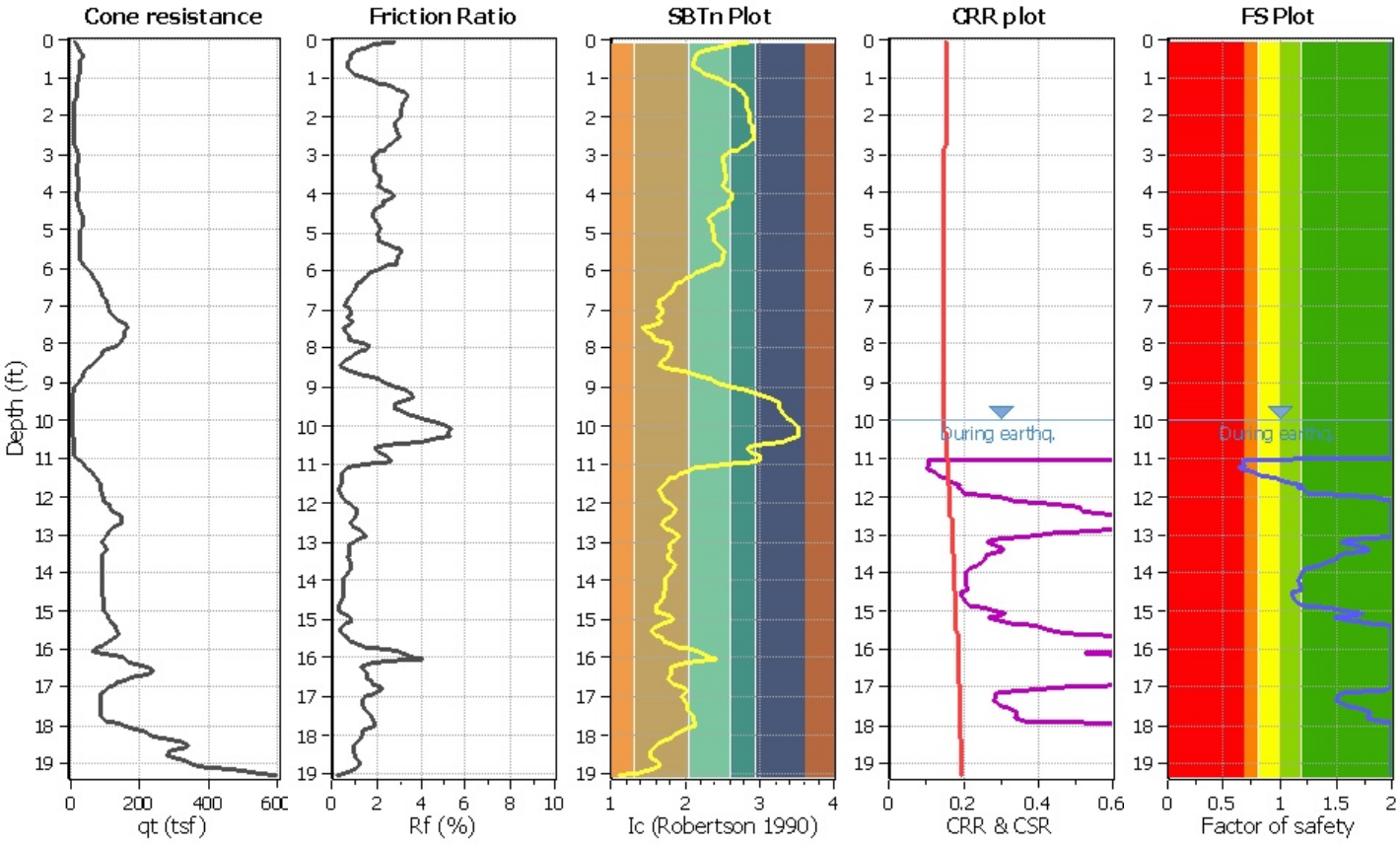
LIQUEFACTION ANALYSIS REPORT

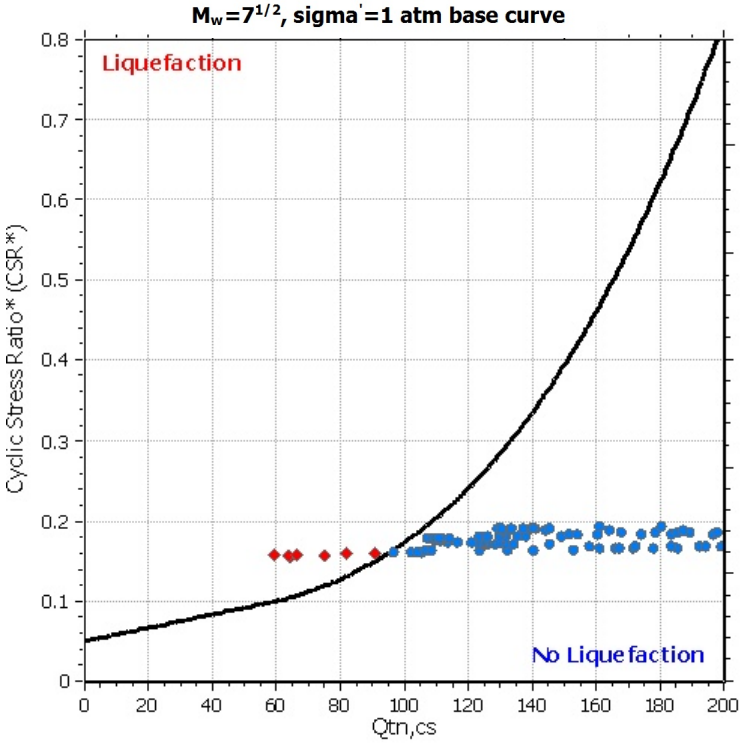
Project title : Bella Mar **Location : 408 Hollister Street**

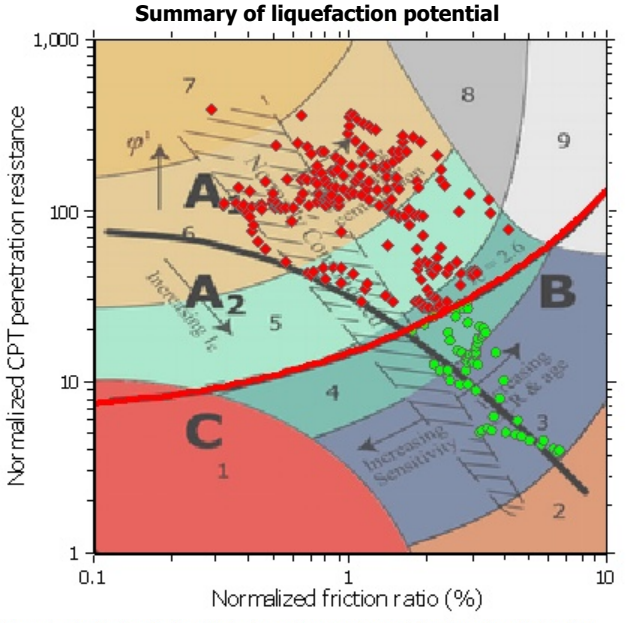
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

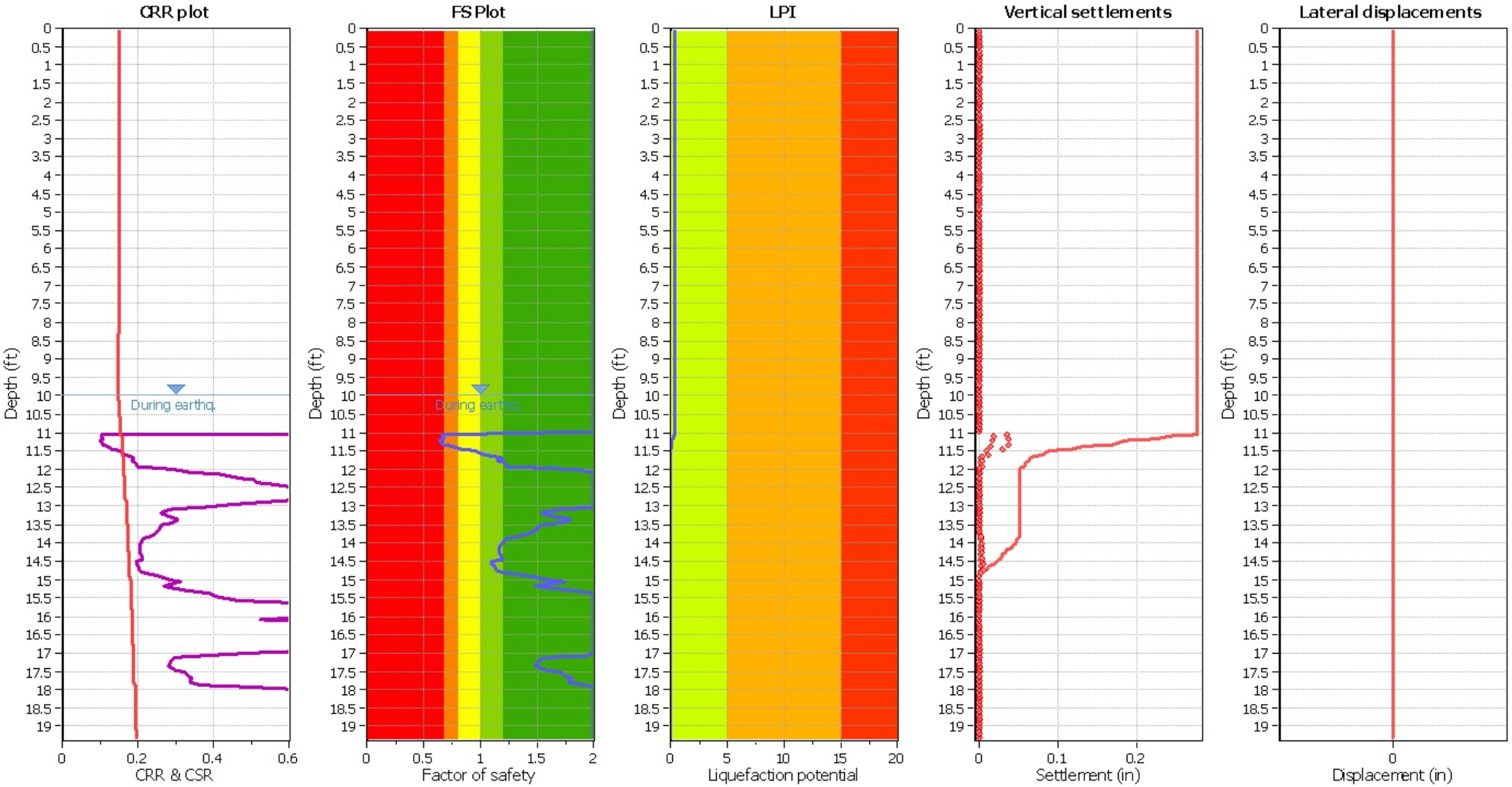






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

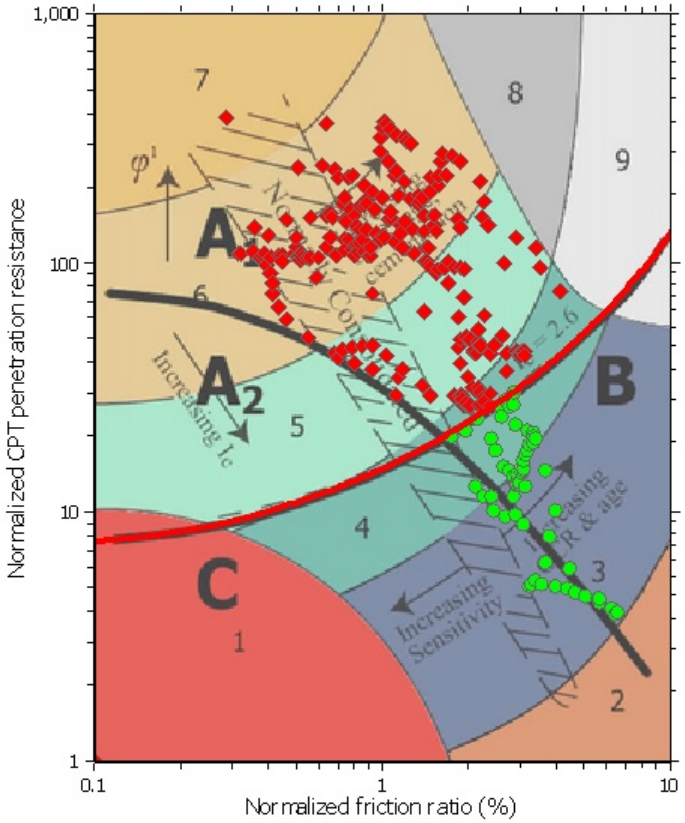
F.S. color scheme

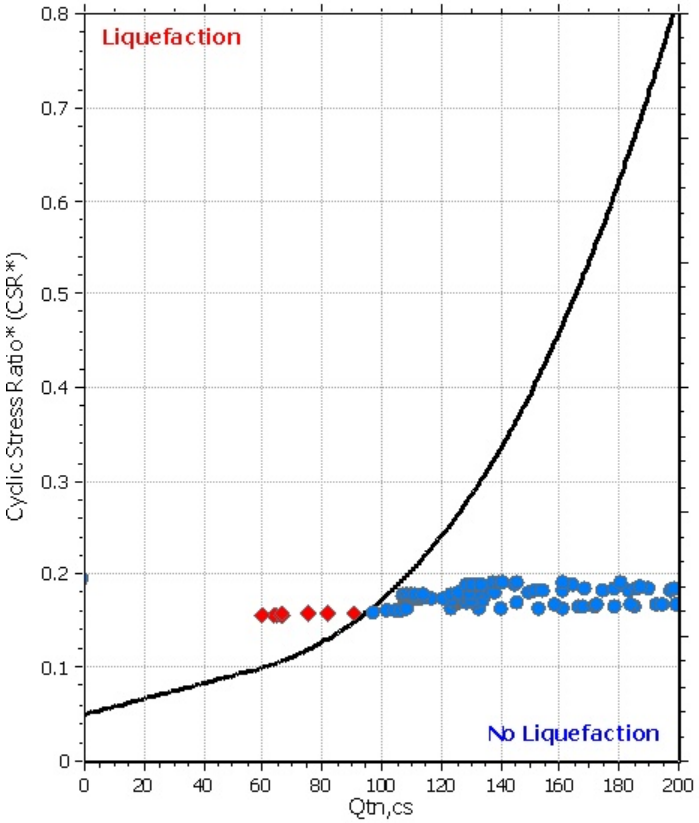
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

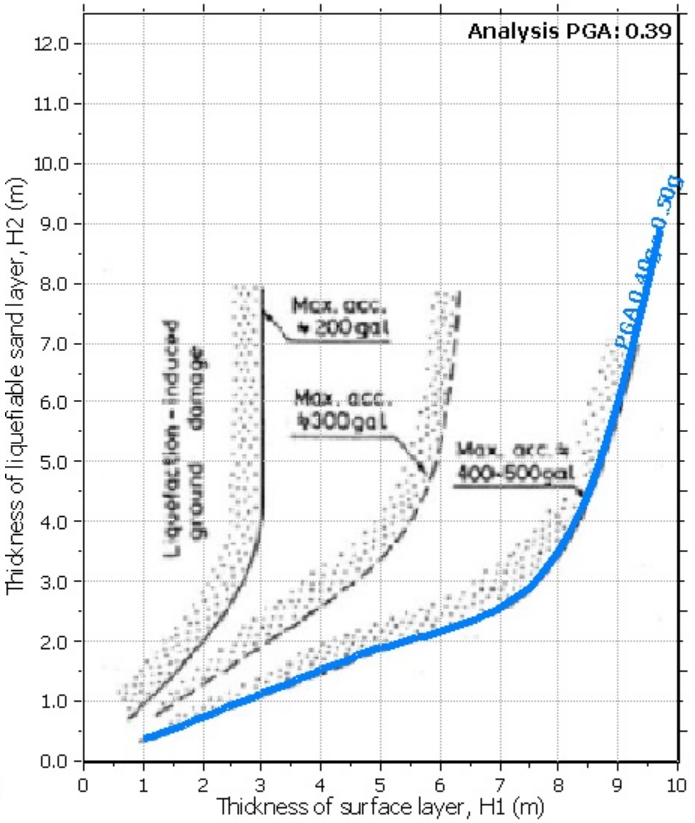
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo







Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Geocon Incorporated
6960 Flanders Drive
San Diego, California 92121-2974
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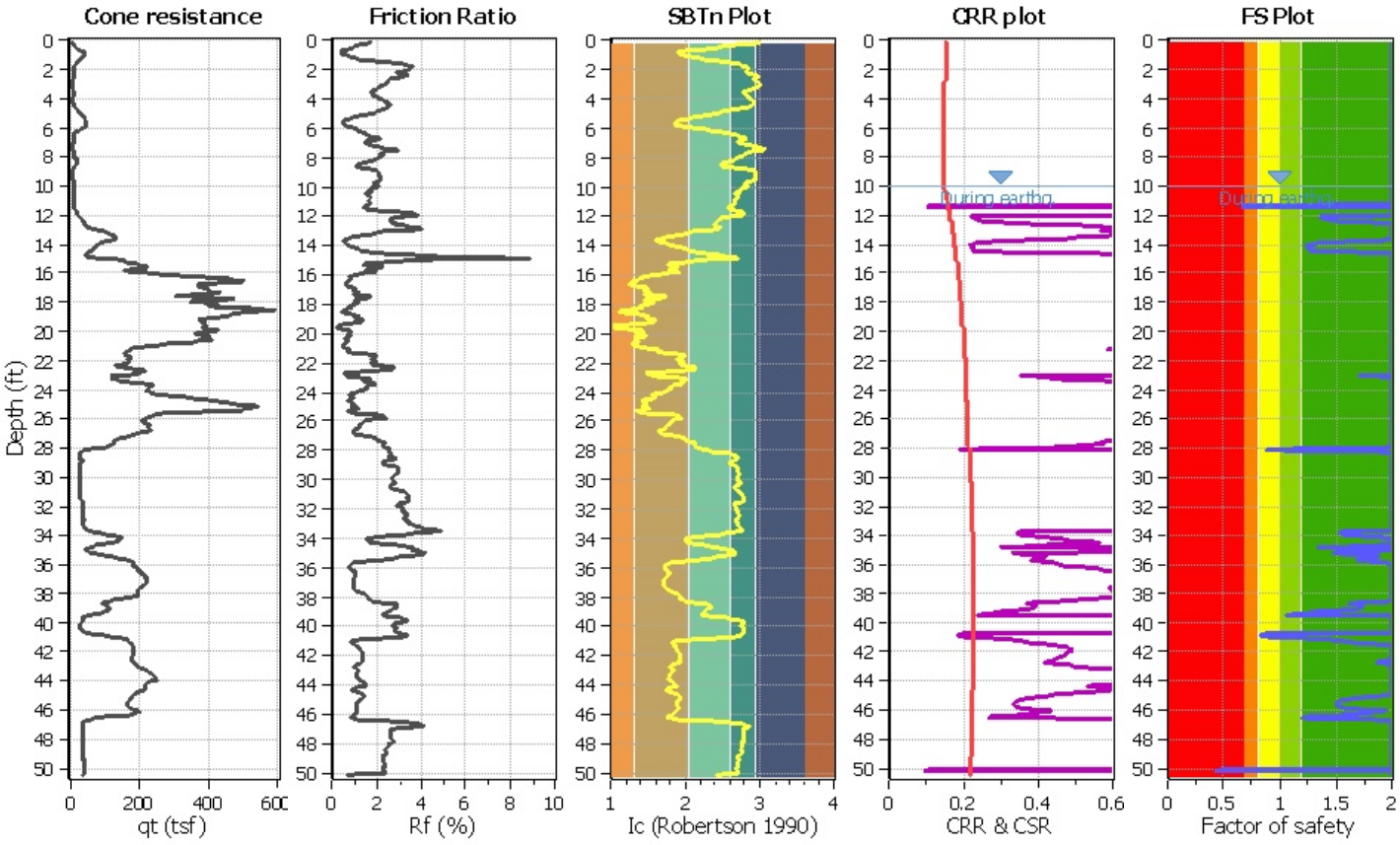
LIQUEFACTION ANALYSIS REPORT

Project title : Bella Mar **Location : 408 Hollister Street**

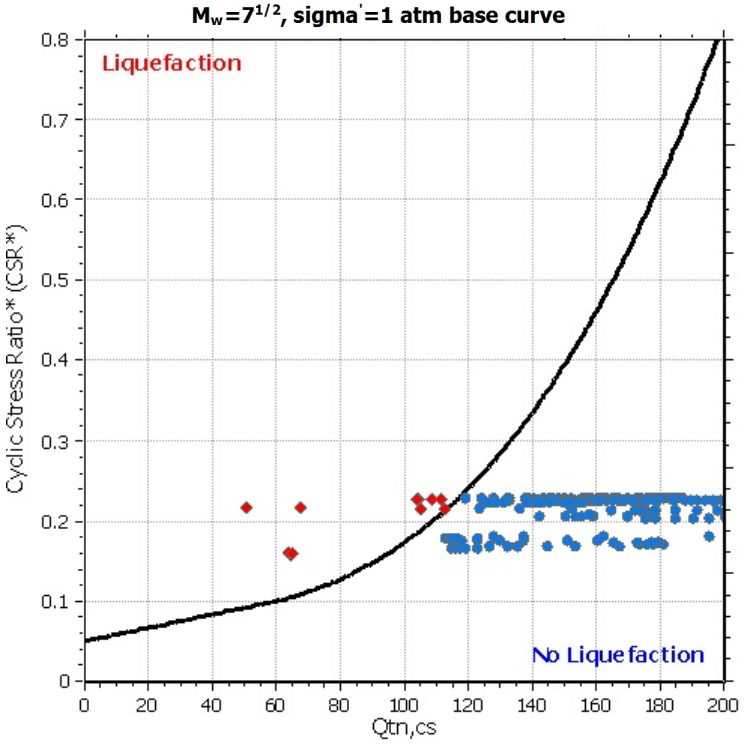
CPT file : CPT-2

Input parameters and analysis data

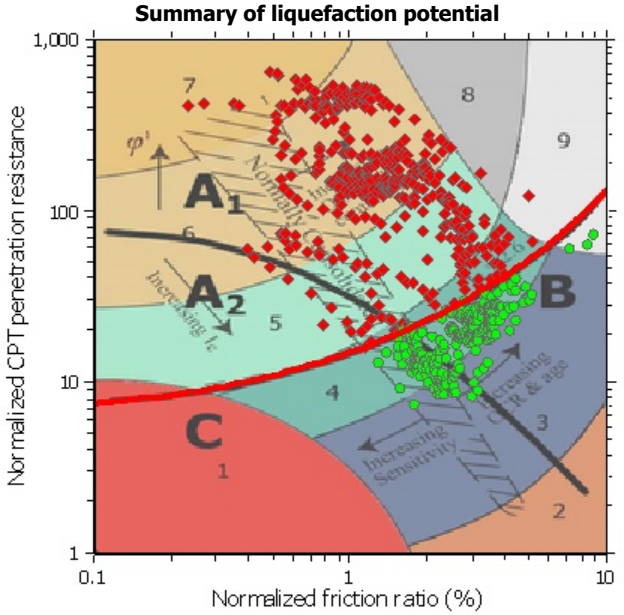
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

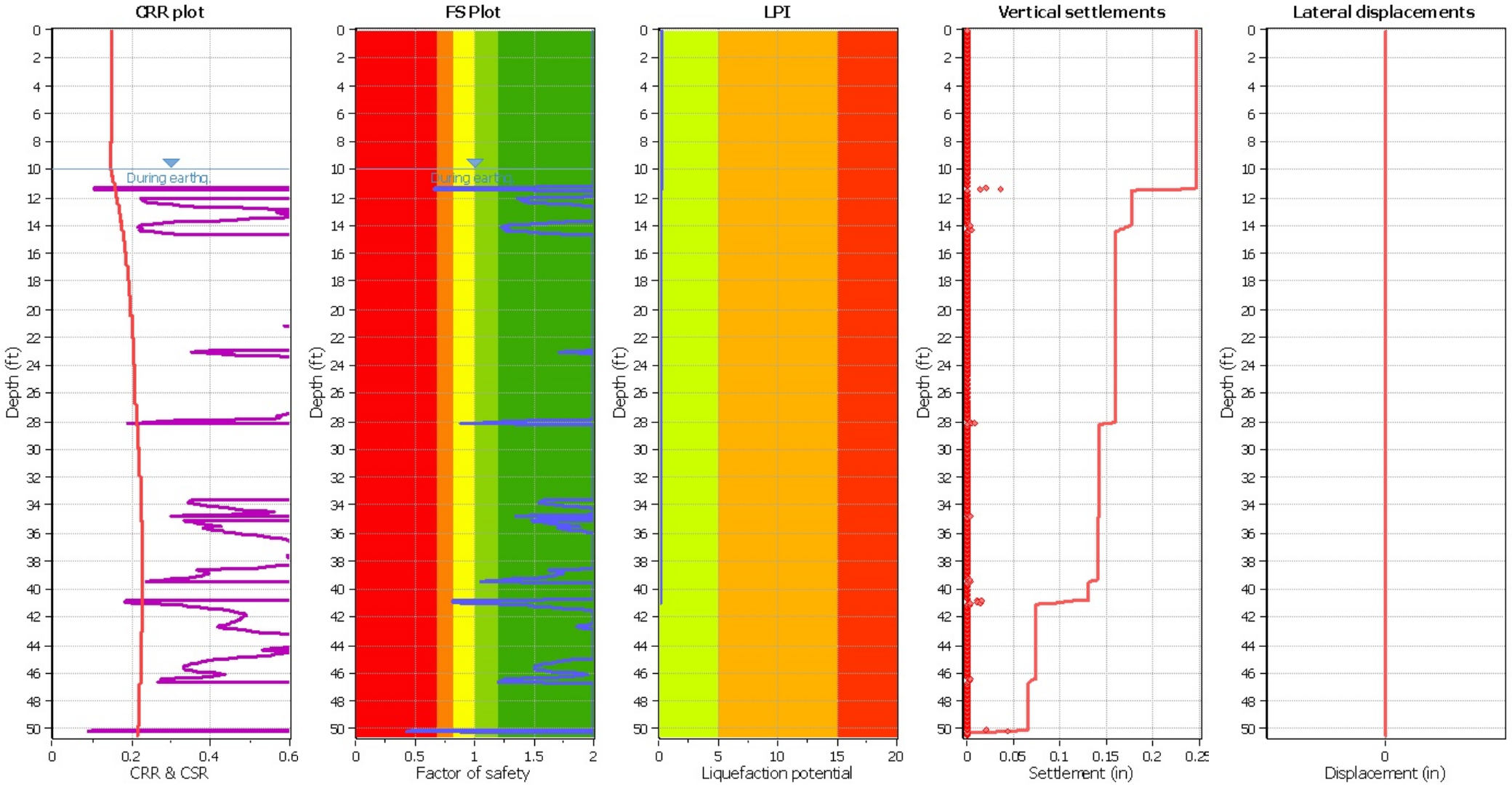


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

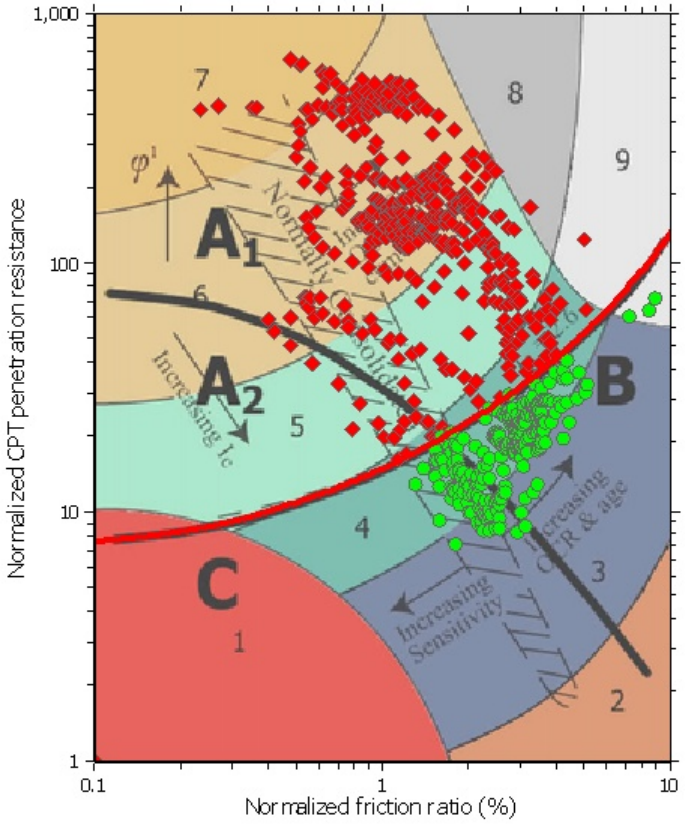
F.S. color scheme

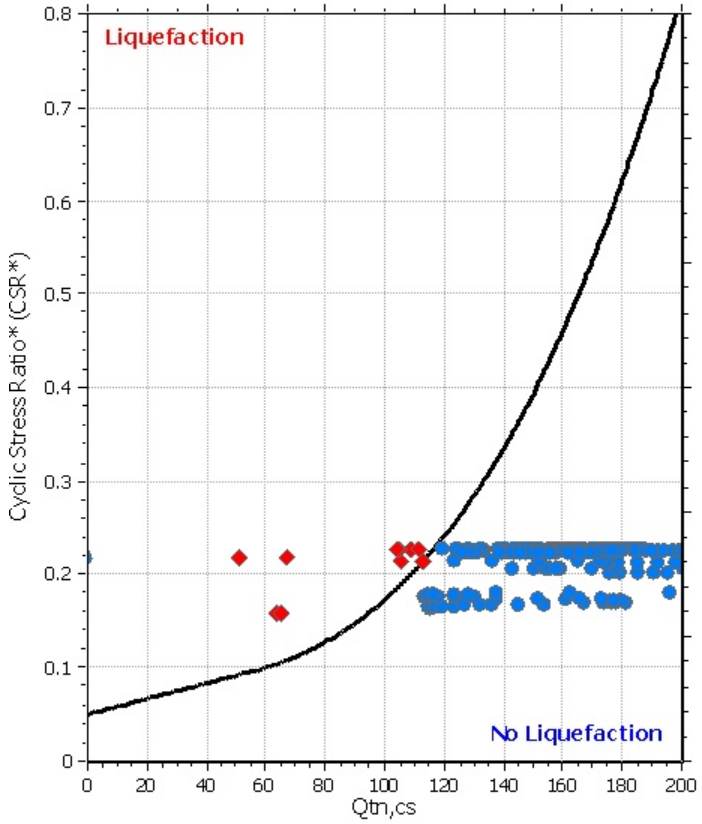
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

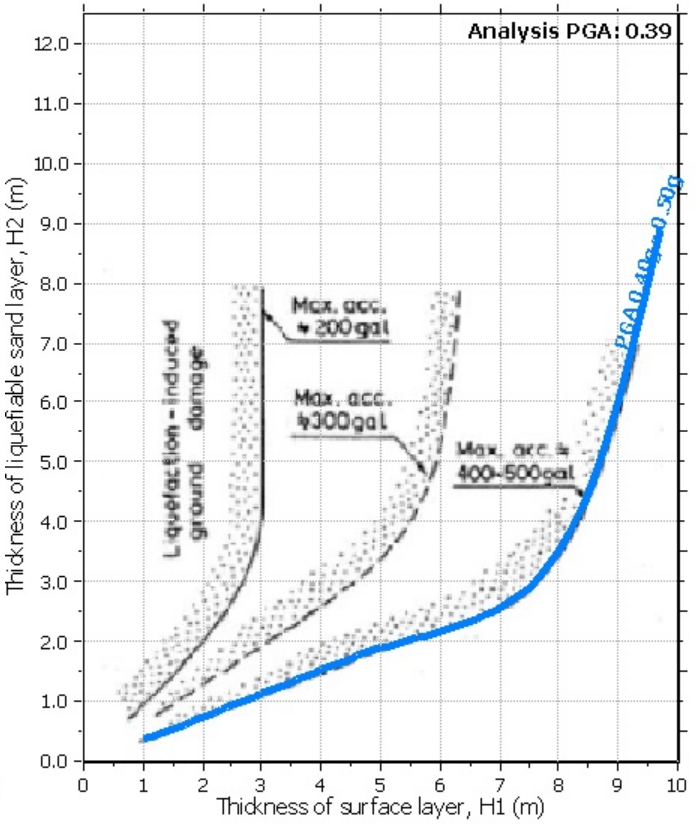
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo







Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Geocon Incorporated
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San Diego, California 92121-2974
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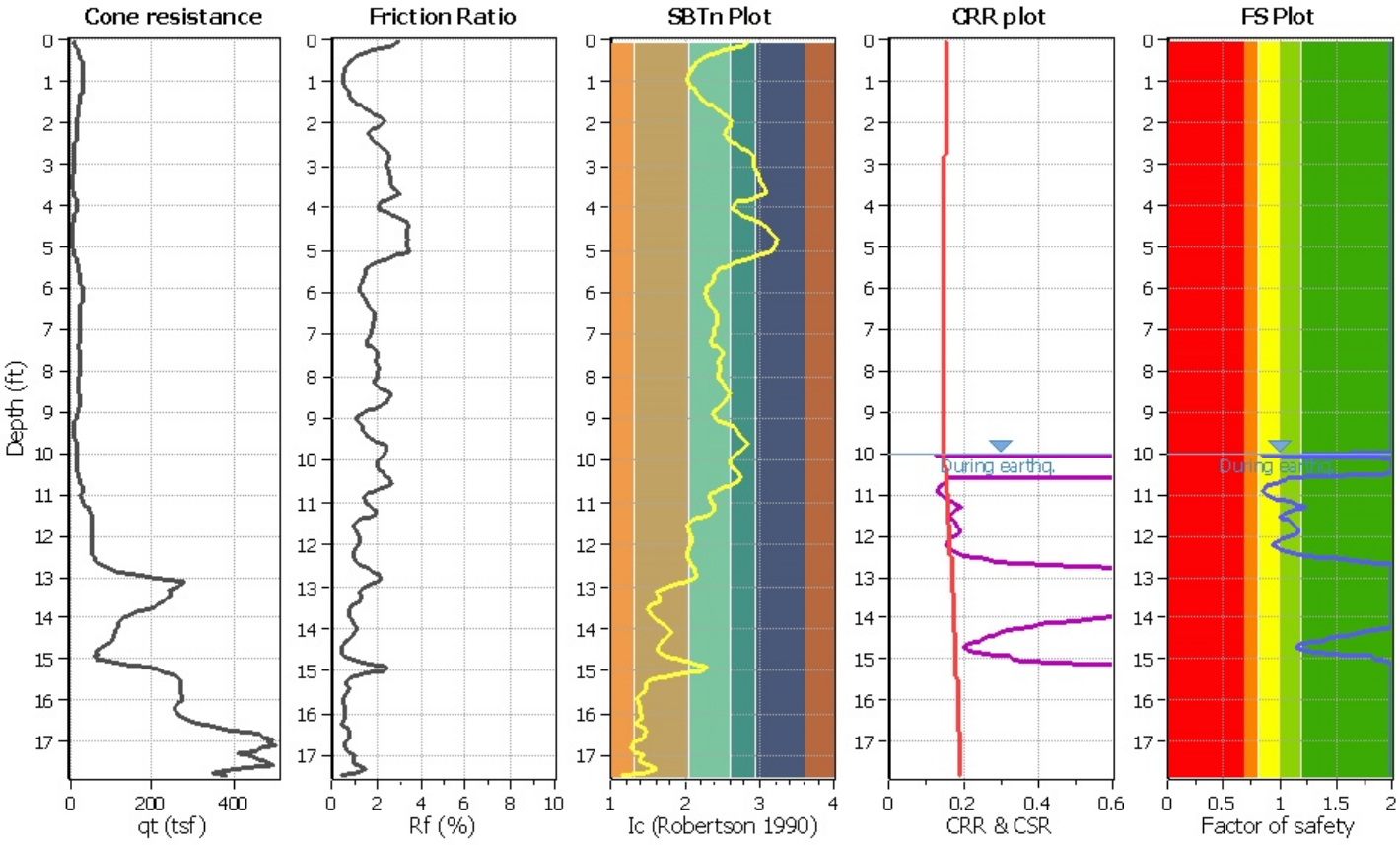
LIQUEFACTION ANALYSIS REPORT

Project title : Bella Mar **Location : 408 Hollister Street**

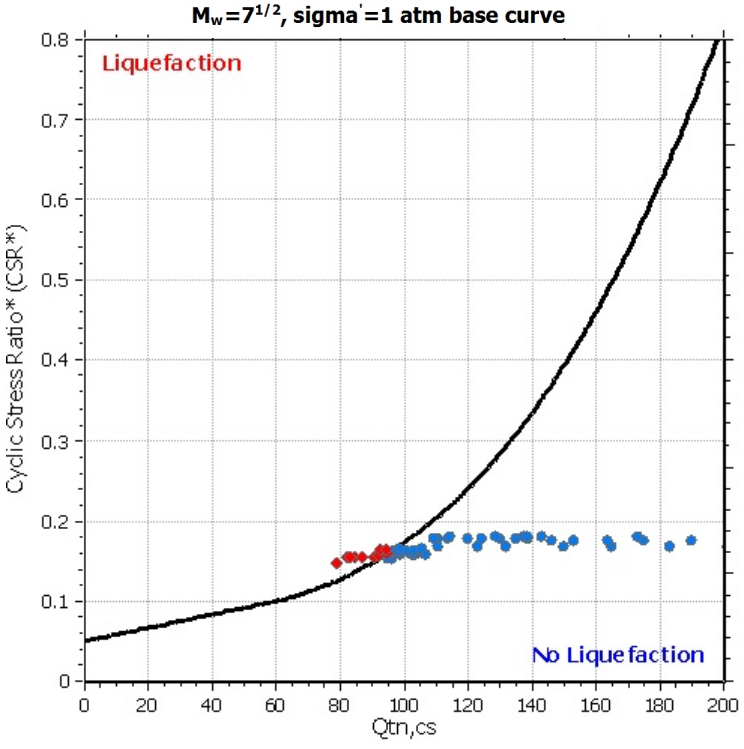
CPT file : CPT-3

Input parameters and analysis data

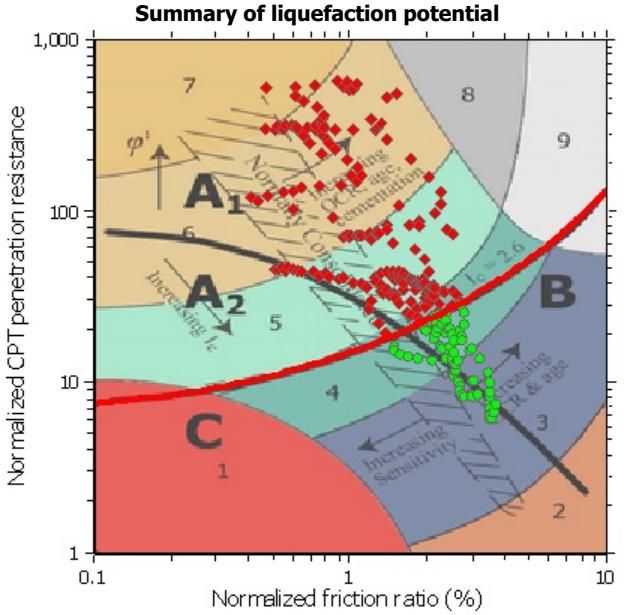
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

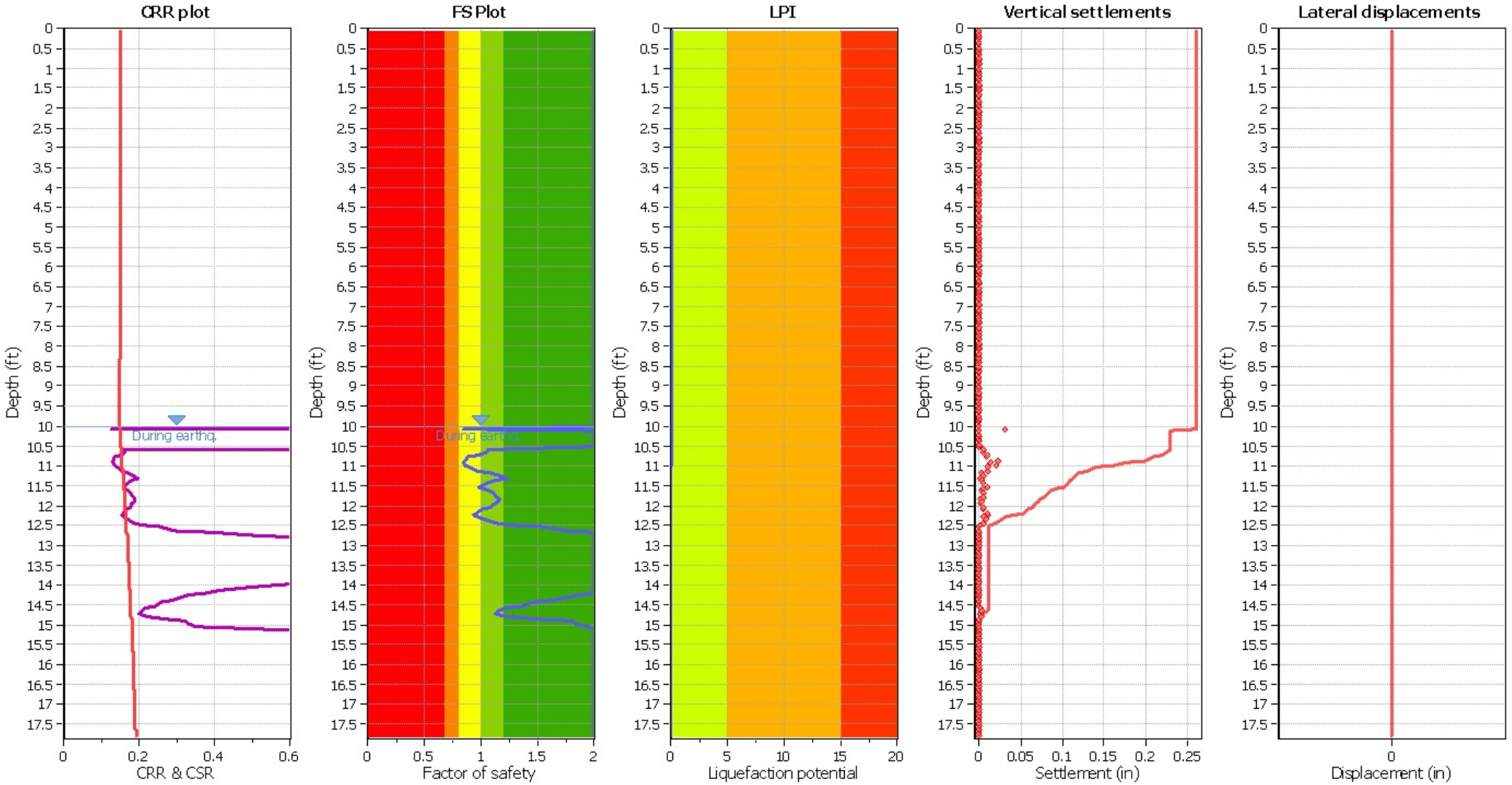


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

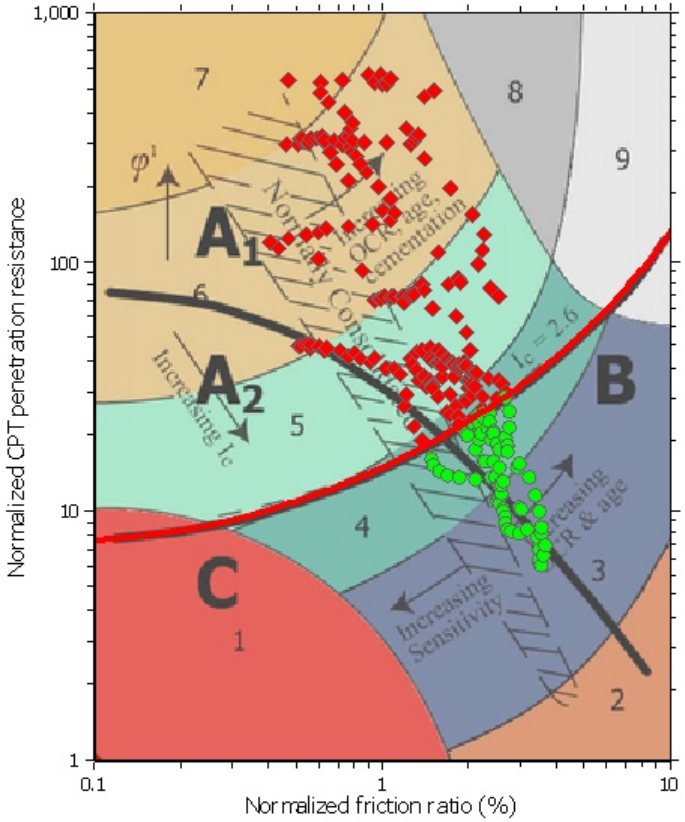
F.S. color scheme

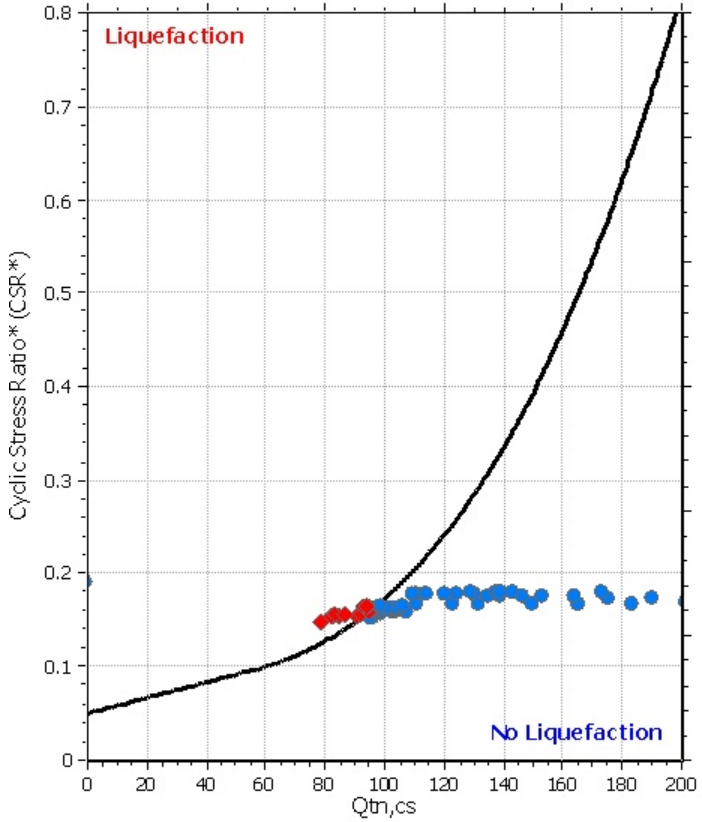
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

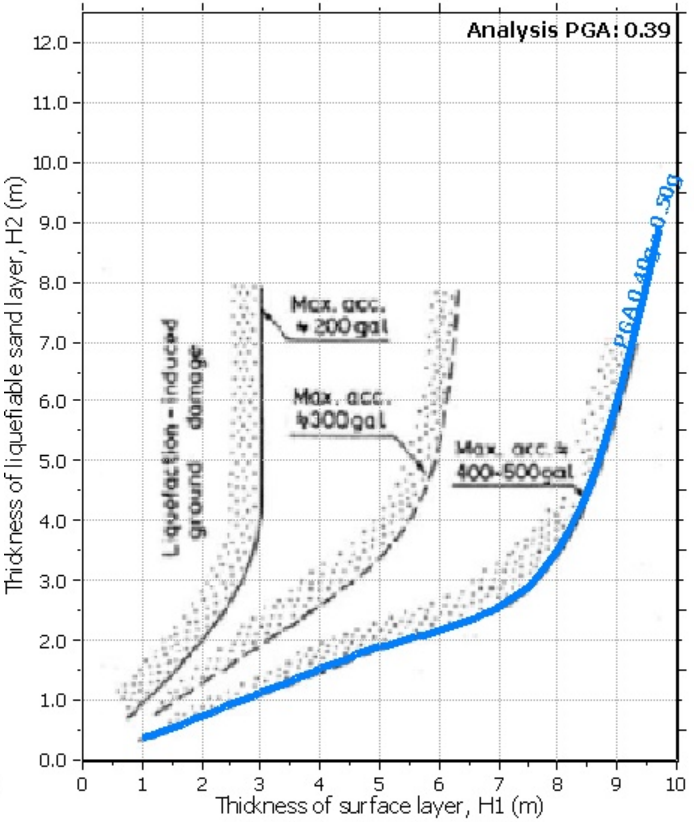
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo







Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

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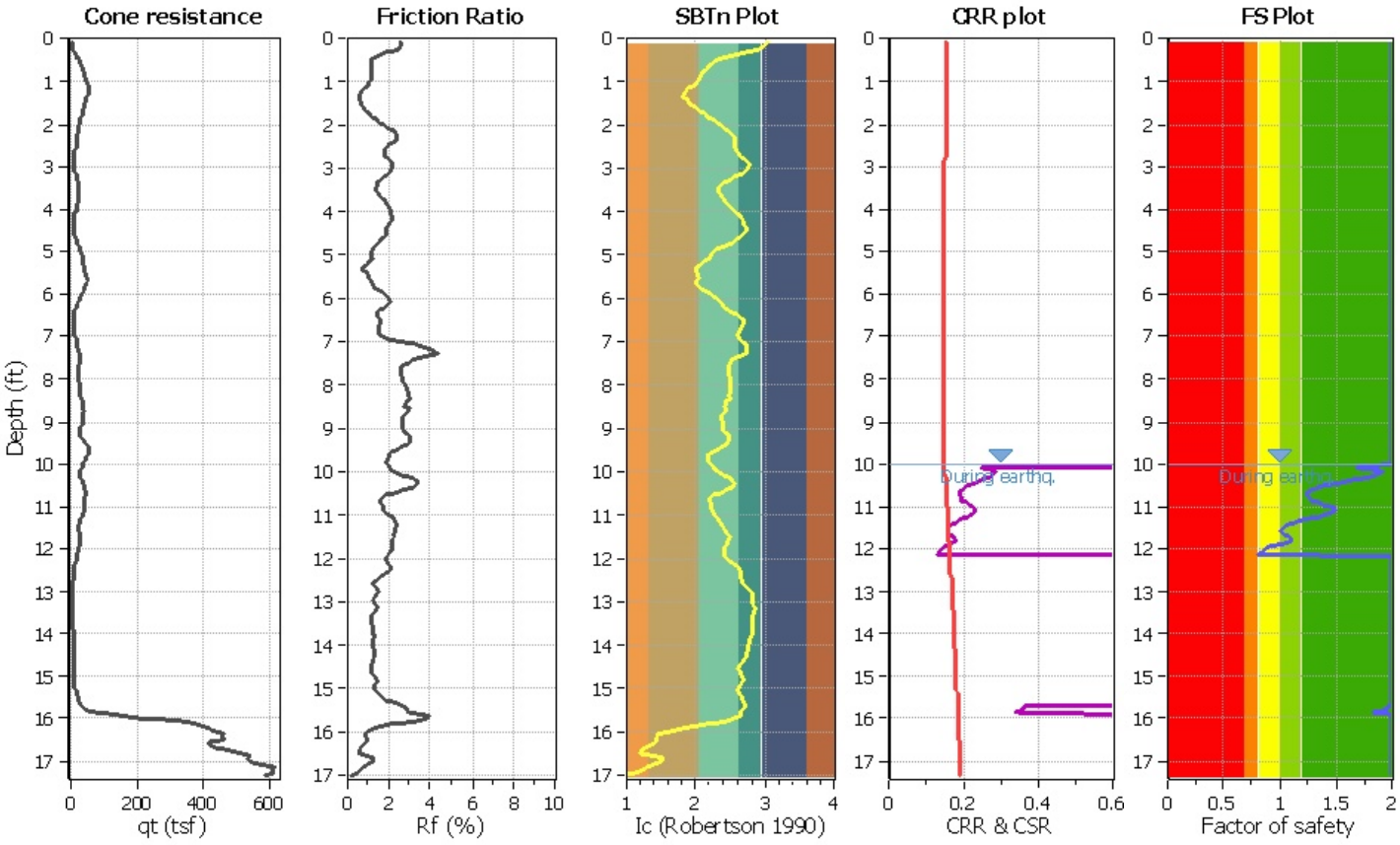
LIQUEFACTION ANALYSIS REPORT

Project title : Bella Mar **Location : 408 Hollister Street**

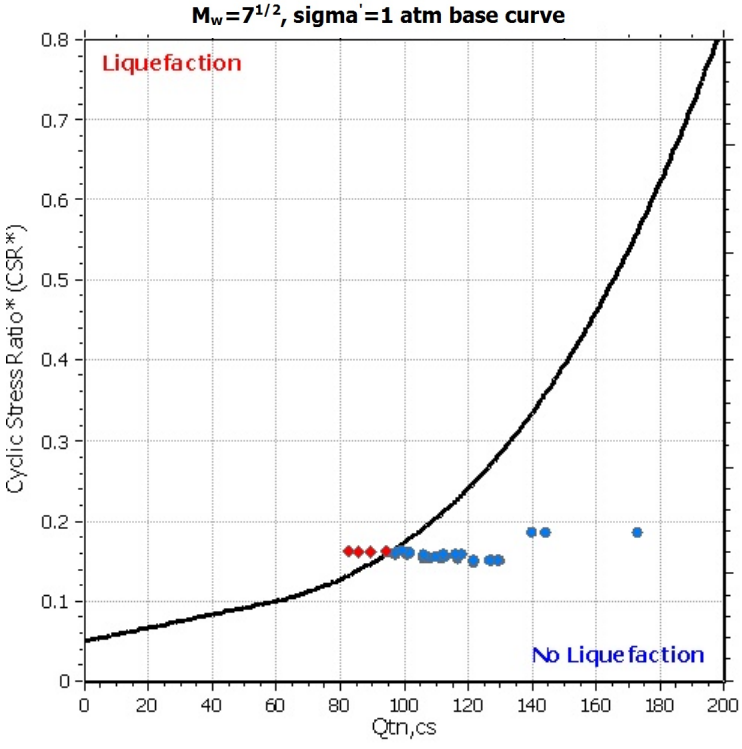
CPT file : CPT-4

Input parameters and analysis data

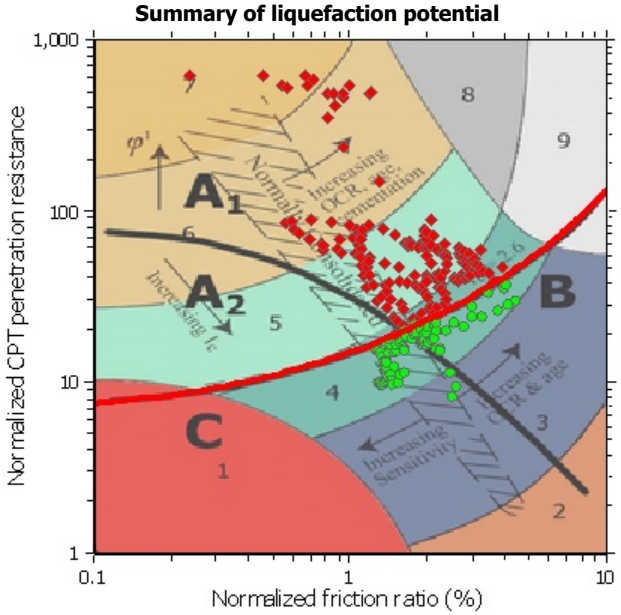
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

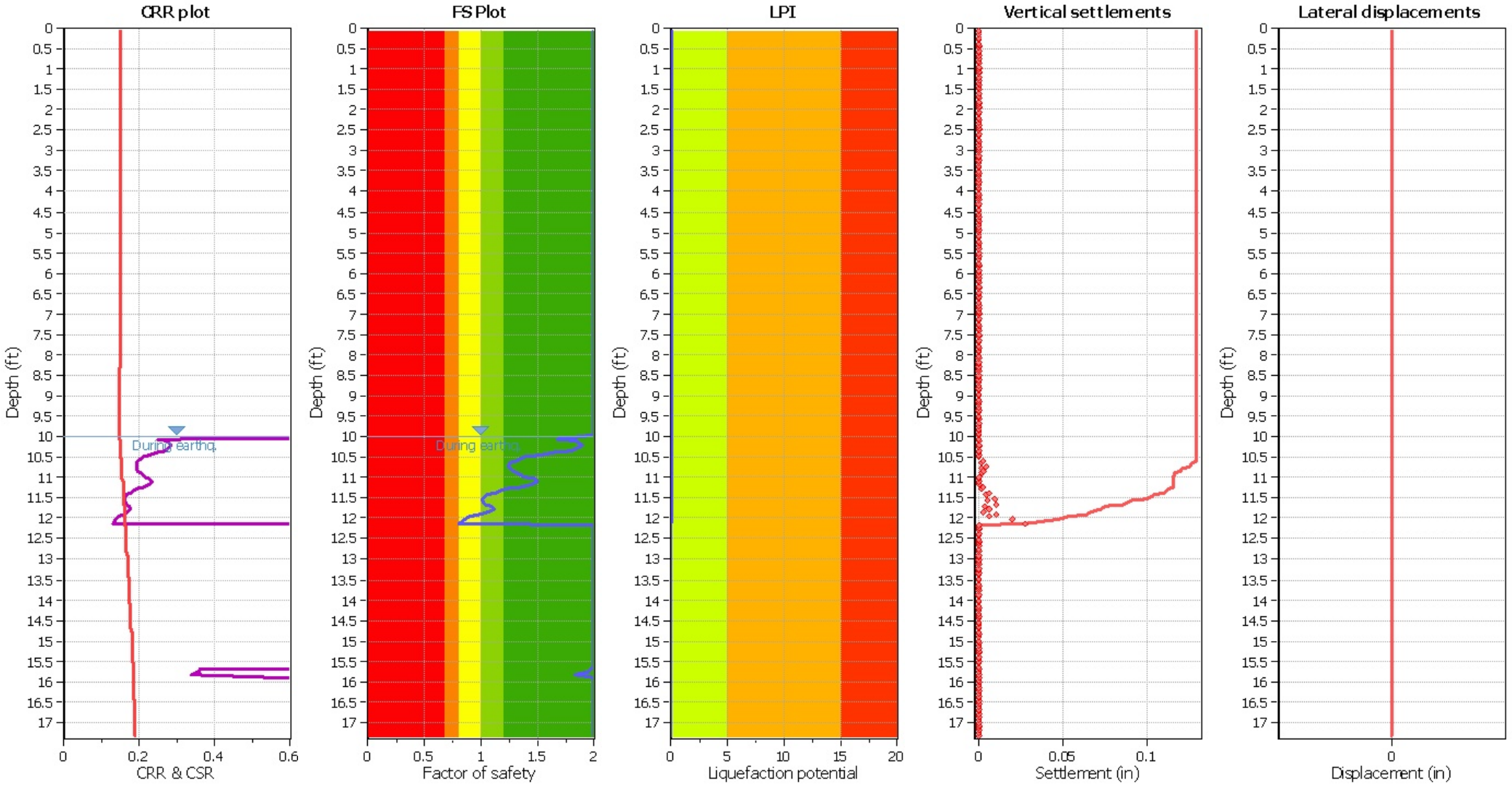


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

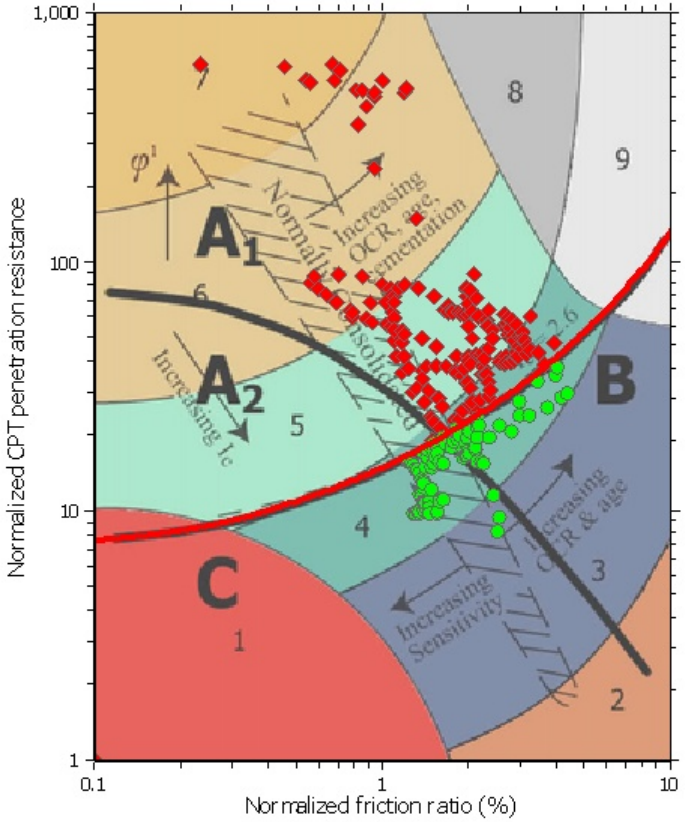
F.S. color scheme

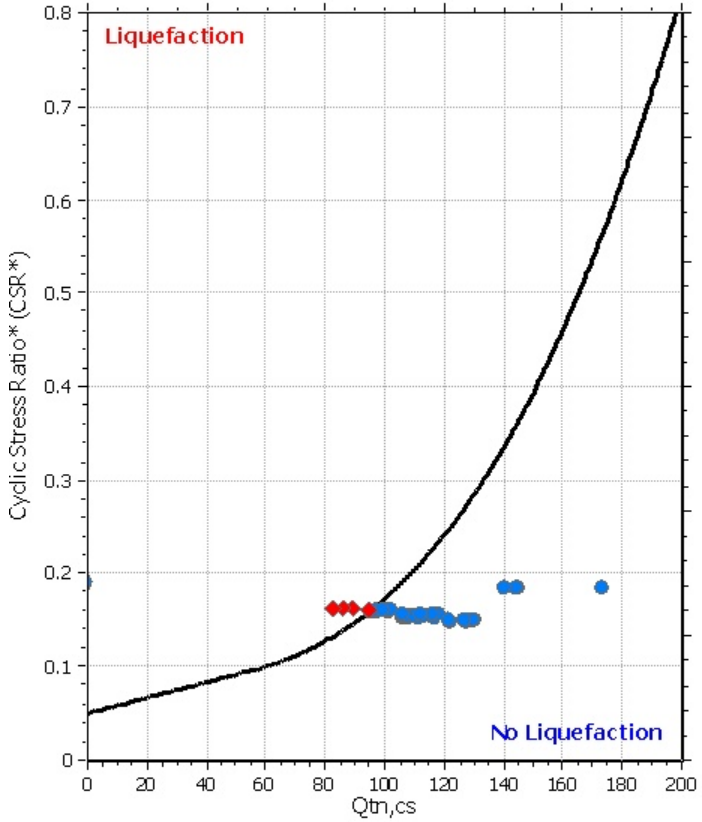
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

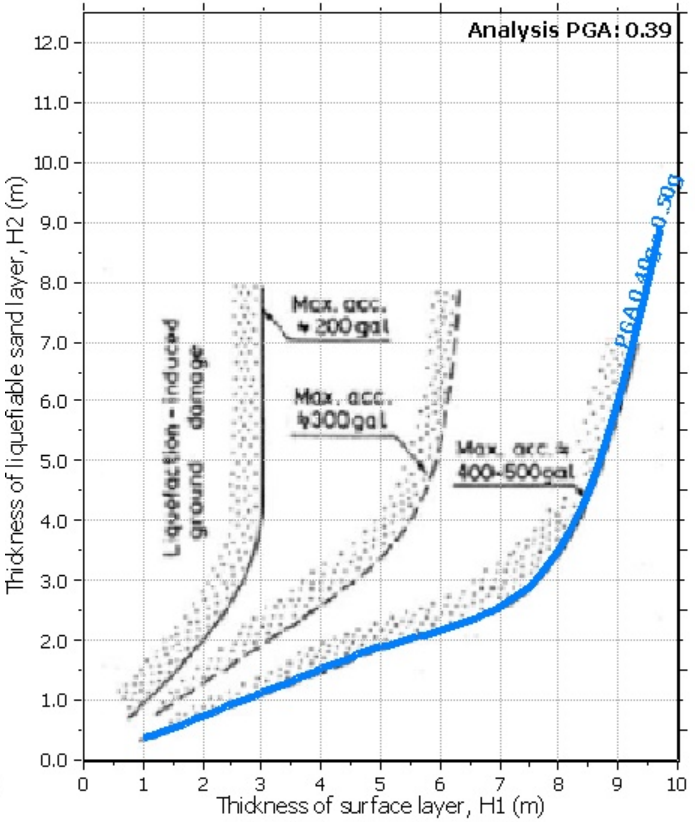
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo







Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

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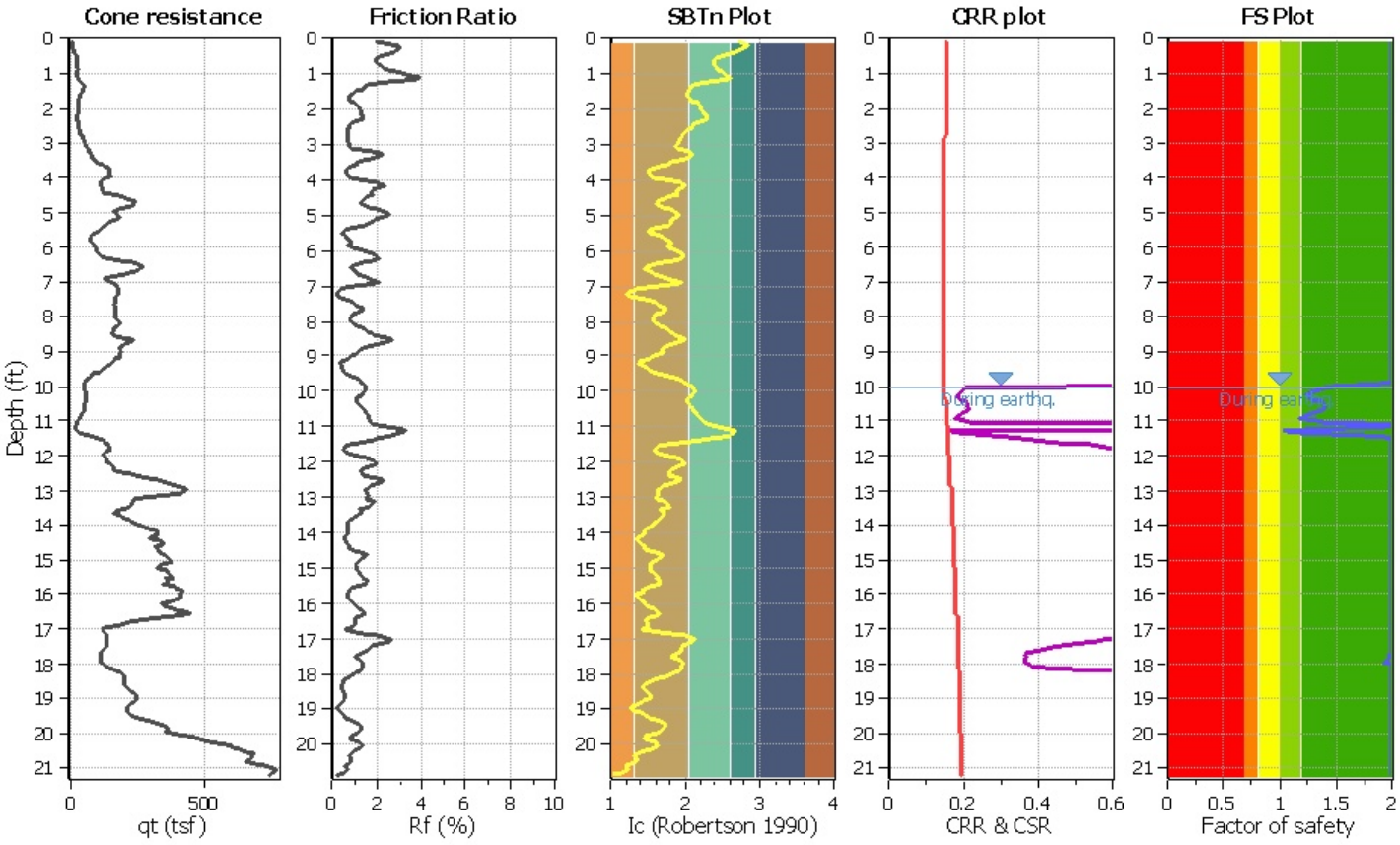
LIQUEFACTION ANALYSIS REPORT

Project title : Bella Mar **Location : 408 Hollister Street**

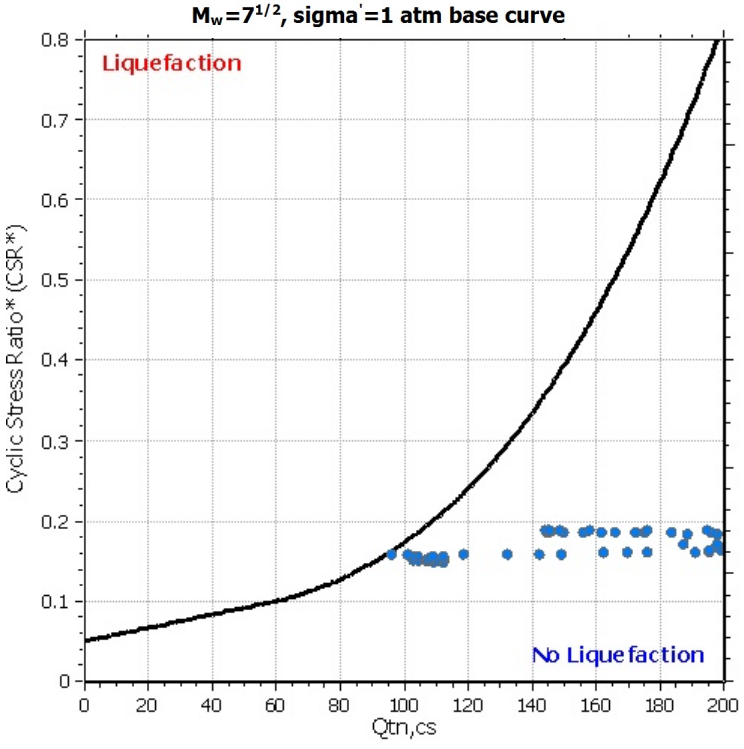
CPT file : CPT-5

Input parameters and analysis data

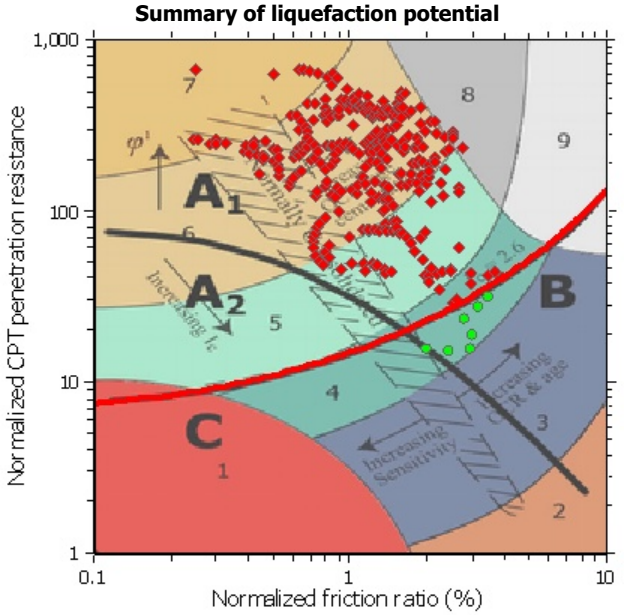
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

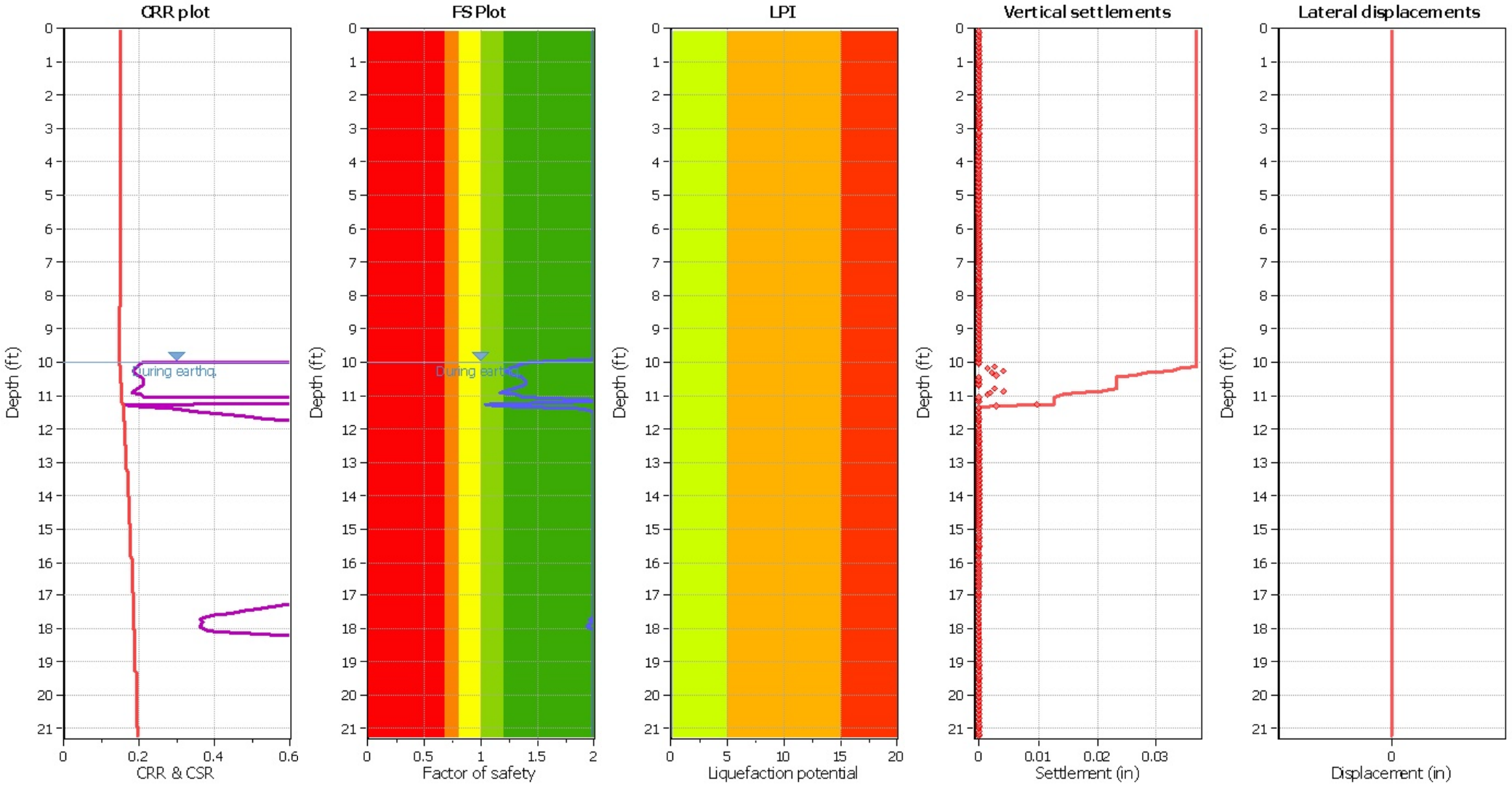


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

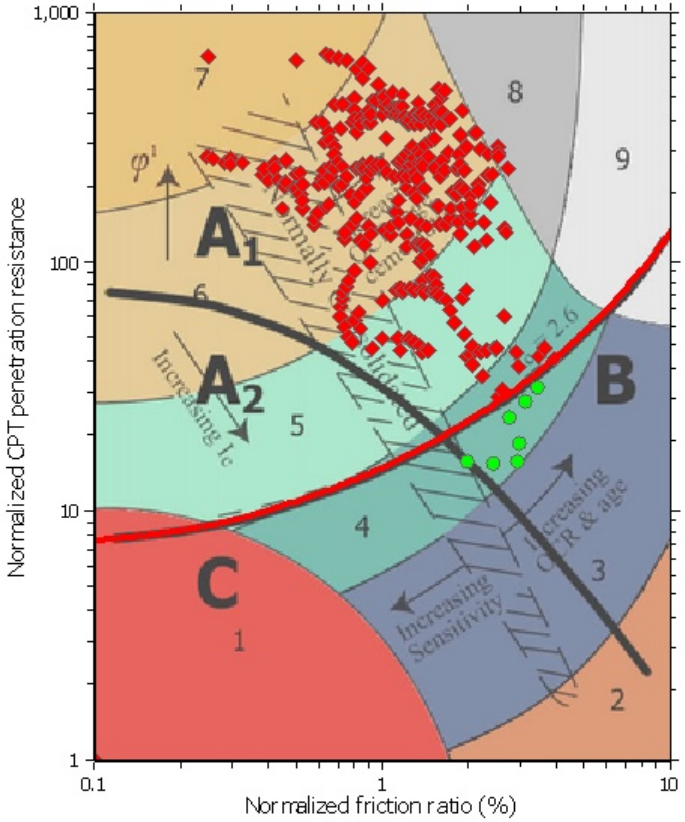
F.S. color scheme

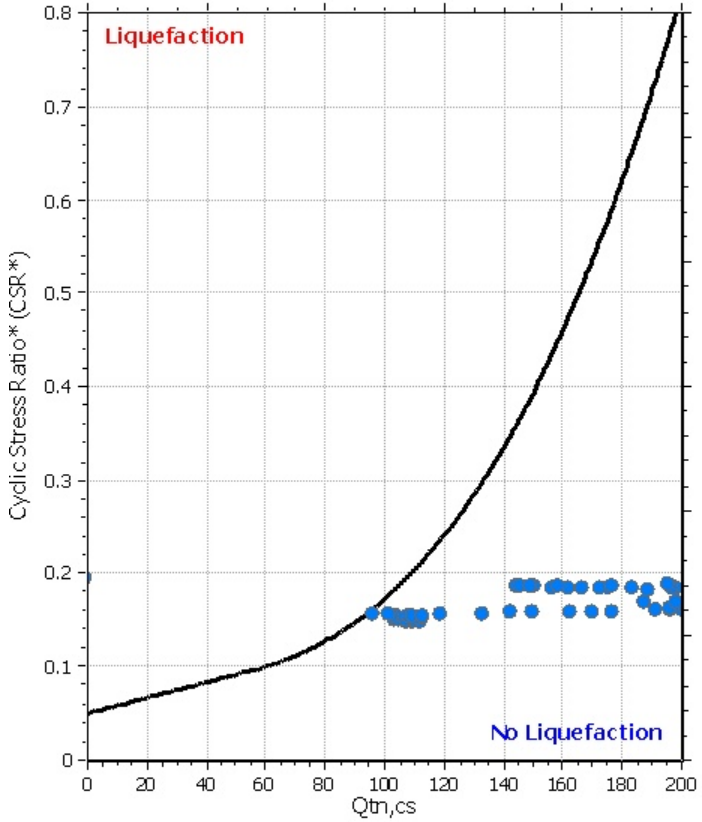
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

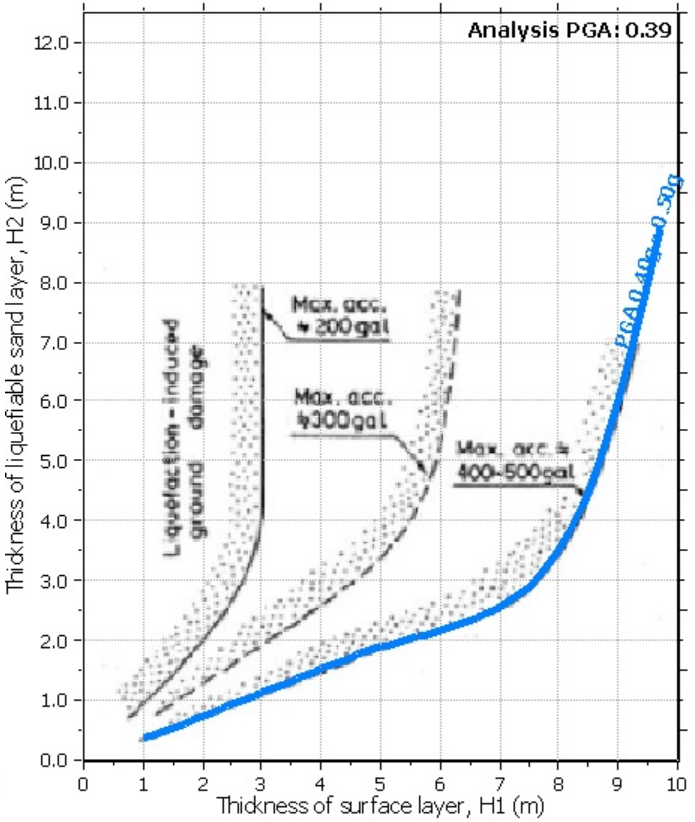
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo







Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

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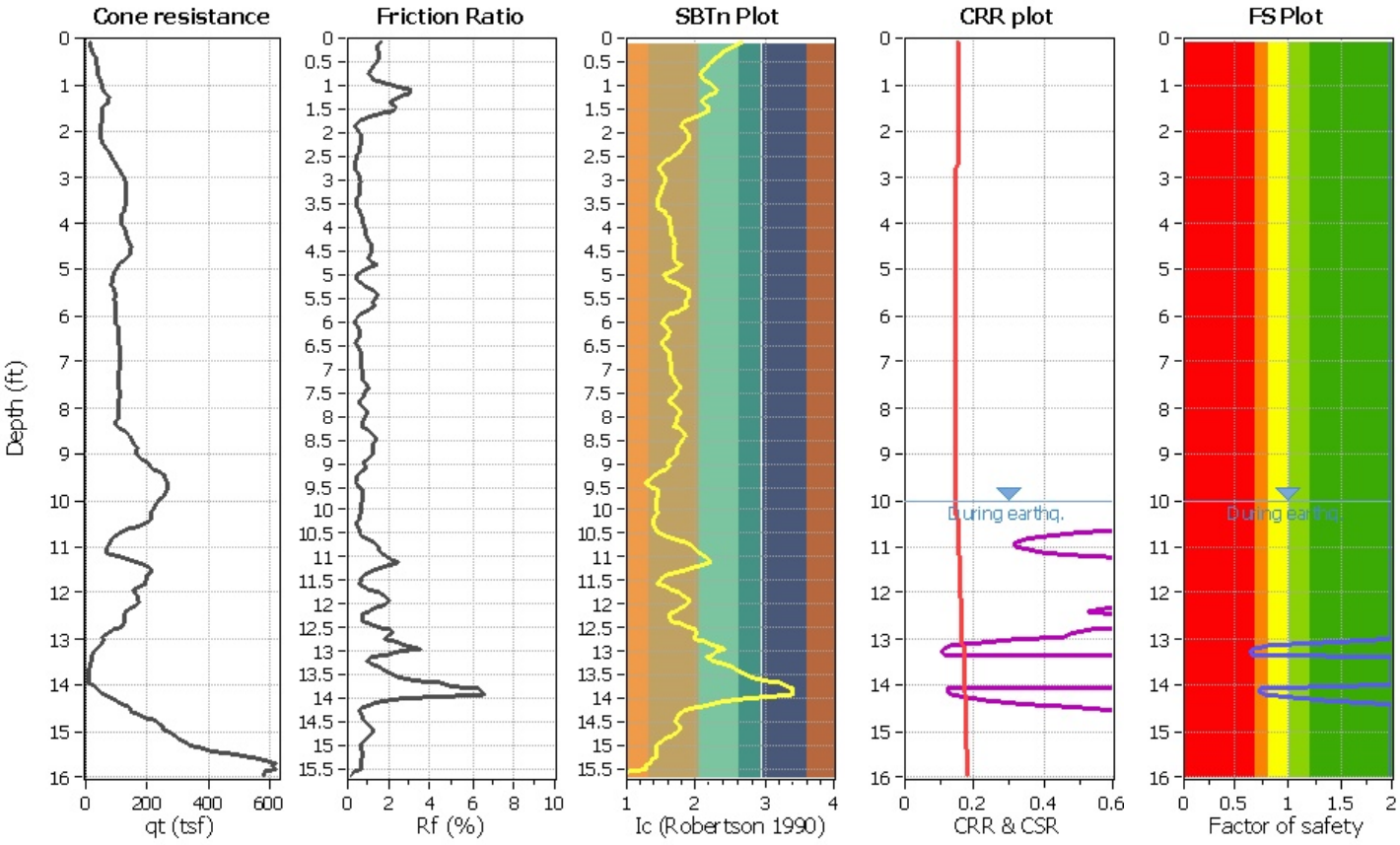
LIQUEFACTION ANALYSIS REPORT

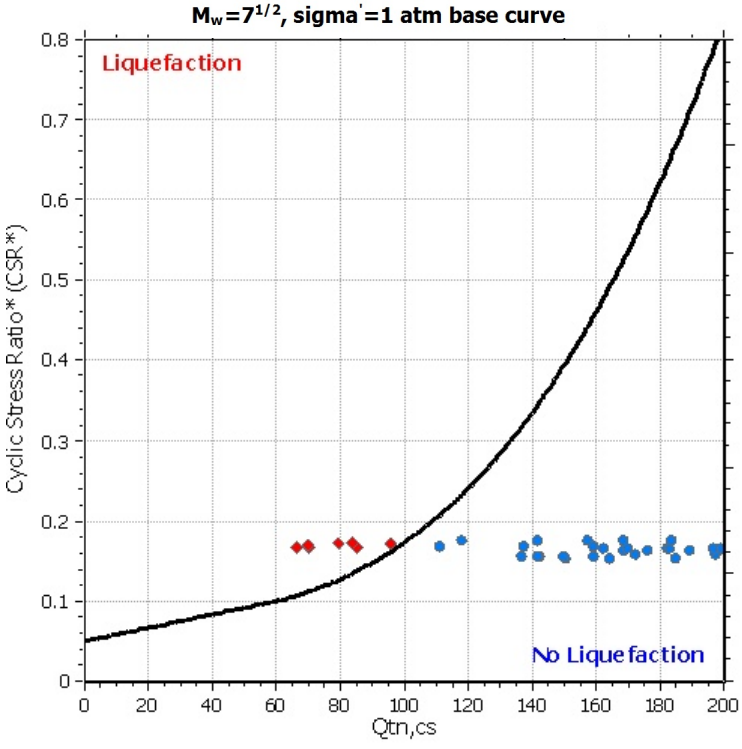
Project title : Bella Mar **Location : 408 Hollister Street**

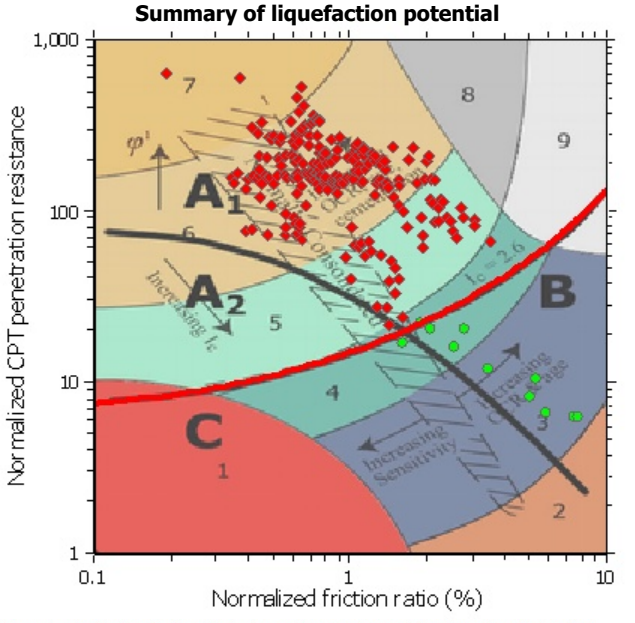
CPT file : CPT-6

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

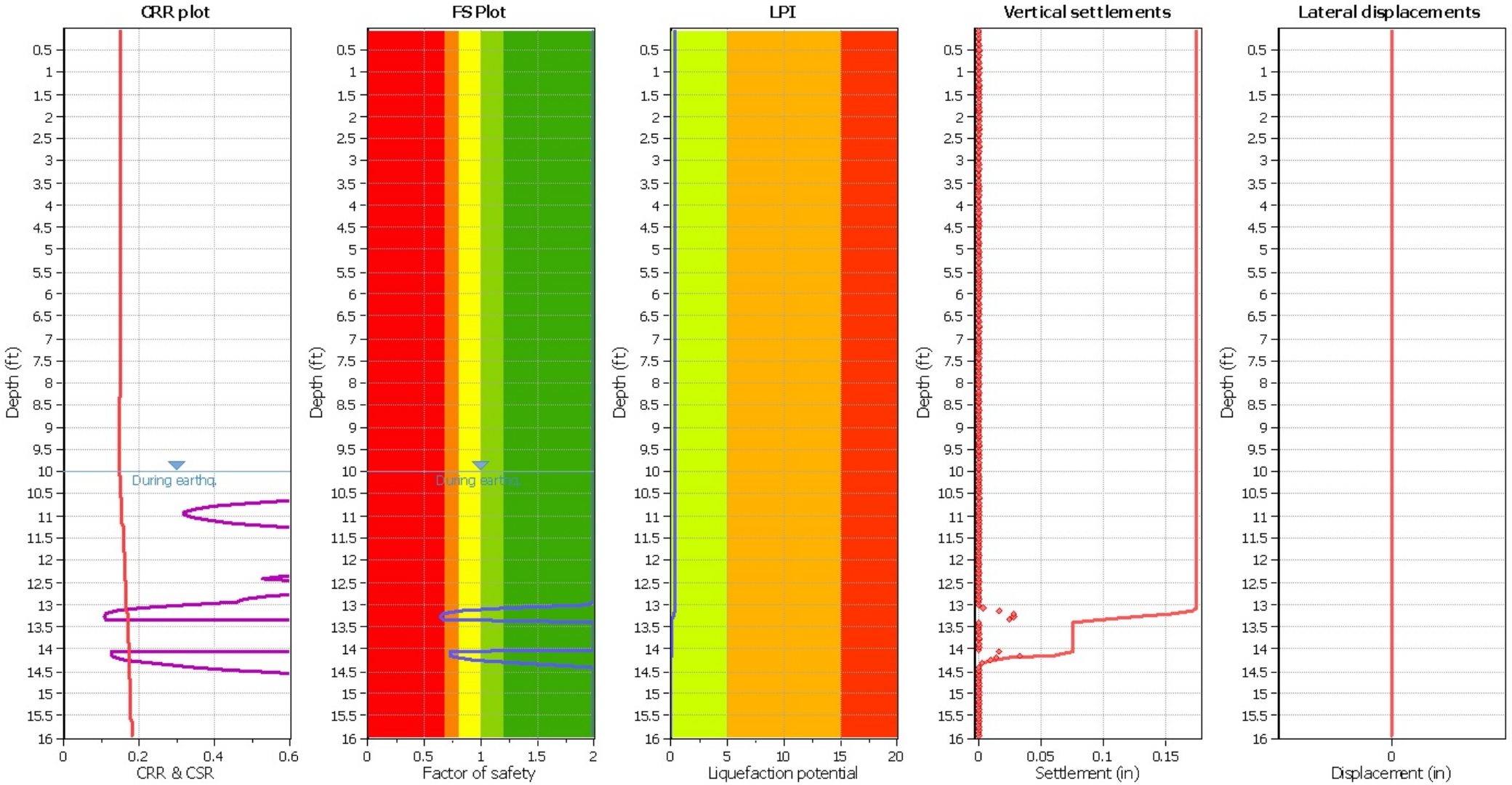






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

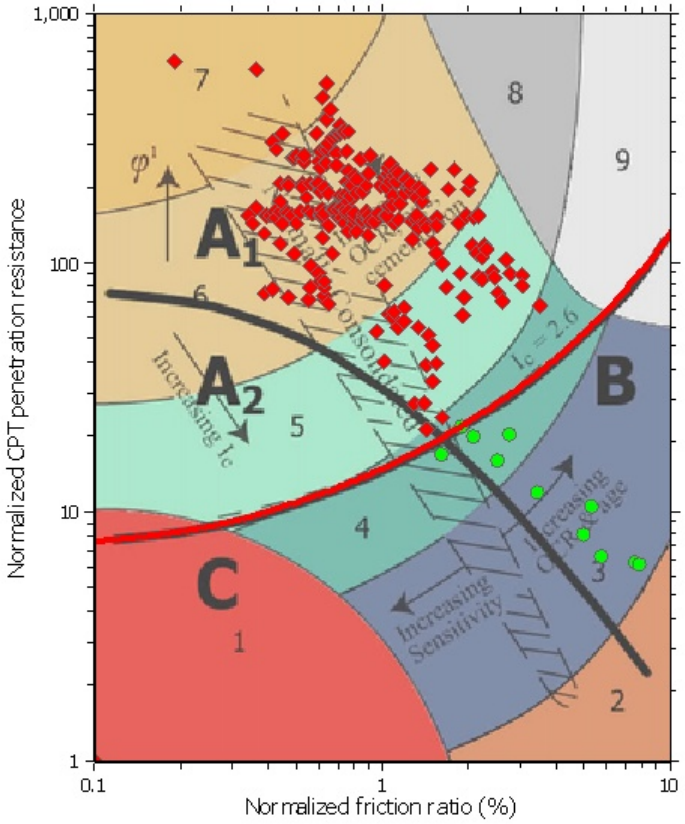
F.S. color scheme

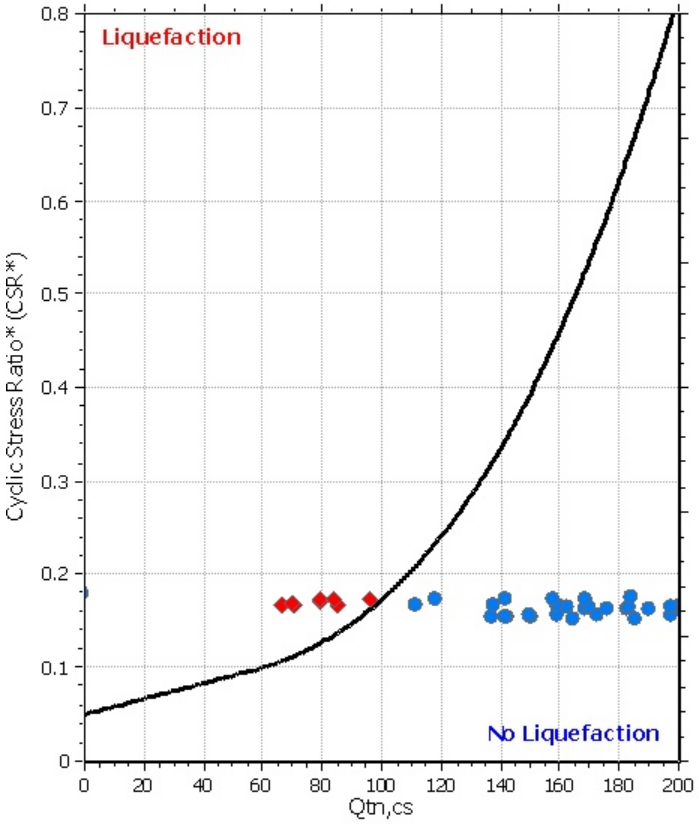
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

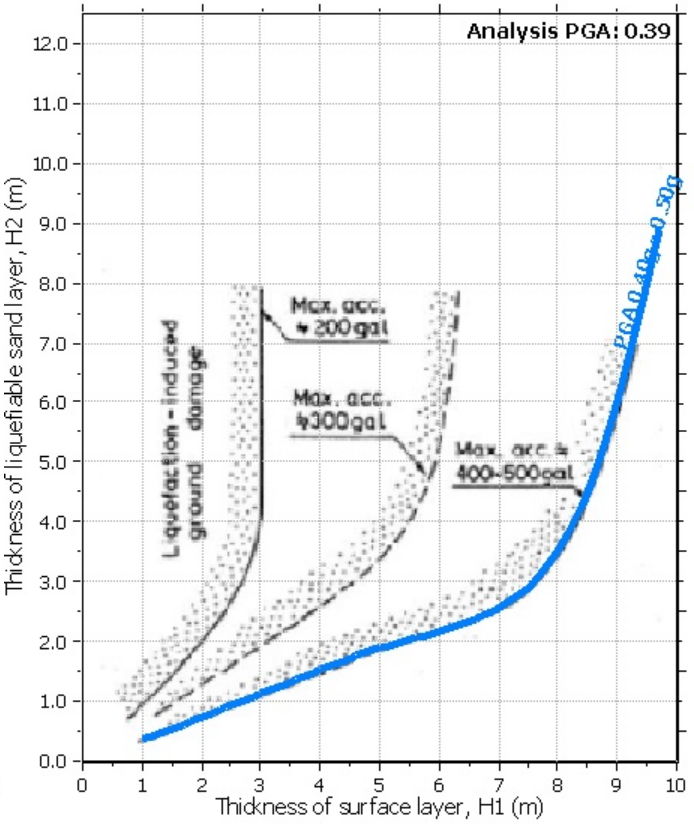
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo







Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

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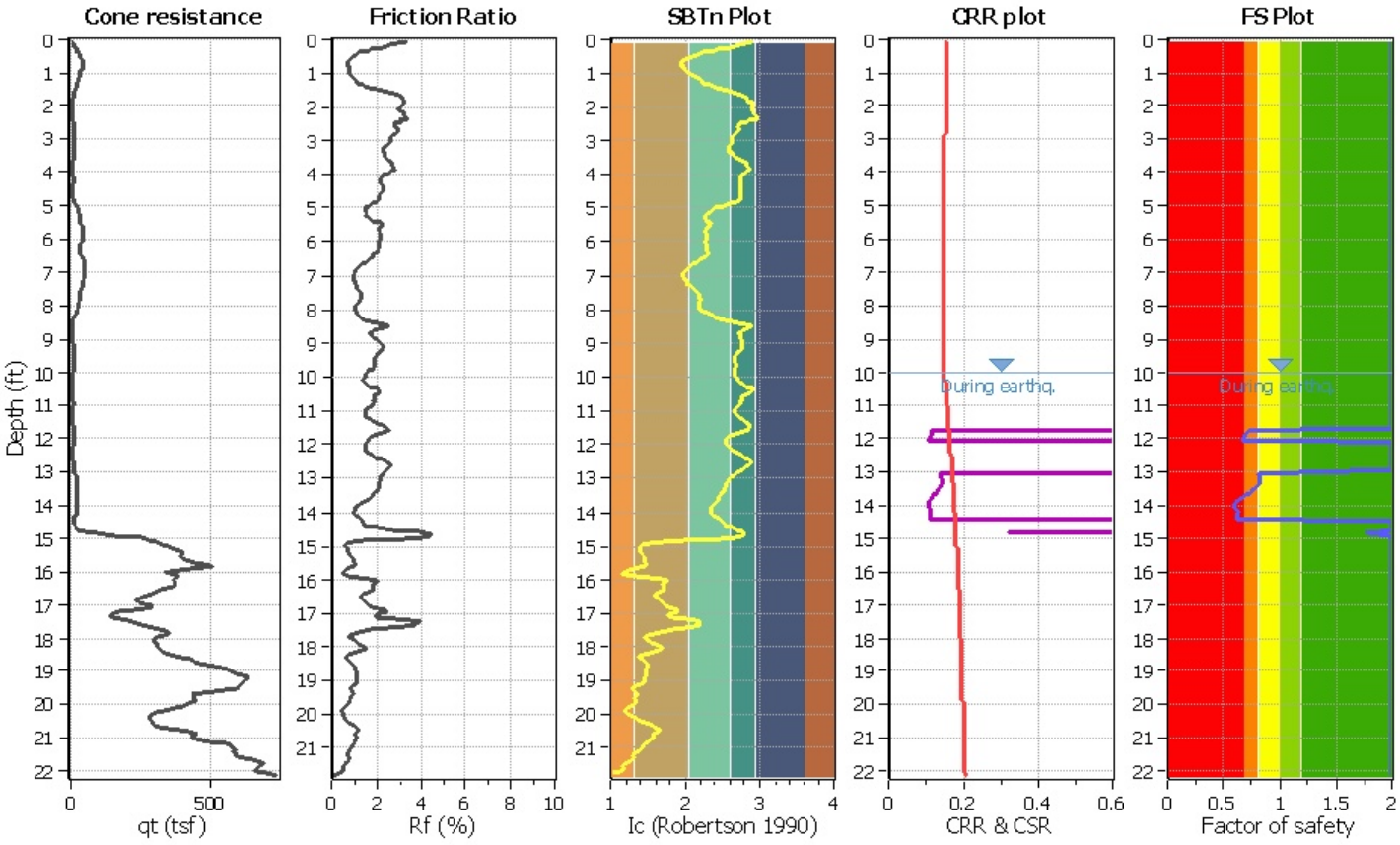
LIQUEFACTION ANALYSIS REPORT

Project title : Bella Mar **Location : 408 Hollister Street**

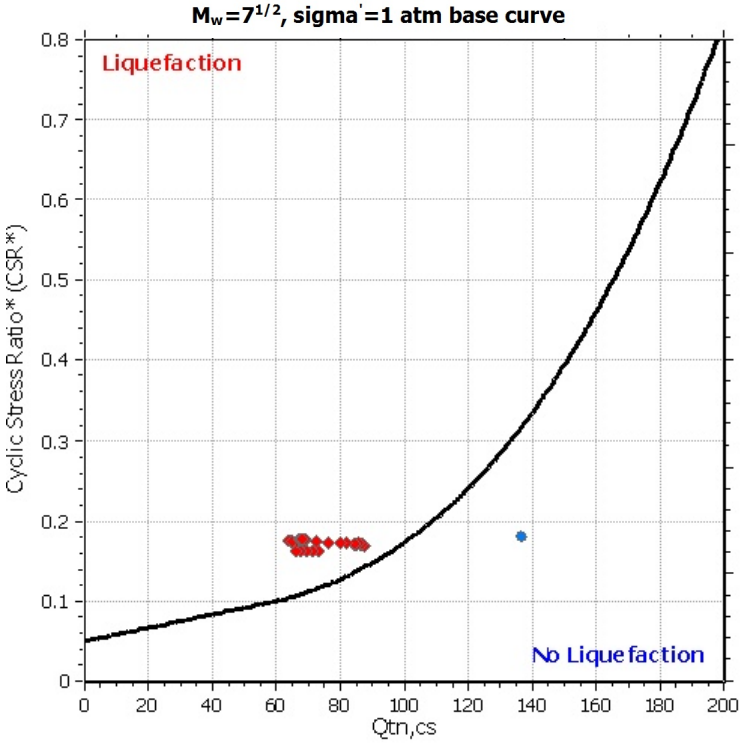
CPT file : CPT-7

Input parameters and analysis data

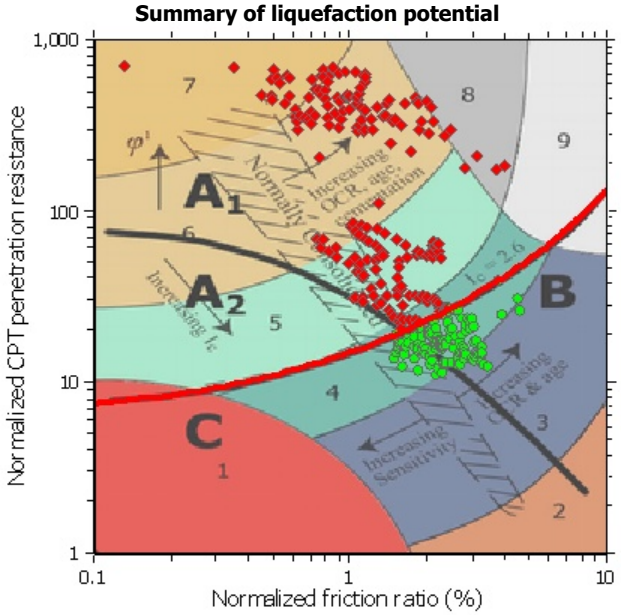
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

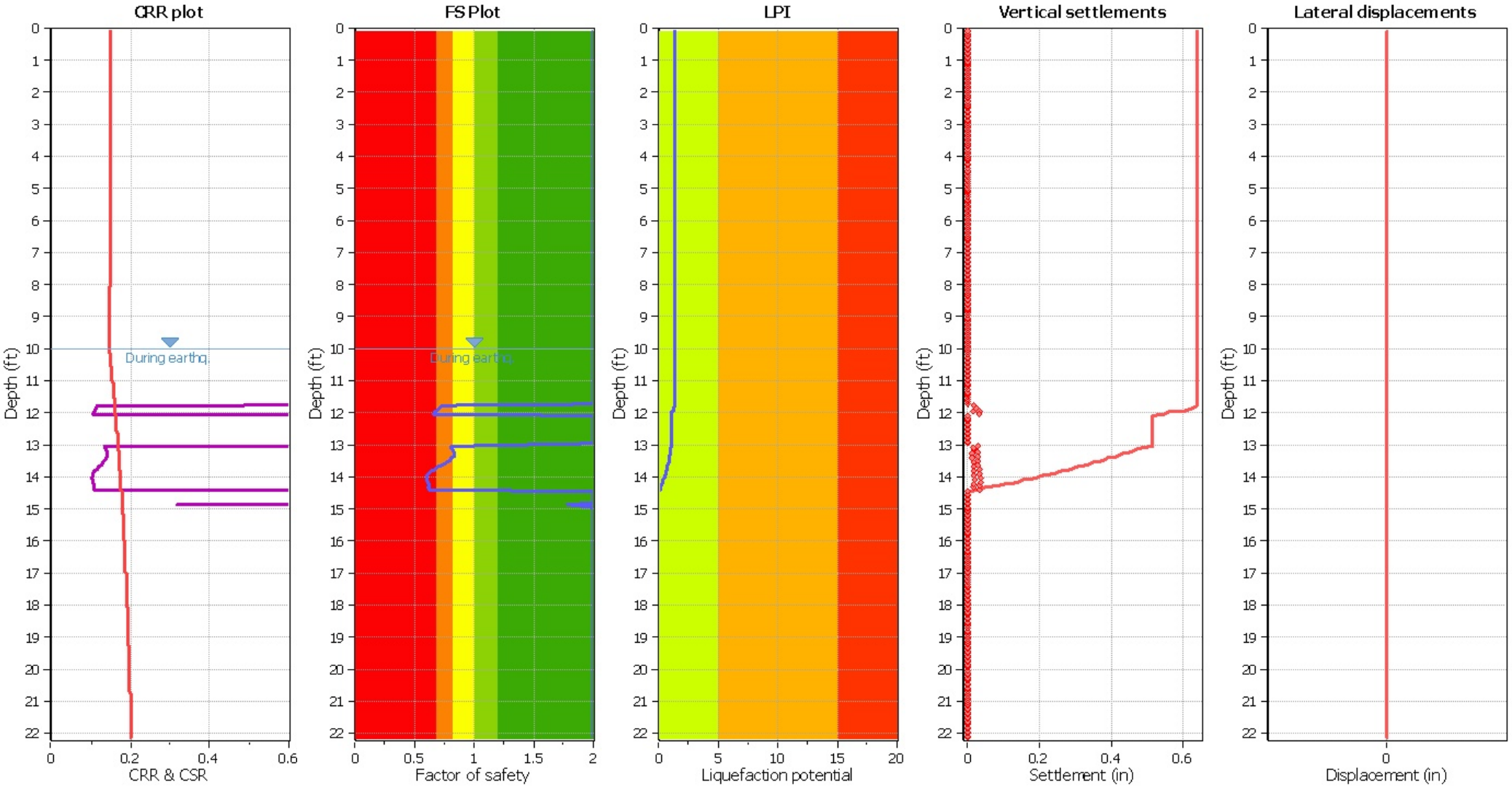


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

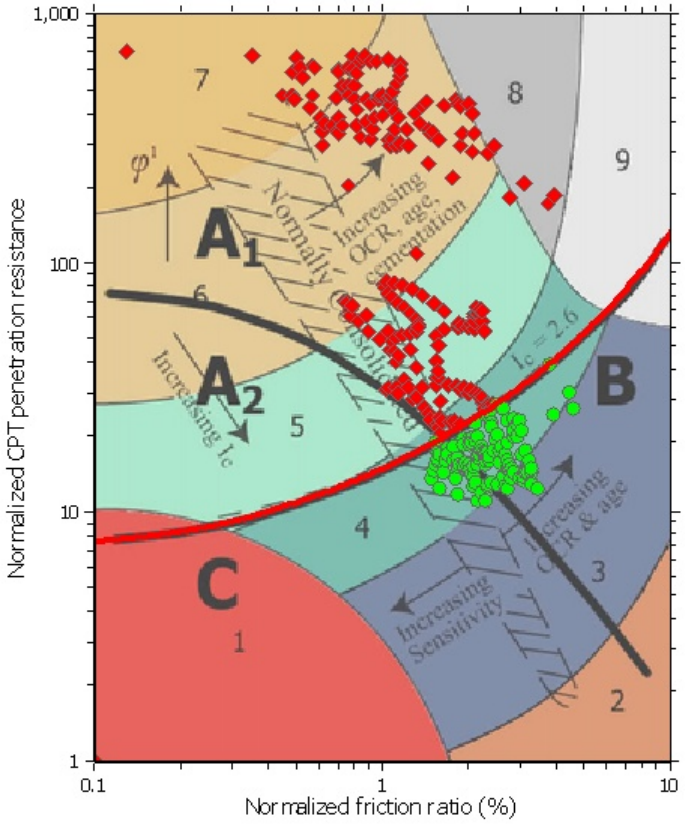
F.S. color scheme

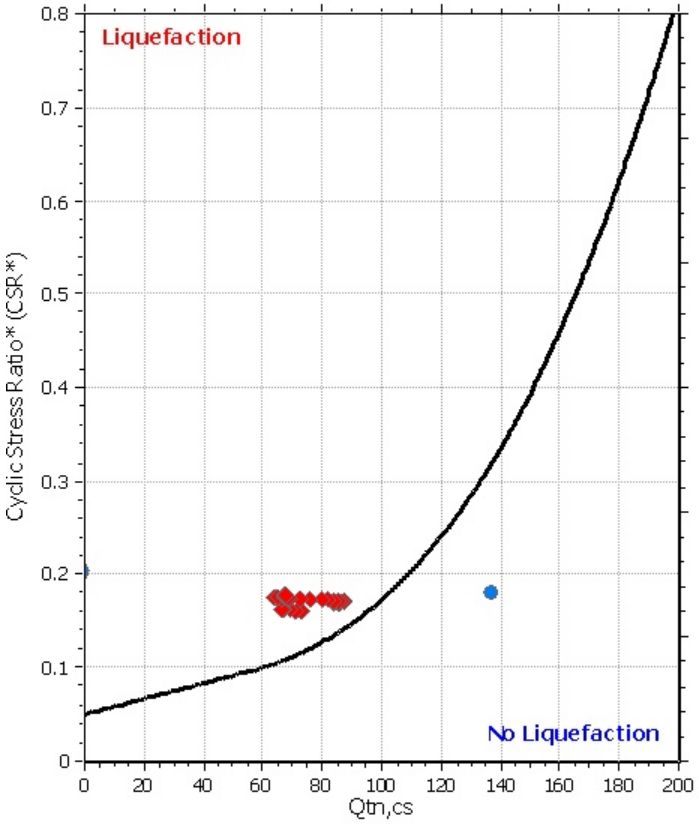
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

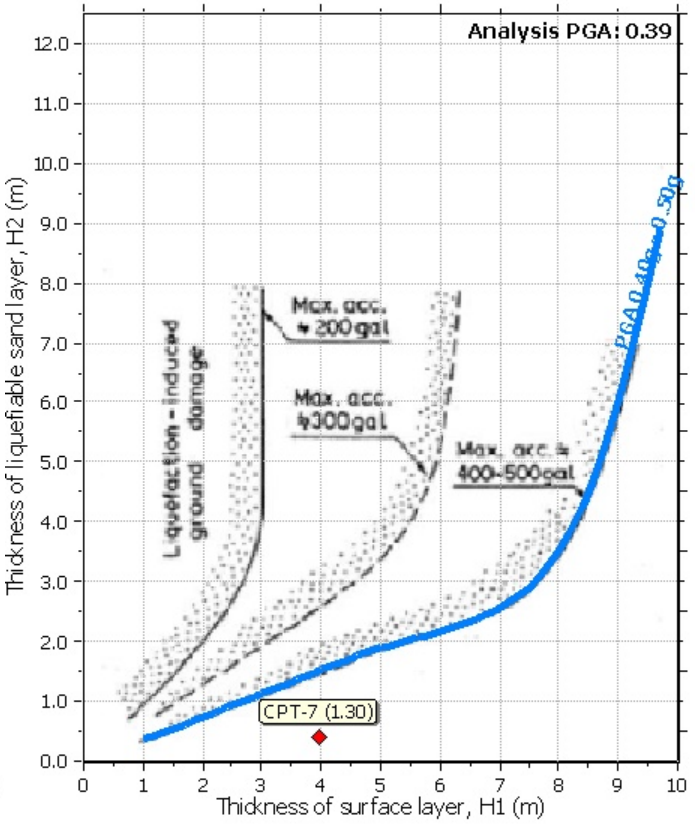
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo







Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Geocon Incorporated
6960 Flanders Drive
San Diego, California 92121-2974
www.geoconinc.com

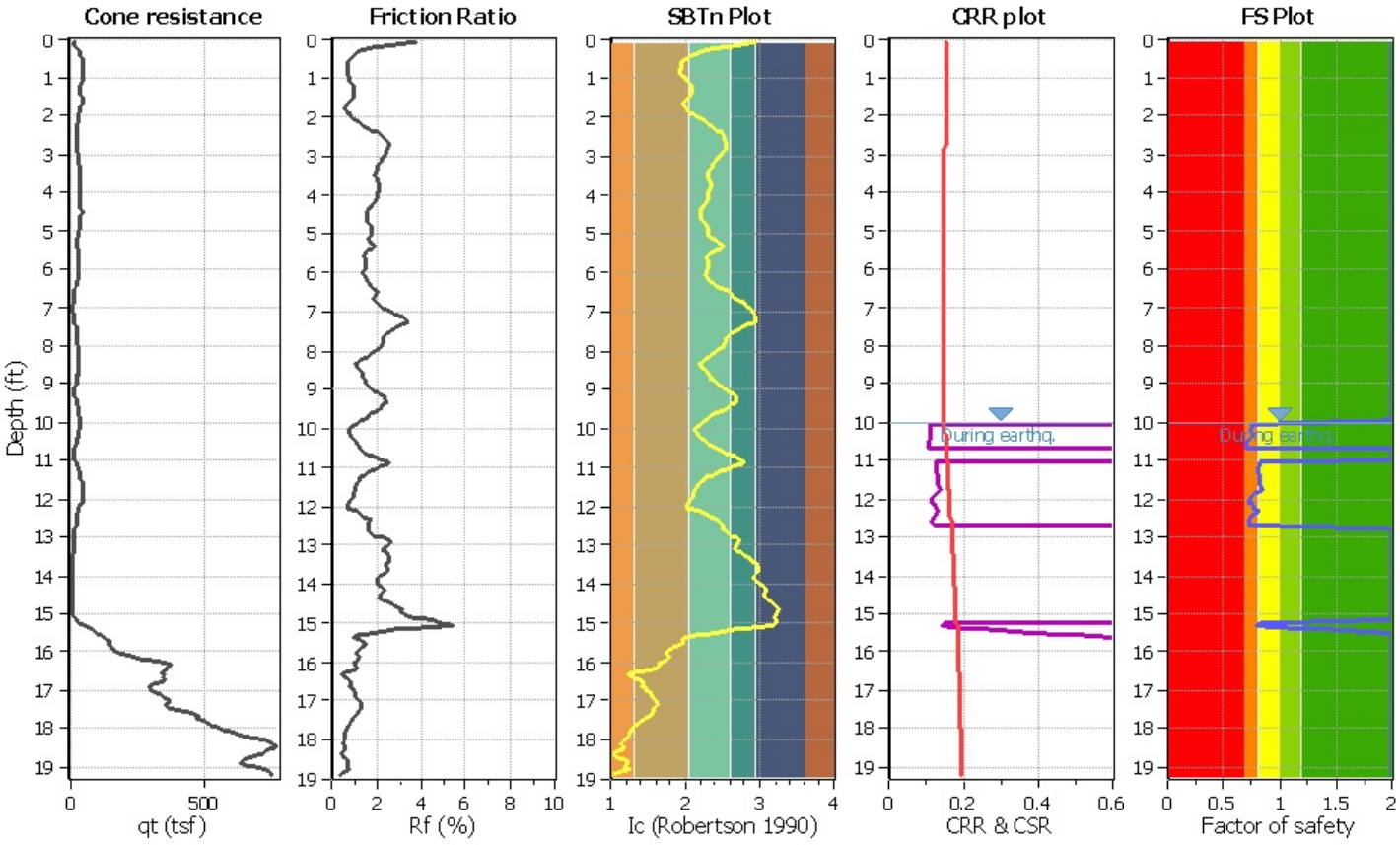
LIQUEFACTION ANALYSIS REPORT

Project title : Bella Mar **Location : 408 Hollister Street**

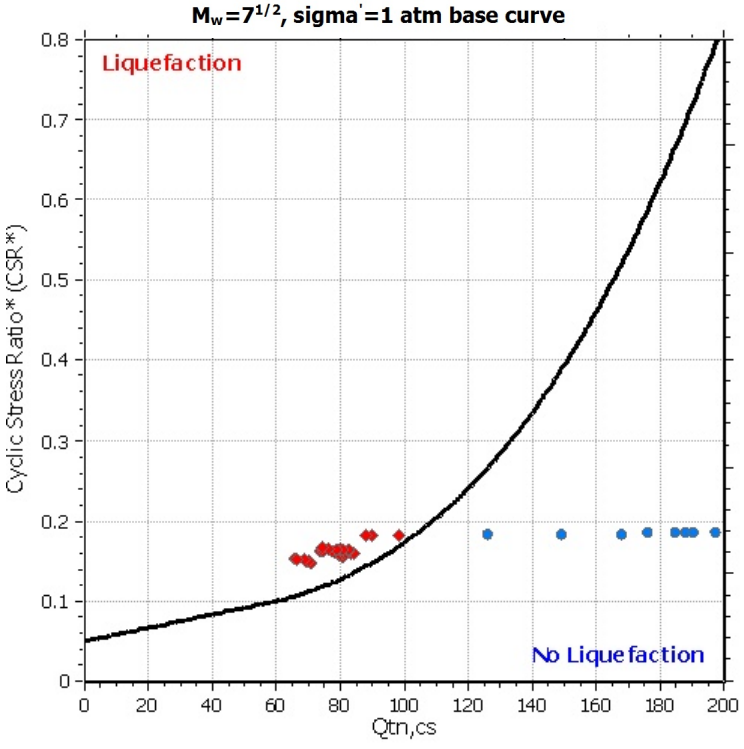
CPT file : CPT-8

Input parameters and analysis data

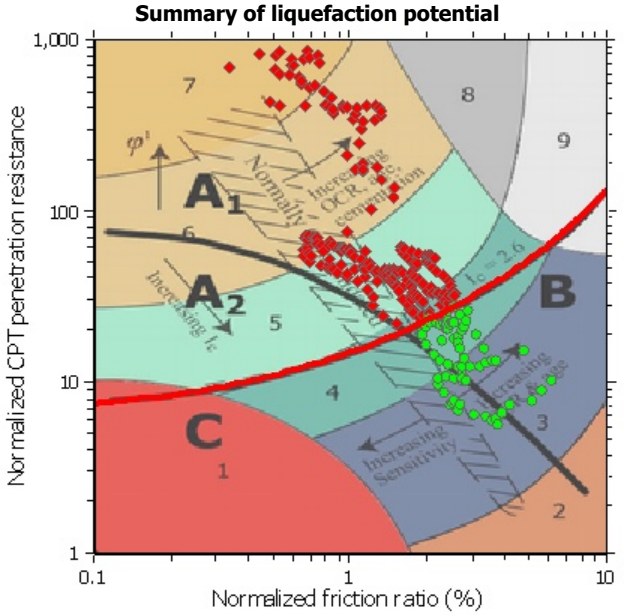
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

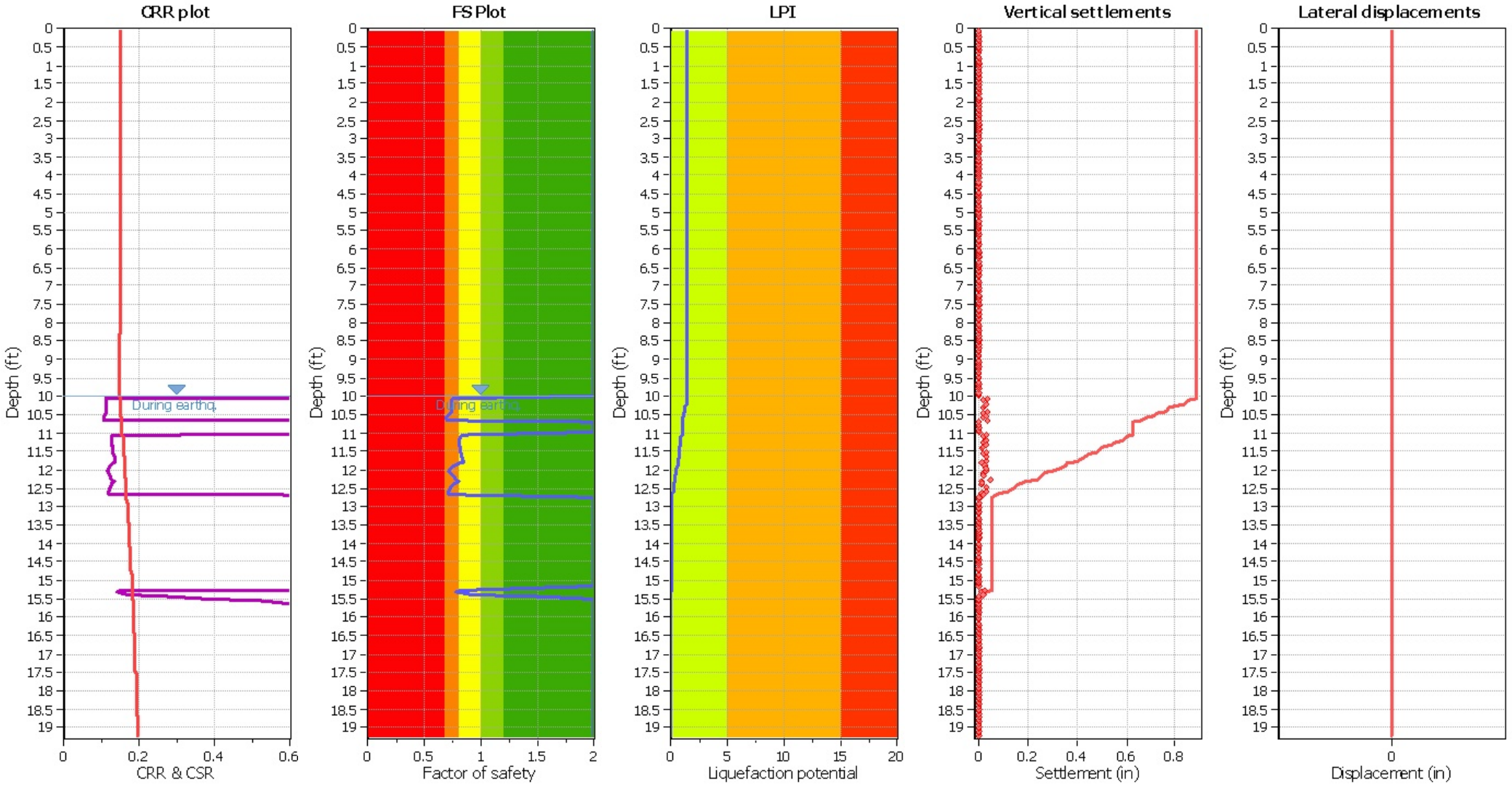


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.12	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

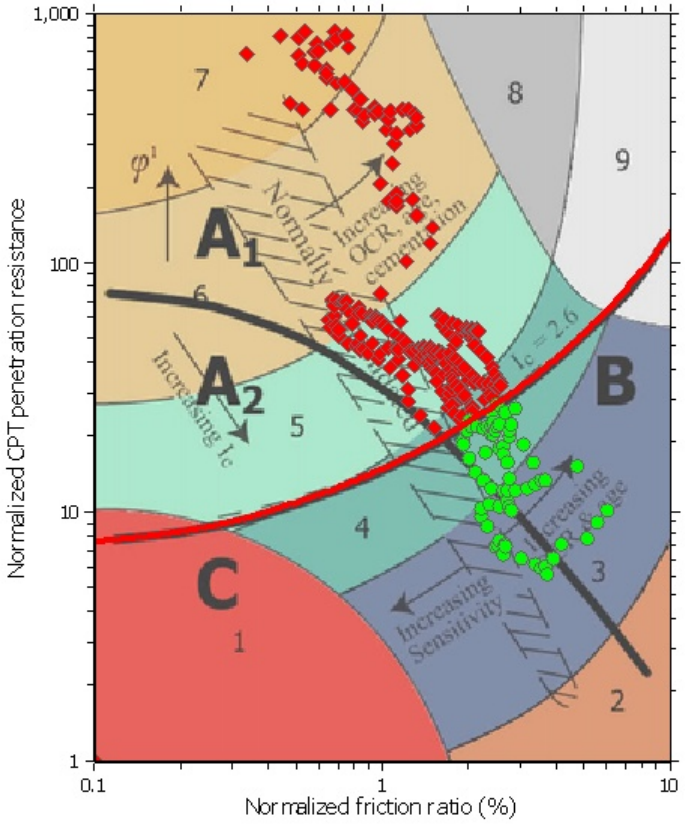
F.S. color scheme

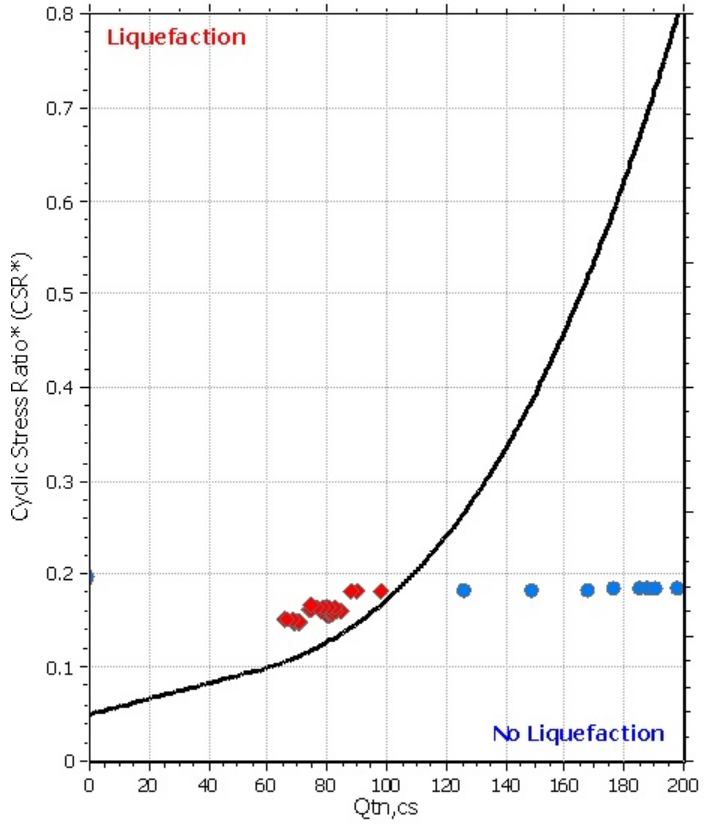
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

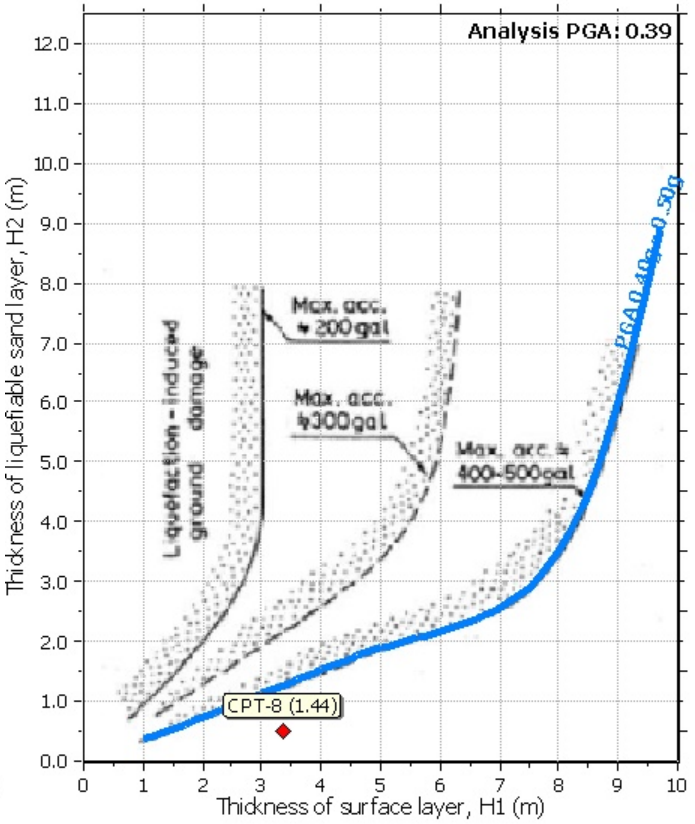
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo







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San Diego, California 92121-2974
www.geoconinc.com

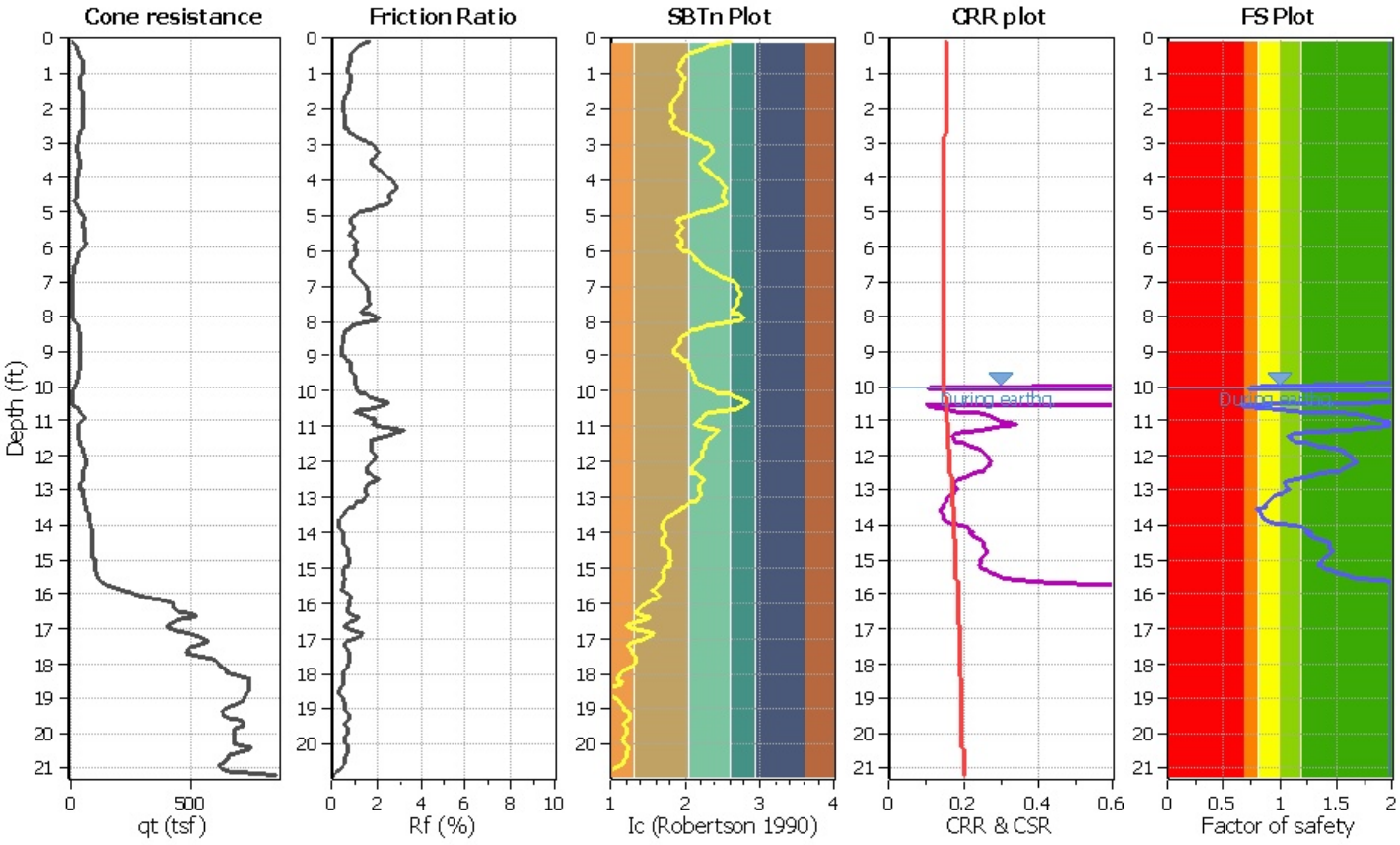
LIQUEFACTION ANALYSIS REPORT

Project title : Bella Mar **Location : 408 Hollister Street**

CPT file : CPT-9

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.12	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.39	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



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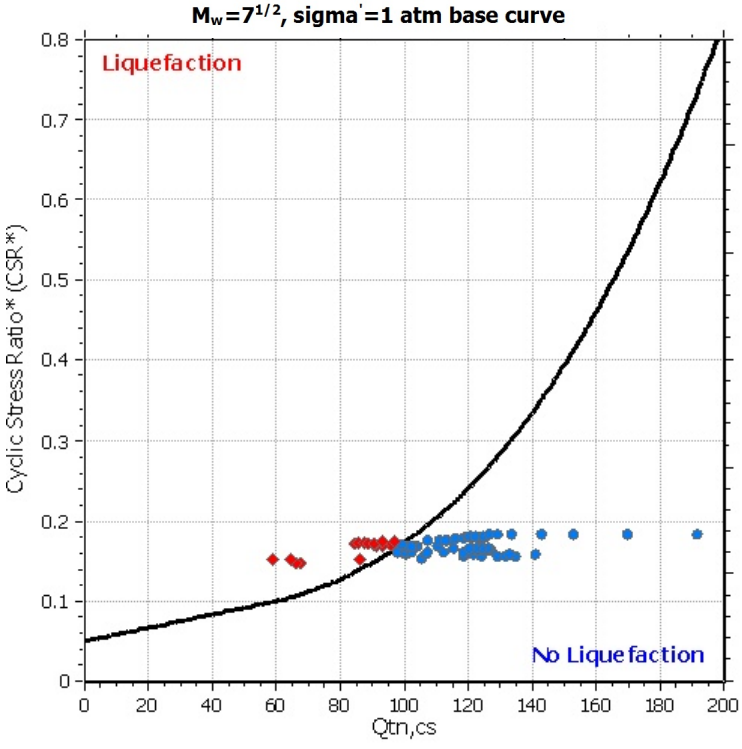
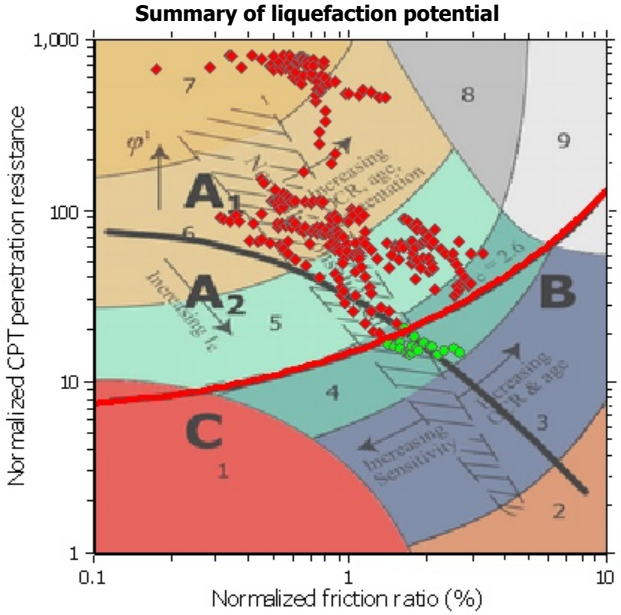


Summary of liquefaction potential



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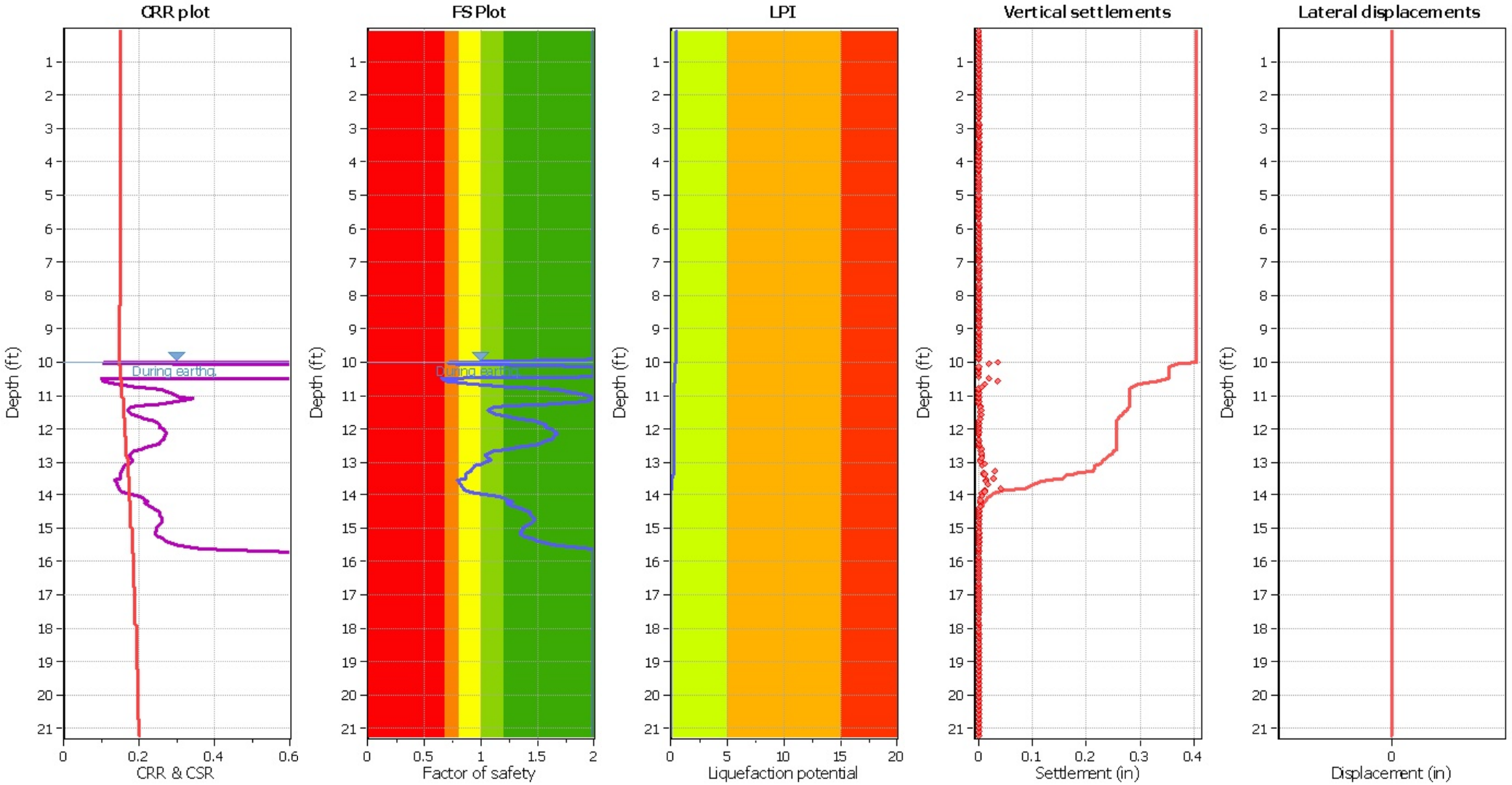
Liquefaction analysis overall plot



Input parameters and analysis data

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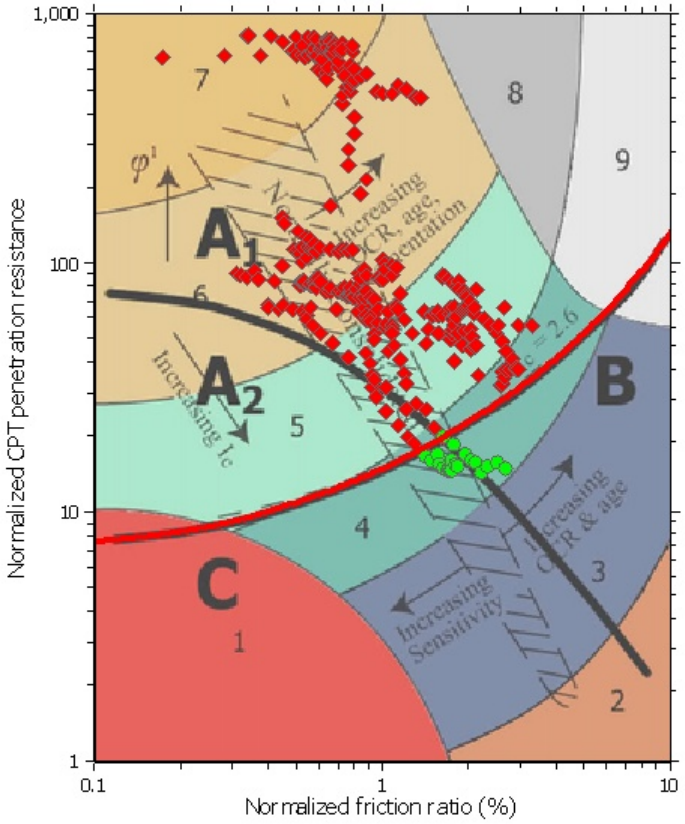
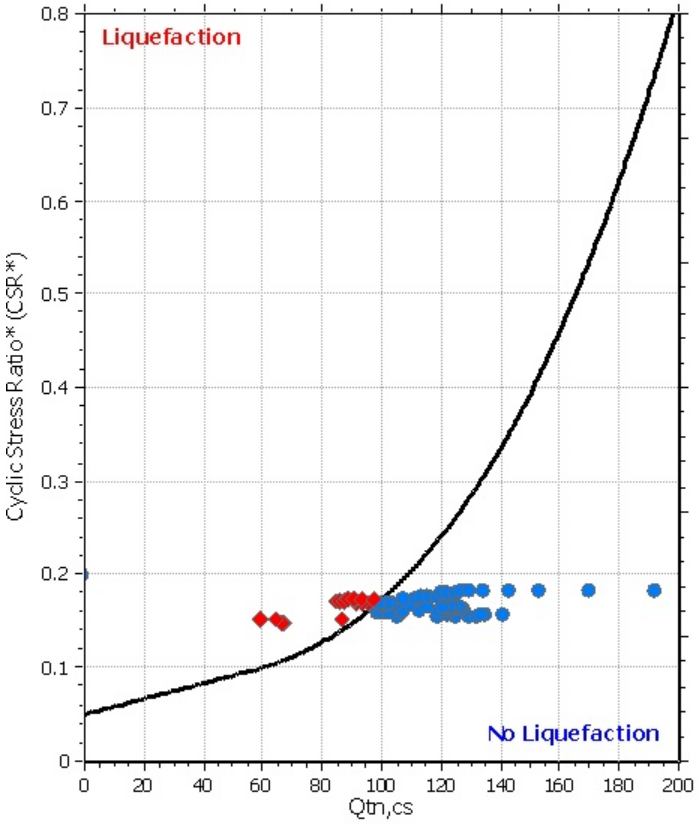
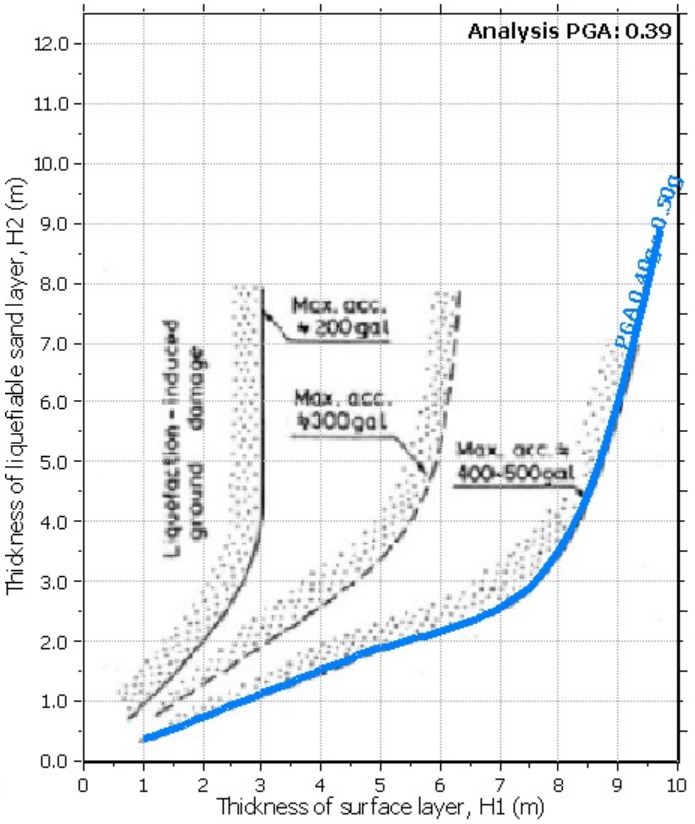
Liquefaction analysis summary plo







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Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
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Peak ground acceleration:	0.39	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

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Hammer Energy Correction Factors

Reference: Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10

Project Name: 408 Hollister Street Date: 4/4/2019

Project Number: G2129-52-03

Hole Diameter, Inches: 8 Hole Diameter Correction, C_B: 1.15

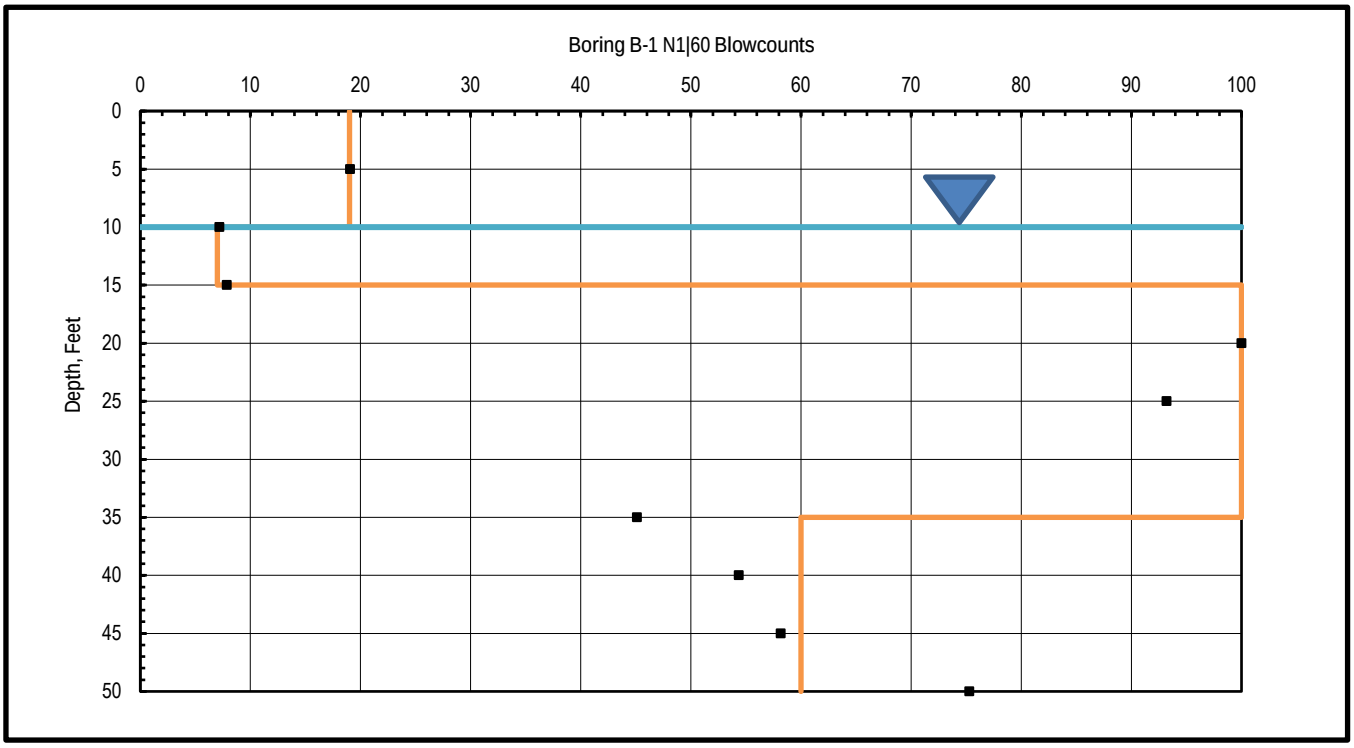
Average Unit Weight, γ (pcf): 120

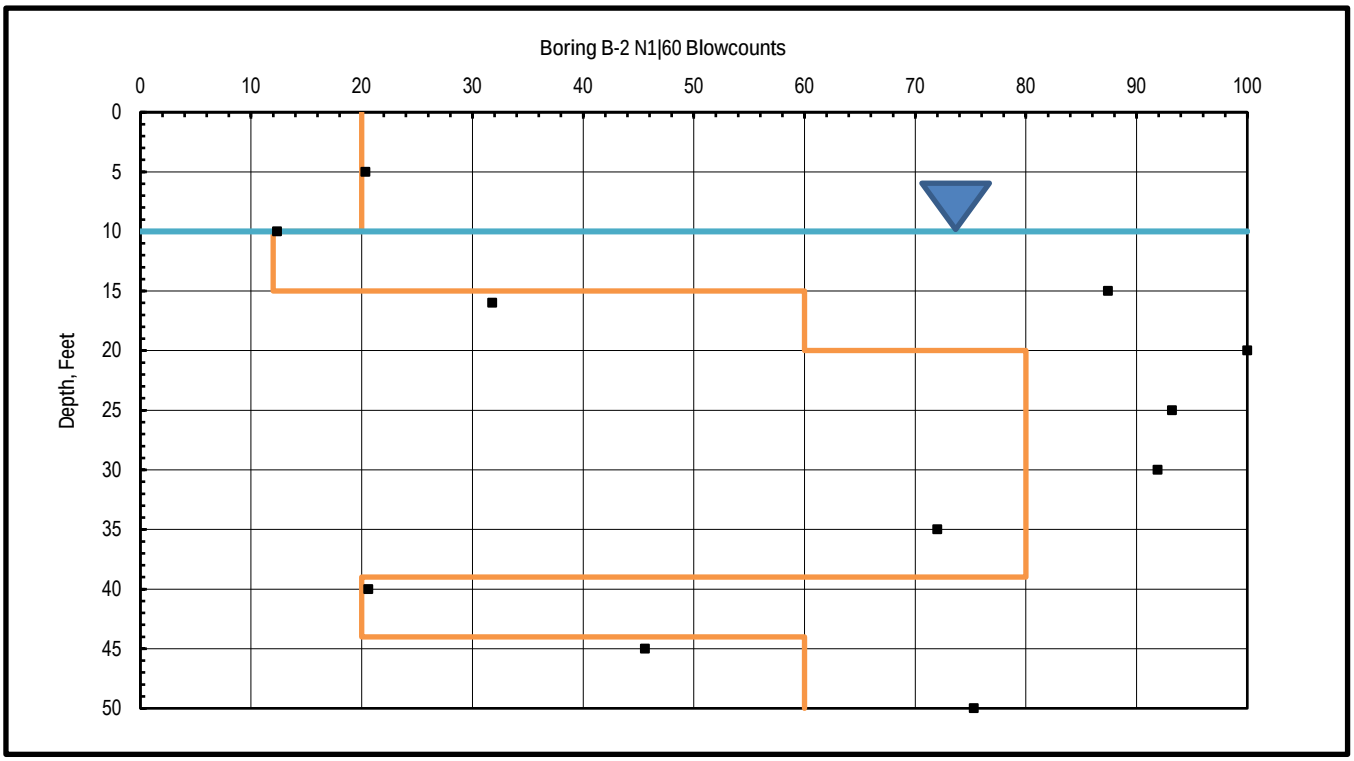
Adjustment Factor for 350 LB Hammer Above Groundwater: 1.00 <-- Enter 1.0 if an adjustment is not required; Applied to "MC" Samples

Adjustment Factor for 350 LB Hammer Below Groundwater: 1.00 <-- Enter 1.0 if an adjustment is not required; Applied to "MC" Samples

Approximate Depth to Groundwater in Boring B-1 and B-2: 10 *Auto, Cathead, or Downhole Hammer

					Adjust for each GWT Level		Energy Correction, C_E (1.0 Safe-T-Driver/Cathead, 1.3 Automatic)				
Sample	Depth, Feet	Field Blow Count (per Foot)	Type of Sampler (MC or SPT)	Hammer Type* (A/C/D)	Equiv. SPT Blow Count, N	σ'_v, psf	Overburden Pressure Correction, C_N	Energy Ratio Correction, C_E	Rod Length Correction, C_R	Sampling Correction, C_S	N1|60 Blowcounts (Prior to Fines)
B1-2	5.0	15	MC	A	10.0	600.0	1.70	1.3	0.75	1.00	19.06
B1-3	10.0	7	MC	A	4.7	1200.0	1.29	1.3	0.80	1.00	7.21
B1-4	15.0	8	MC	A	5.3	1488.0	1.16	1.3	0.85	1.00	7.86
B1-5	20.0	100	MC	A	66.7	1776.0	1.06	1.3	0.95	1.00	100.00
B1-6	25.0	100	MC	A	66.7	2064.0	0.98	1.3	0.95	1.00	93.20
B1-7	35.0	52	MC	A	34.7	2640.0	0.87	1.3	1.00	1.00	45.11
B1-8	40.0	66	MC	A	44.0	2928.0	0.83	1.3	1.00	1.00	54.37
B1-9	45.0	74	MC	A	49.3	3216.0	0.79	1.3	1.00	1.00	58.16
B1-10	50.0	100	MC	A	66.7	3504.0	0.76	1.3	1.00	1.00	75.30
B2-2	5.0	16	MC	A	10.7	600.0	1.70	1.3	0.75	1.00	20.33
B2-3	10.0	12	MC	A	8.0	1200.0	1.29	1.3	0.80	1.00	12.35
B2-4	15.0	89	MC	A	59.3	1488.0	1.16	1.3	0.85	1.00	87.41
B2-5	16.0	33	MC	A	22.0	1545.6	1.14	1.3	0.85	1.00	31.80
B2-6	20.0	100	MC	A	66.7	1776.0	1.06	1.3	0.95	1.00	100.00
B2-7	25.0	100	MC	A	66.7	2064.0	0.98	1.3	0.95	1.00	93.20
B2-8	30.0	100	MC	A	66.7	2352.0	0.92	1.3	1.00	1.00	91.91
B2-9	35.0	83	MC	A	55.3	2640.0	0.87	1.3	1.00	1.00	72.00
B2-10	40.0	25	MC	A	16.7	2928.0	0.83	1.3	1.00	1.00	20.59
B2-11	45.0	58	MC	A	38.7	3216.0	0.79	1.3	1.00	1.00	45.59
B2-12	50.0	100	MC	A	66.7	3504.0	0.76	1.3	1.00	1.00	75.30





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

LR / SW

HAMMER ENERGY CORRECTIONS

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

PROJECT NO. G2129-52-03

GEOCON

Liquefaction Analysis Using SPT

References
1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*
2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

Project Name:	408 Hollister Street
Project Number:	G2129-52-03
Boring:	B-1

a_{max}/g	0.39	Include Kσ (Y/N)	N
Magnitude	6.1	Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Groundwater Depth, Ft	10.0	Minimum Factor of Safety for Liquefaction	1
Reference Pressure, p_a	2000		
Unit Weight of Water	62.4		
Soil Unit Weight, pcf	120		

Enter for Fine-Grained Materials | Old | New | MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$ | From Graph

Depth, ft	$N_1\|_{60}$	Fines Content, FC (%)	Water Content, w_C (%)	Liquid Limit	Plastic Limit	Plasticity Index	$N_1\|_{60}$, Adj. for Fines	$N_1\|_{60}$, Adj. for Fines	σ, psf	σ', psf	r_d	K_σ	NCEER $CRR_{7.5}$	RAUCH $CRR_{7.5}$	CSR M=7.5	Fines Liquefiable (Y/N)	Liquefaction Potential	Factor of Safety	Volumetric Strain, %	Settlement, in.
1	19	20	25.0	--	--	--	24.1	21.5	120.0	120.0	1.00	1.00	0.272	0.276	0.151	--	Above GWT	1.803		
2	19	20	25.0	--	--	--	24.1	21.5	240.0	240.0	1.00	1.00	0.272	0.276	0.150	--	Above GWT	1.807		
3	19	20	25.0	--	--	--	24.1	21.5	360.0	360.0	0.99	1.00	0.272	0.276	0.150	--	Above GWT	1.811		
4	19	20	25.0	--	--	--	24.1	21.5	480.0	480.0	0.99	1.00	0.272	0.276	0.150	--	Above GWT	1.816		
5	19	20	25.0	--	--	--	24.1	21.5	600.0	600.0	0.99	1.00	0.272	0.276	0.149	--	Above GWT	1.820		
6	19	20	25.0	--	--	--	24.1	21.5	720.0	720.0	0.99	1.00	0.272	0.276	0.149	--	Above GWT	1.824		
7	19	20	25.0	--	--	--	24.1	21.5	840.0	840.0	0.99	1.00	0.272	0.276	0.149	--	Above GWT	1.828		
8	19	20	25.0	--	--	--	24.1	21.5	960.0	960.0	0.98	1.00	0.272	0.276	0.148	--	Above GWT	1.832		
9	19	20	25.0	--	--	--	24.1	21.5	1080.0	1080.0	0.98	1.00	0.272	0.276	0.148	--	Above GWT	1.837		
10	7	14	25.0	--	--	--	9.5	8.1	1200.0	1200.0	0.98	1.00	0.104	0.109	0.148	--	LIQUEFIABLE	0.708	3.1	0.372
11	7	14	25.0	--	--	--	9.5	8.1	1320.0	1257.6	0.98	1.00	0.104	0.109	0.155	--	LIQUEFIABLE	0.676	3.1	0.372
12	7	14	25.0	--	--	--	9.5	8.1	1440.0	1315.2	0.97	1.00	0.104	0.109	0.161	--	LIQUEFIABLE	0.649	3.1	0.372
13	7	14	25.0	--	--	--	9.5	8.1	1560.0	1372.8	0.97	1.00	0.104	0.109	0.167	--	LIQUEFIABLE	0.627	3.1	0.372
14	7	14	25.0	--	--	--	9.5	8.1	1680.0	1430.4	0.97	1.00	0.104	0.109	0.172	--	LIQUEFIABLE	0.608	3.1	0.372
15	100	14	30.0	--	--	--	106.4	106.3	1800.0	1488.0	0.97	1.00	0.800	0.800	0.177	--	NL	4.532		
16	100	14	30.0	--	--	--	106.4	106.3	1920.0	1545.6	0.97	1.00	0.800	0.800	0.181	--	NL	4.423		
17	100	14	16.0	--	--	--	106.4	106.3	2040.0	1603.2	0.96	1.00	0.800	0.800	0.185	--	NL	4.328		
18	100	14	16.0	--	--	--	106.4	106.3	2160.0	1660.8	0.96	1.00	0.800	0.800	0.188	--	NL	4.244		
19	100	14	16.0	--	--	--	106.4	106.3	2280.0	1718.4	0.96	1.00	0.800	0.800	0.192	--	NL	4.171		
20	100	14	16.0	--	--	--	106.4	106.3	2400.0	1776.0	0.96	1.00	0.800	0.800	0.195	--	NL	4.106		
21	100	14	16.0	--	--	--	106.4	106.3	2520.0	1833.6	0.95	1.00	0.800	0.800	0.198	--	NL	4.048		
22	100	14	16.0	--	--	--	106.4	106.3	2640.0	1891.2	0.95	1.00	0.800	0.800	0.200	--	NL	3.998		
23	100	14	16.0	--	--	--	106.4	106.3	2760.0	1948.8	0.95	1.00	0.800	0.800	0.202	--	NL	3.953		
24	100	14	16.0	--	--	--	106.4	106.3	2880.0	2006.4	0.95	1.00	0.800	0.800	0.204	--	NL	3.913		
25	100	14	16.0	--	--	--	106.4	106.3	3000.0	2064.0	0.94	1.00	0.800	0.800	0.206	--	NL	3.879		
26	100	14	16.0	--	--	--	106.4	106.3	3120.0	2121.6	0.94	1.00	0.800	0.800	0.208	--	NL	3.848		
27	100	14	16.0	--	--	--	106.4	106.3	3240.0	2179.2	0.93	1.00	0.800	0.800	0.209	--	NL	3.823		
28	100	14	16.0	--	--	--	106.4	106.3	3360.0	2236.8	0.93	1.00	0.800	0.800	0.210	--	NL	3.801		
29	100	14	16.0	--	--	--	106.4	106.3	3480.0	2294.4	0.93	1.00	0.800	0.800	0.211	--	NL	3.783		
30	100	14	16.0	--	--	--	106.4	106.3	3600.0	2352.0	0.92	1.00	0.800	0.800	0.212	--	NL	3.768		
31	100	14	16.0	--	--	--	106.4	106.3	3720.0	2409.6	0.92	1.00	0.800	0.800	0.213	--	NL	3.757		
32	100	14	16.0	--	--	--	106.4	106.3	3840.0	2467.2	0.91	1.00	0.800	0.800	0.213	--	NL	3.750		
33	100	14	16.0	--	--	--	106.4	106.3	3960.0	2524.8	0.90	1.00	0.800	0.800	0.214	--	NL	3.746		
34	100	14	16.0	--	--	--	106.4	106.3	4080.0	2582.4	0.90	1.00	0.800	0.800	0.214	--	NL	3.745		
35	60	14	25.0	--	--	--	64.7	64.1	4200.0	2640.0	0.89	1.00	0.800	0.800	0.213	--	NL	3.748		
36	60	14	25.0	--	--	--	64.7	64.1	4320.0	2697.6	0.88	1.00	0.800	0.800	0.213	--	NL	3.753		
37	60	14	25.0	--	--	--	64.7	64.1	4440.0	2755.2	0.88	1.00	0.800	0.800	0.213	--	NL	3.762		
38	60	14	25.0	--	--	--	64.7	64.1	4560.0	2812.8	0.87	1.00	0.800	0.800	0.212	--	NL	3.774		
39	60	14	25.0	--	--	--	64.7	64.1	4680.0	2870.4	0.86	1.00	0.800	0.800	0.211	--	NL	3.789		
40	60	14	25.0	--	--	--	64.7	64.1	4800.0	2928.0	0.85	1.00	0.800	0.800	0.210	--	NL	3.807		
41	60	14	25.0	--	--	--	64.7	64.1	4920.0	2985.6	0.84	1.00	0.800	0.800	0.209	--	NL	3.827		
42	60	14	25.0	--	--	--	64.7	64.1	5040.0	3043.2	0.83	1.00	0.800	0.800	0.208	--	NL	3.850		
43	60	14	25.0	--	--	--	64.7	64.1	5160.0	3100.8	0.82	1.00	0.800	0.800	0.206	--	NL	3.877		
44	60	14	25.0	--	--	--	64.7	64.1	5280.0	3158.4	0.81	1.00	0.800	0.800	0.205	--	NL	3.905		
45	60	70	25.0	--	--	--	77.0	66.0	5400.0	3216.0	0.80	1.00	0.800	0.800	0.203	--	NL	3.936		
46	60	70	25.0	--	--	--	77.0	66.0	5520.0	3273.6	0.79	1.00	0.800	0.800	0.202	--	NL	3.969		
47	60	70	25.0	--	--	--	77.0	66.0	5640.0	3331.2	0.78	1.00	0.800	0.800	0.200	--	NL	4.004		
48	60	70	25.0	--	--	--	77.0	66.0	5760.0	3388.8	0.77	1.00	0.800	0.800	0.198	--	NL	4.042		
49	60	10	25.0	--	--	--	62.2	62.9	5880.0	3446.4	0.76	1.00	0.800	0.800	0.196	--	NL	4.081		
50	60	10	25.0	--	--	--	62.2	62.9	6000.0	3504.0	0.75	1.00	0.800	0.800	0.194	--	NL	4.121		
51	60	10	25.0	--	--	--	62.2	62.9	6120.0	3561.6	0.74	1.00	0.800	0.800	0.192	--	NL	4.163		

Total Settlement, S_{LIQ} (in.) = 1.86

Total Liquifiable Layers = 5

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Liquefaction Analysis Using SPT

References
1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*
2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

Project Name:	408 Hollister Street
Project Number:	G2129-52-03
Boring:	B-2

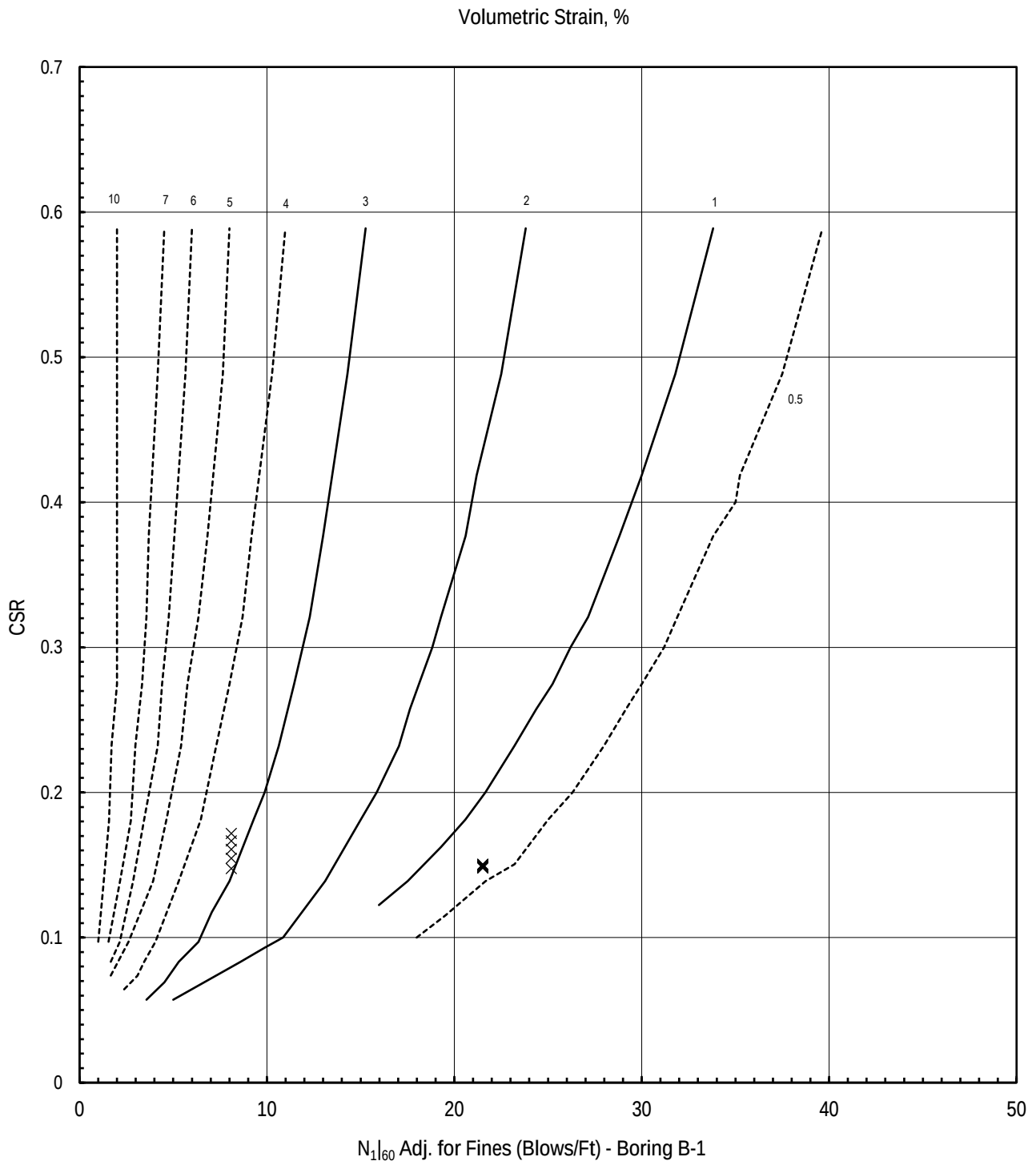
a_{max}/g	0.39	Include Kσ (Y/N)	N
Magnitude	6.1	Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Groundwater Depth, Ft	10.0	Minimum Factor of Safety for Liquefaction	1
Reference Pressure, p_a	2000		
Unit Weight of Water	62.4		
Soil Unit Weight, pcf	120		

Enter for Fine-Grained Materials | Old | New | MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$ | From Graph

Depth, ft	$N_1\|_{60}$	Fines Content, FC (%)	Water Content, w_C (%)	Liquid Limit	Plastic Limit	Plasticity Index	$N_1\|_{60}$, Adj. for Fines	$N_1\|_{60}$, Adj. for Fines	σ, psf	σ', psf	r_d	K_σ	NCEER $CRR_{7.5}$	RAUCH $CRR_{7.5}$	CSR M=7.5	Fines Liquefiable (Y/N)	Liquefaction Potential	Factor of Safety	Volumetric Strain, %	Settlement, in.
1	20	30	25.0	--	--	--	27.8	23.9	120.0	120.0	1.00	1.00	0.344	0.362	0.151	--	Above GWT	2.281		
2	20	30	25.0	--	--	--	27.8	23.9	240.0	240.0	1.00	1.00	0.344	0.362	0.150	--	Above GWT	2.286		
3	20	30	25.0	--	--	--	27.8	23.9	360.0	360.0	0.99	1.00	0.344	0.362	0.150	--	Above GWT	2.292		
4	20	30	25.0	--	--	--	27.8	23.9	480.0	480.0	0.99	1.00	0.344	0.362	0.150	--	Above GWT	2.297		
5	20	30	25.0	--	--	--	27.8	23.9	600.0	600.0	0.99	1.00	0.344	0.362	0.149	--	Above GWT	2.303		
6	20	30	25.0	--	--	--	27.8	23.9	720.0	720.0	0.99	1.00	0.344	0.362	0.149	--	Above GWT	2.308		
7	20	30	25.0	--	--	--	27.8	23.9	840.0	840.0	0.99	1.00	0.344	0.362	0.149	--	Above GWT	2.313		
8	20	30	25.0	--	--	--	27.8	23.9	960.0	960.0	0.98	1.00	0.344	0.362	0.148	--	Above GWT	2.318		
9	20	30	25.0	--	--	--	27.8	23.9	1080.0	1080.0	0.98	1.00	0.344	0.362	0.148	--	Above GWT	2.324		
10	12	14	25.0	--	--	--	14.7	13.4	1200.0	1200.0	0.98	1.00	0.160	0.157	0.148	--	NL	1.087		
11	12	14	25.0	--	--	--	14.7	13.4	1320.0	1257.6	0.98	1.00	0.160	0.157	0.155	--	NL	1.038		
12	12	14	25.0	--	--	--	14.7	13.4	1440.0	1315.2	0.97	1.00	0.160	0.157	0.161	--	LIQUEFIABLE	0.997	2.2	0.264
13	12	14	25.0	--	--	--	14.7	13.4	1560.0	1372.8	0.97	1.00	0.160	0.157	0.167	--	LIQUEFIABLE	0.963	2.2	0.264
14	12	14	25.0	--	--	--	14.7	13.4	1680.0	1430.4	0.97	1.00	0.160	0.157	0.172	--	LIQUEFIABLE	0.934	2.2	0.264
15	60	14	20.0	--	--	--	64.7	64.1	1800.0	1488.0	0.97	1.00	0.800	0.800	0.177	--	NL	4.532		
16	60	14	20.0	--	--	--	64.7	64.1	1920.0	1545.6	0.97	1.00	0.800	0.800	0.181	--	NL	4.423		
17	60	14	20.0	--	--	--	64.7	64.1	2040.0	1603.2	0.96	1.00	0.800	0.800	0.185	--	NL	4.328		
18	60	14	20.0	--	--	--	64.7	64.1	2160.0	1660.8	0.96	1.00	0.800	0.800	0.188	--	NL	4.244		
19	60	8	20.0	--	--	--	61.1	62.3	2280.0	1718.4	0.96	1.00	0.800	0.800	0.192	--	NL	4.171		
20	80	8	20.0	--	--	--	81.3	83.0	2400.0	1776.0	0.96	1.00	0.800	0.800	0.195	--	NL	4.106		
21	80	8	20.0	--	--	--	81.3	83.0	2520.0	1833.6	0.95	1.00	0.800	0.800	0.198	--	NL	4.048		
22	80	8	20.0	--	--	--	81.3	83.0	2640.0	1891.2	0.95	1.00	0.800	0.800	0.200	--	NL	3.998		
23	80	8	20.0	--	--	--	81.3	83.0	2760.0	1948.8	0.95	1.00	0.800	0.800	0.202	--	NL	3.953		
24	80	8	20.0	--	--	--	81.3	83.0	2880.0	2006.4	0.95	1.00	0.800	0.800	0.204	--	NL	3.913		
25	80	8	10.0	--	--	--	81.3	83.0	3000.0	2064.0	0.94	1.00	0.800	0.800	0.206	--	NL	3.879		
26	80	8	10.0	--	--	--	81.3	83.0	3120.0	2121.6	0.94	1.00	0.800	0.800	0.208	--	NL	3.848		
27	80	8	10.0	--	--	--	81.3	83.0	3240.0	2179.2	0.93	1.00	0.800	0.800	0.209	--	NL	3.823		
28	80	8	10.0	--	--	--	81.3	83.0	3360.0	2236.8	0.93	1.00	0.800	0.800	0.210	--	NL	3.801		
29	80	8	10.0	--	--	--	81.3	83.0	3480.0	2294.4	0.93	1.00	0.800	0.800	0.211	--	NL	3.783		
30	80	8	25.0	--	--	--	81.3	83.0	3600.0	2352.0	0.92	1.00	0.800	0.800	0.212	--	NL	3.768		
31	80	8	25.0	--	--	--	81.3	83.0	3720.0	2409.6	0.92	1.00	0.800	0.800	0.213	--	NL	3.757		
32	80	8	25.0	--	--	--	81.3	83.0	3840.0	2467.2	0.91	1.00	0.800	0.800	0.213	--	NL	3.750		
33	80	8	25.0	--	--	--	81.3	83.0	3960.0	2524.8	0.90	1.00	0.800	0.800	0.214	--	NL	3.746		
34	80	8	25.0	--	--	--	81.3	83.0	4080.0	2582.4	0.90	1.00	0.800	0.800	0.214	--	NL	3.745		
35	80	8	25.0	--	--	--	81.3	83.0	4200.0	2640.0	0.89	1.00	0.800	0.800	0.213	--	NL	3.748		
36	80	8	25.0	--	--	--	81.3	83.0	4320.0	2697.6	0.88	1.00	0.800	0.800	0.213	--	NL	3.753		
37	80	8	25.0	--	--	--	81.3	83.0	4440.0	2755.2	0.88	1.00	0.800	0.800	0.213	--	NL	3.762		
38	80	8	25.0	--	--	--	81.3	83.0	4560.0	2812.8	0.87	1.00	0.800	0.800	0.212	--	NL	3.774		
39	20	75	25.0	--	--	--	29.0	26.0	4680.0	2870.4	0.86	1.00	0.386	0.410	0.211	--	NL	1.828		
40	20	75	25.0	--	--	--	29.0	26.0	4800.0	2928.0	0.85	1.00	0.386	0.410	0.210	--	NL	1.836		
41	20	75	25.0	--	--	--	29.0	26.0	4920.0	2985.6	0.84	1.00	0.386	0.410	0.209	--	NL	1.846		
42	20	75	25.0	--	--	--	29.0	26.0	5040.0	3043.2	0.83	1.00	0.386	0.410	0.208	--	NL	1.858		
43	20	75	25.0	--	--	--	29.0	26.0	5160.0	3100.8	0.82	1.00	0.386	0.410	0.206	--	NL	1.870		
44	60	20	25.0	--	--	--	68.4	65.8	5280.0	3158.4	0.81	1.00	0.800	0.800	0.205	--	NL	3.905		
45	60	20	25.0	--	--	--	68.4	65.8	5400.0	3216.0	0.80	1.00	0.800	0.800	0.203	--	NL	3.936		
46	60	20	25.0	--	--	--	68.4	65.8	5520.0	3273.6	0.79	1.00	0.800	0.800	0.202	--	NL	3.969		
47	60	8	25.0	--	--	--	61.1	62.3	5640.0	3331.2	0.78	1.00	0.800	0.800	0.200	--	NL	4.004		
48	60	8	25.0	--	--	--	61.1	62.3	5760.0	3388.8	0.77	1.00	0.800	0.800	0.198	--	NL	4.042		
49	60	8	25.0	--	--	--	61.1	62.3	5880.0	3446.4	0.76	1.00	0.800	0.800	0.196	--	NL	4.081		
50	60	8	25.0	--	--	--	61.1	62.3	6000.0	3504.0	0.75	1.00	0.800	0.800	0.194	--	NL	4.121		
51	60	8	25.0	--	--	--	61.1	62.3	6120.0	3561.6	0.74	1.00	0.800	0.800	0.192	--	NL	4.163		

Total Settlement, S_{LIQ} (in.) = 0.79

Total Liquifiable Layers = 3



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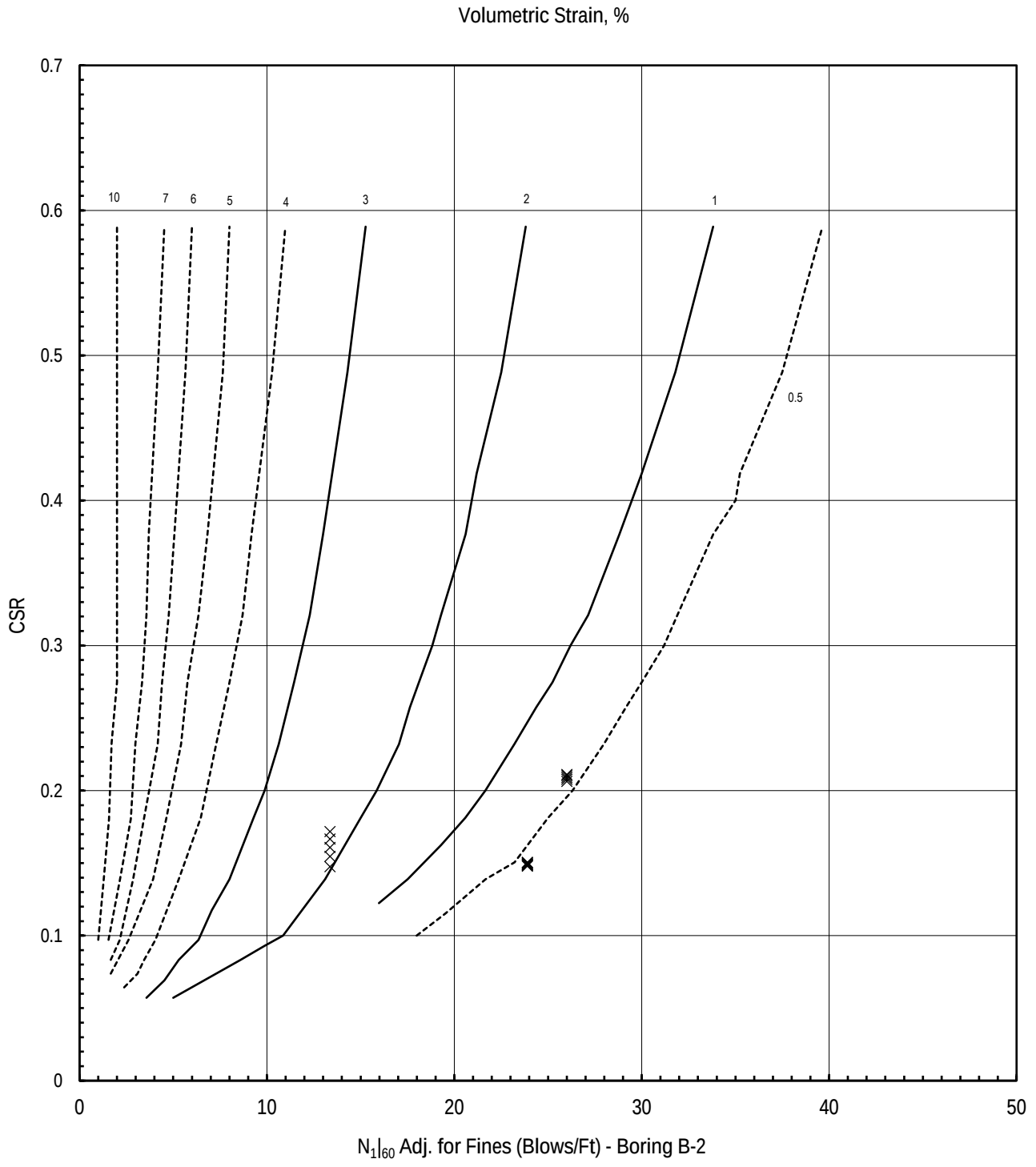
SAN DIEGO, CALIFORNIA

PROJECT NO. G2129-52-03



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PROJECT NO. G2129-52-03

APPENDIX E

APPENDIX E

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the *2018 City of San Diego Storm Water Standards* (SWS). If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

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

TABLE E-I
HYDROLOGIC SOIL GROUP DEFINITIONS

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

The property is designated as Tujunga Sand (TuB) and Visalia Gravelly Sandy Laom (VbB) which are classified as Soil Group A with a saturated hydraulic conductivity rate of greater than 1.42 inches per hour.

In Situ Testing

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

TABLE E-II
SOIL PERMEABILITY DEFINITIONS

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

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

We performed 6 Aardvark Permeameter tests at on the property. The results of the tests provide parameters regarding the saturated hydraulic conductivity and infiltration characteristics of on-site soil and geologic units. Table E-III presents the results of the estimated field saturated hydraulic conductivity and estimated infiltration rates obtained from the Aardvark Permeameter tests. The field sheets are also attached herein. Based on a discussion in the *SWS*, the infiltration rate should be considered equal to the saturated hydraulic conductivity rate. We applied a feasibility factor of safety of 2.0 to our estimated infiltration rates to provide input on Worksheet C.4-1. Soil infiltration rates from in-situ tests can vary significantly from one location to another due to the heterogeneous characteristics inherent to most soil. The Geologic Map, Figure 2 presents the locations of the permeability tests.

TABLE E-III
FIELD PERMEAMETER INFILTRATION TEST RESULTS

Test No.	Geologic Unit	Test Elevation (feet, MSL)	Field-Saturated Hydraulic Conductivity/Infiltration Rate, k_{sat} (inch/hour)	Worksheet Infiltration Rate[1] (inch/hour)
P-1	Qal	15	0.30	0.15
P-2	Qal	13	0.25	0.13
P-3	Qal	13	0.14	0.07
P-4	Qal	14	0.13	0.07
P-5	Qal	14	0.20	0.10
P-6	Qal	16	0.12	0.06
Average			0.19	0.10

[1] Using a factor of safety of 2.

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

TABLE E-IV
INFILTRATION CATEGORIES

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

*Using a Factor of Safety of 2.

Based on our observations and test results, the infiltration rates for the alluvium are less than 0.5 inches per hour. Therefore, full infiltration on the property should be considered infeasible based on the calculated infiltrations rates. However, partial infiltration may be considered feasible within the alluvium based on the infiltration rates (greater than 0.05 inches per hour including a factor of safety of 2). Vertical cutoff walls or liners should be installed on the sides and a drain should be installed at the base of the basin.

Soil Types

Undocumented Fill and Topsoil – Undocumented fill and topsoil exists on the property to depths of up to 3 feet. The undocumented fill was not tested or observed during placement and should be considered to be highly variable on the property and within adjacent properties and right-of-ways. The

undocumented fill and topsoil materials should be considered to possess relatively high hydroconsolidation characteristics. Water that is allowed to migrate within the undocumented fill and topsoil materials soil cannot be controlled, would destabilize support for the existing improvements, and would shrink and swell. Therefore, full and partial infiltration should not be allowed within the undocumented fill or topsoil materials. The planned storm water devices should be extended to expose alluvium at the base and liners should be installed on the sides of the devices to prevent lateral water migration into the fill and topsoil.

Alluvium – The alluvium generally consists of loose to very dense, dark brown to grayish-brown, mixed sands, silts and clays with trace gravels and cobbles. Based on the results of our in-situ testing, the infiltration rates within the alluvium ranges from 0.12 to 0.30 inches/hour (0.06 to 0.15 including a factor of safety of 2). Based on our conversations with the City of San Diego, we expect a partial infiltration condition exists on the property.

Compacted Fill – The proposed compacted fill will be comprised of on-site materials that will consist predominantly of sand, silt and clay. The fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density. Based on our limited investigation and background research, sufficient materials do not exist on-site to perform select grading to allow infiltration within fill materials. In our experience, compacted fill using the on-site materials does not possess infiltration rates appropriate with infiltration and the water would destabilize the existing fill causing distress to existing and proposed improvements. Therefore, full and partial infiltration should be considered infeasible. Mitigation measures would include extending the infiltration devices into the alluvial materials and lining the storm water devices.

Groundwater Elevations

The SWS indicates that the depth to the groundwater table beneath an infiltration BMP must be greater than 10 feet for infiltration to be allowed. We encountered groundwater at depths of 13.5 and 16 feet in current Borings B-1 and B-2 (elevations of about 6.5 and 4 feet above Mean Sea Level). In addition, CWE reported that groundwater was encountered at depths ranging from 8 to 13 feet (elevations of about 7 to 12 feet above Mean Sea Level). We also encountered saturated soils at depths of approximately 5 feet in current Borings B-1 and B-2 (elevation of approximately 15 feet above MSL). The project should be designed with a groundwater elevation of 10 feet MSL.

There is likely not enough vertical space between planned bottom of basin elevations and 10 feet above the groundwater elevation; therefore, full and partial infiltration should be considered infeasible. Mitigation measures to lower the groundwater elevation are not feasible or reasonable for the planned development.

New or Existing Utilities

Utilities are located adjacent to the property on the eastern property boundaries and existing streets. Therefore, full infiltration within the areas near these utilities should be considered infeasible. Setbacks for infiltration should be incorporated. The setback for infiltration devices should be a minimum of 10 feet and a 1:1 plane of 1 foot below the closest edge of the deepest adjacent utility. Partial infiltration may be feasible if liners or cut-off walls will be installed on the sidewalls of the proposed devices.

Existing or Planned Structures

Structures exist on the southern property line of the subject project. Water should not be allowed to infiltrate in areas where it could affect the neighboring properties and adjacent structures. Infiltration should be considered infeasible due to the lateral migration characteristics of the soil. Mitigation for existing structures consists of not allowing water infiltration within 10 feet of the existing foundations.

Soil or Groundwater Contamination

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

Slopes and Other Geologic Hazards

As previously discussed, an existing slope associated with the Otay River is located adjacent to the northern property line. Water migration and the resulting seepage forces negatively affects the stability of slopes and causes erosion. The SWS recommends a minimum setback of 50 feet from the top of existing sensitive slopes. Due to the potential for lateral water migration within the existing soil, full or partial infiltration should be considered infeasible within this setback zone from the existing slopes.

The County of San Diego Hazard Mitigation Plan (2017) maps the site in a liquefiable area. The liquefaction analysis (included in Appendix D of this report) indicates the onsite soils to depths of approximately 20 feet below proposed grade could be prone to up to approximately 2 inches of total liquefaction settlement during PGA_M ground motion. Table C.5-1 pf the 2018 Storm Water Standards (SWS) states BMPs (particularly infiltration BMPs) must not be sited in areas with high potential for liquefaction or landslides to minimize earthquake/landslide risks. Therefore, full and partial infiltration devices should be considered infeasible for the property. We expect the planned mitigation measures for liquefaction would include supporting the planned structures on post-tensioned or mat foundations.

Storm Water Evaluation Narrative

The site is underlain by fill soils, topsoil and alluvium to depths of approximately 35 feet overlying Old Paralic Deposits and San Diego Formation. We performed 6 infiltration tests within the alluvium and the results indicate rates less than 0.5 inches per hour (with an applied factor of safety of 2). Therefore, full infiltration is considered infeasible within the alluvium.

The project area is mapped within a liquefaction zone. In addition, our calculations show a potential for liquefaction exists within the alluvium underlying the property. Therefore, infiltration should be considered infeasible to help prevent an increased thickness of liquefiable soil. In addition, groundwater exists at depths ranging from approximately 8 and 16 feet below the existing ground surface (approximate elevations ranging from 4 and 12 feet MSL). The elevation where infiltration is feasible is limited to the required 10 feet above the groundwater elevation. There is likely not enough vertical space between planned bottom of basin elevations and 10 feet above the groundwater elevation. Therefore, full and partial infiltration devices should be considered infeasible for the property.

Storm Water Evaluation Conclusion

Due to the liquefaction potential at the site and the depth of the groundwater relative to the bottom of planned storm water devices, infiltration should be considered infeasible and planned storm water device should be lined.

Storm Water Management Devices

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

Storm Water Standard Worksheets

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

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

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

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

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

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

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	2	0.50
Site Soil Variability	0.25	3	0.75
Depth to Groundwater/ Impervious Layer	0.25	2	0.50
Suitability Assessment Safety Factor, $S_A = \sum p$			2.25

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]
Part 1 - Full Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
Bella Mar – 408 Hollister Street	Design
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[11]? ☒ 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.

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.

[10] 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.

[11] Available data include 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[10]
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.

Based on the USGS Soil Survey, the property possesses a Hydrologic Soil Group A classification. In addition, we encountered field infiltration rates of:

P-1: 0.30 inches/hour (0.15 with a FOS of 2.0)
P-2: 0.25 inches/hour (0.13 with a FOS of 2.0)
P-3: 0.14 inches/hour (0.07 with a FOS of 2.0)
P-4: 0.13 inches/hour (0.07 with a FOS of 2.0)
P-5: 0.20 inches/hour (0.10 with a FOS of 2.0)
P-6: 0.12 inches/hour (0.06 with a FOS of 2.0)

This results in an average infiltration rate of 0.19 inches/hour (0.10 with a FOS of 2.0).

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
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[10]	
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[10]	
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.

The site is underlain by fill soils, topsoil and alluvium to depths of approximately 35 feet overlying Old Paralic Deposits and San Diego Formation. We performed 6 infiltration tests within the alluvium and the results indicate rates less than 0.5 inches per hour (with an applied factor of safety of 2). Therefore, full infiltration is considered infeasible within the alluvium.

The project area is mapped within a liquefaction zone. In addition, our calculations show a potential for liquefaction exists within the alluvium underlying the property. Therefore, infiltration should be considered infeasible to help prevent an increased thickness of liquefiable soil. In addition, groundwater exists at depths ranging from approximately 8 and 16 feet below the existing ground surface (approximate elevations ranging from 4 and 12 feet MSL). The elevation where infiltration is feasible is limited to the required 10 feet above the groundwater elevation. There is likely not enough vertical space between planned bottom of basin elevations and 10 feet above the groundwater elevation. Therefore, full and partial infiltration devices should be considered infeasible for the property.

Part 1 Result – Full Infiltration Geotechnical Screening[12]	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

[12] 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[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
Bella Mar – 408 Hollister Street	Design

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).

Based on the USGS Soil Survey, the property possesses a Hydrologic Soil Group A classification. In addition, we encountered field infiltration rates of:

P-1: 0.30 inches/hour (0.15 with a FOS of 2.0)
P-2: 0.25 inches/hour (0.13 with a FOS of 2.0)
P-3: 0.14 inches/hour (0.07 with a FOS of 2.0)
P-4: 0.13 inches/hour (0.07 with a FOS of 2.0)
P-5: 0.20 inches/hour (0.10 with a FOS of 2.0)
P-6: 0.12 inches/hour (0.06 with a FOS of 2.0)

This results in an average infiltration rate of 0.19 inches/hour (0.10 with a FOS of 2.0).

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 4B. 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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
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
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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I- 8A[10]	
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

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

The site is underlain by fill soils, topsoil and alluvium to depths of approximately 35 feet overlying Old Paralic Deposits and San Diego Formation. We performed 6 infiltration tests within the alluvium and the results indicate rates less than 0.5 inches per hour (with an applied factor of safety of 2). Therefore, full infiltration is considered infeasible within the alluvium.

The project area is mapped within a liquefaction zone. In addition, our calculations show a potential for liquefaction exists within the alluvium underlying the property. Therefore, infiltration should be considered infeasible to help prevent an increased thickness of liquefiable soil. In addition, groundwater exists at depths ranging from approximately 8 and 16 feet below the existing ground surface (approximate elevations ranging from 4 and 12 feet MSL). The elevation where infiltration is feasible is limited to the required 10 feet above the groundwater elevation. There is likely not enough vertical space between planned bottom of basin elevations and 10 feet above the groundwater elevation. Therefore, full and partial infiltration devices should be considered infeasible for the property.

Part 2 – Partial Infiltration Geotechnical Screening Result[13]	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

[13] 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.

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	408 Hollister Street	Date:	4/26/2017
Project Number:	G2129-52-01	By:	JML
Borehole Location:	P-1	Ref. EL (feet, MSL):	19.0
		Bottom EL (feet, MSL):	15.0

Borehole Diameter, d (in.):	4.50
Borehole Depth, **H** (feet):	4.00
Distance Between Reservoir & Top of Borehole (in.):	30.00
Depth to Water Table, **s** (feet):	10.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Wetted Area, **A** (in^2): 86.59

Distance Between Resevoir and APM Float, **D** (in.):	72.25
Head Height Calculated, **h** (in.):	4.74
Head Height Recorded, **h** (in.):	5.00
Distance Between Constant Head and Water Table, **L** (in.):	76.74

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0			17.305			
2	5	5.00		17.130	0.175	0.175	0.969
3	10	5.00		16.840	0.290	0.465	1.605
4	15	5.00		16.650	0.190	0.655	1.052
5	20	5.00		16.480	0.170	0.825	0.941
6	25	5.00		16.295	0.185	1.010	1.024
7	30	5.00		16.115	0.180	1.190	0.996
8	35	5.00		15.835	0.280	1.470	1.550
9	40	5.00		15.680	0.155	1.625	0.858
10	45	5.00		15.520	0.160	1.785	0.886
11	50	5.00		15.360	0.160	1.945	0.886
12							
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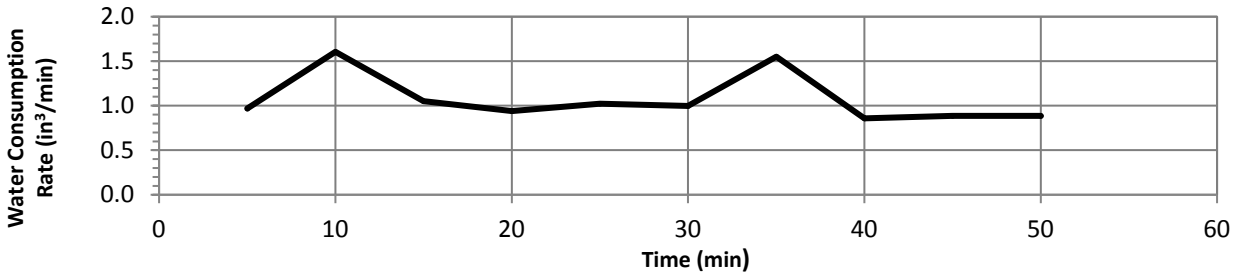
28							

Steady Flow Rate, Q (in^3/min): 0.886



Field-Saturated Hydraulic Conductivity (Infiltration Rate)

Case 1: L/h > 3 K_{sat} = 5.03E-03 in/min 0.302 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	408 Hollister Street	Date:	4/26/2017
Project Number:	G2129-52-01	By:	JML
Borehole Location:	P-2	Ref. EL (feet, MSL):	17.0
		Bottom EL (feet, MSL):	13.0

Borehole Diameter, d (in.):	4.50
Borehole Depth, **H** (feet):	4.00
Distance Between Reservoir & Top of Borehole (in.):	30.00
Depth to Water Table, **s** (feet):	10.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Wetted Area, **A** (in^2): 86.59

Distance Between Resevoir and APM Float, **D** (in.):	72.25
Head Height Calculated, **h** (in.):	4.74
Head Height Recorded, **h** (in.):	5.00
Distance Between Constant Head and Water Table, **L** (in.):	76.74

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0			21.135			
2	5	5.00		20.890	0.245	0.245	1.356
3	10	5.00		20.685	0.205	0.450	1.135
4	15	5.00		20.480	0.205	0.655	1.135
5	20	5.00		20.285	0.195	0.850	1.079
6	25	5.00		20.120	0.165	1.015	0.913
7	30	5.00		19.965	0.155	1.170	0.858
8	35	5.00		19.925	0.040	1.210	0.221
9	40	5.00		19.865	0.060	1.270	0.332
10	45	5.00		19.750	0.115	1.385	0.637
11	50	5.00		19.620	0.130	1.515	0.720
12	55	5.00		19.490	0.130	1.645	0.720
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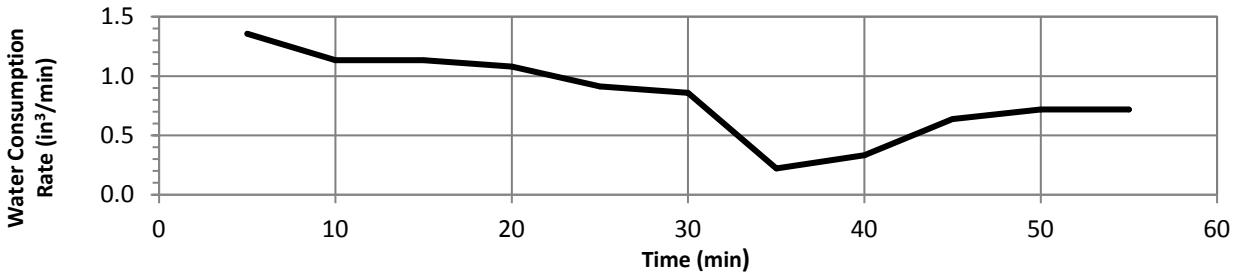
Steady Flow Rate, Q (in^3/min): 0.720



Field-Saturated Hydraulic Conductivity (Infiltration Rate)

Case 1: L/h > 3 K_{sat} = 4.09E-03 in/min 0.245 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	408 Hollister Street	Date:	4/26/2017
Project Number:	G2129-52-01	By:	JML
Borehole Location:	P-3	Ref. EL (feet, MSL):	17.0
		Bottom EL (feet, MSL):	13.0

Borehole Diameter, d (in.):	4.50
Borehole Depth, **H** (feet):	4.00
Distance Between Reservoir & Top of Borehole (in.):	30.00
Depth to Water Table, **s** (feet):	10.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Wetted Area, **A** (in^2): 86.59

Distance Between Resevoir and APM Float, **D** (in.):	72.25
Head Height Calculated, **h** (in.):	4.74
Head Height Recorded, **h** (in.):	5.00
Distance Between Constant Head and Water Table, **L** (in.):	76.74

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0			18.215			
2	5	5.00		18.060	0.155	0.155	0.858
3	10	5.00		17.930	0.130	0.285	0.720
4	15	5.00		17.855	0.075	0.360	0.415
5	20	5.00		17.780	0.075	0.435	0.415
6	25	5.00		17.715	0.065	0.500	0.360
7	30	5.00		17.630	0.085	0.585	0.471
8	35	5.00		17.555	0.075	0.660	0.415
9	40	5.00		17.480	0.075	0.735	0.415
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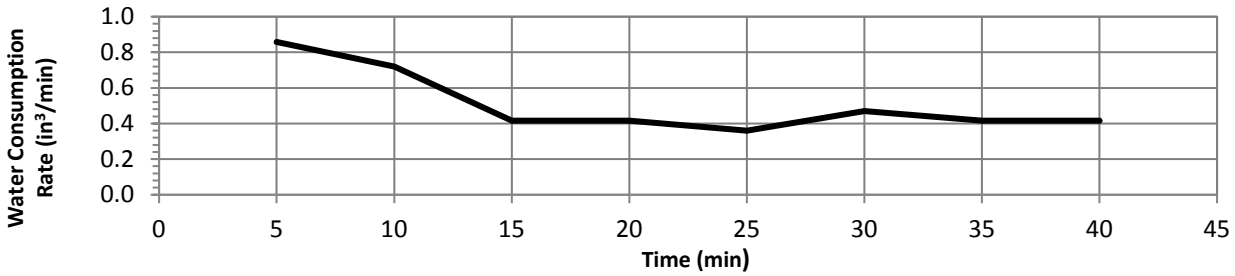
Steady Flow Rate, Q (in^3/min): 0.415



Field-Saturated Hydraulic Conductivity (Infiltration Rate)

Case 1: L/h > 3 K_{sat} = 2.36E-03 in/min 0.142 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	408 Hollister Street	Date:	4/26/2017
Project Number:	G2129-52-01	By:	JML
Borehole Location:	P-4	Ref. EL (feet, MSL):	18.0
		Bottom EL (feet, MSL):	14.0

Borehole Diameter, d (in.):	4.50
Borehole Depth, **H** (feet):	4.00
Distance Between Reservoir & Top of Borehole (in.):	30.00
Depth to Water Table, **s** (feet):	10.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Wetted Area, **A** (in^2): 86.59

Distance Between Resevoir and APM Float, **D** (in.):	72.25
Head Height Calculated, **h** (in.):	4.74
Head Height Recorded, **h** (in.):	5.00
Distance Between Constant Head and Water Table, **L** (in.):	76.74

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0			20.620			
2	5	5.00		20.515	0.105	0.105	0.581
3	10	5.00		20.440	0.075	0.180	0.415
4	15	5.00		20.365	0.075	0.255	0.415
5	20	5.00		20.290	0.075	0.330	0.415
6	25	5.00		20.270	0.020	0.350	0.111
7	30	5.00		20.245	0.025	0.375	0.138
8	35	5.00		20.170	0.075	0.450	0.415
9	40	5.00		20.100	0.070	0.520	0.388
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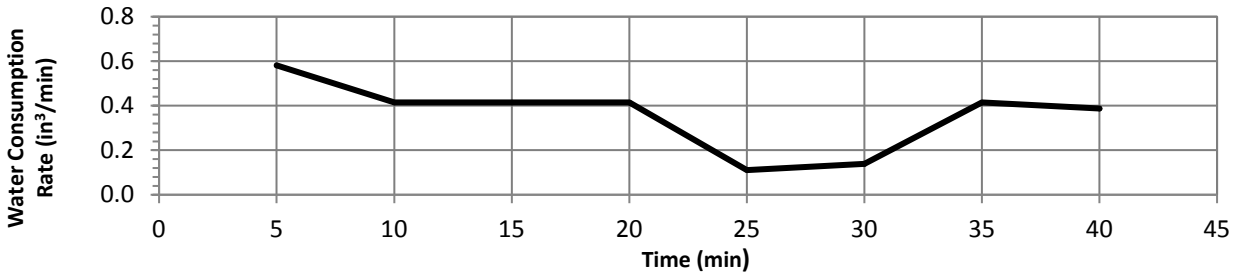
Steady Flow Rate, Q (in^3/min): 0.388



Field-Saturated Hydraulic Conductivity (Infiltration Rate)

Case 1: L/h > 3 K_{sat} = 2.20E-03 in/min 0.132 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	408 Hollister Street	Date:	4/26/2017
Project Number:	G2129-52-01	By:	JML
Borehole Location:	P-5	Ref. EL (feet, MSL):	18.0
		Bottom EL (feet, MSL):	14.0

Borehole Diameter, d (in.):	4.50
Borehole Depth, **H** (feet):	4.00
Distance Between Reservoir & Top of Borehole (in.):	30.00
Depth to Water Table, **s** (feet):	10.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Wetted Area, **A** (in^2): 86.59

Distance Between Resevoir and APM Float, **D** (in.):	72.25
Head Height Calculated, **h** (in.):	4.74
Head Height Recorded, **h** (in.):	5.00
Distance Between Constant Head and Water Table, **L** (in.):	76.74

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0			19.590			
2	5	5.00		19.355	0.235	0.235	1.301
3	10	5.00		19.235	0.120	0.355	0.664
4	15	5.00		19.110	0.125	0.480	0.692
5	20	5.00		19.020	0.090	0.570	0.498
6	25	5.00		18.905	0.115	0.685	0.637
7	30	5.00		18.835	0.070	0.755	0.388
8	35	5.00		18.730	0.105	0.860	0.581
9	40	5.00		18.625	0.105	0.965	0.581
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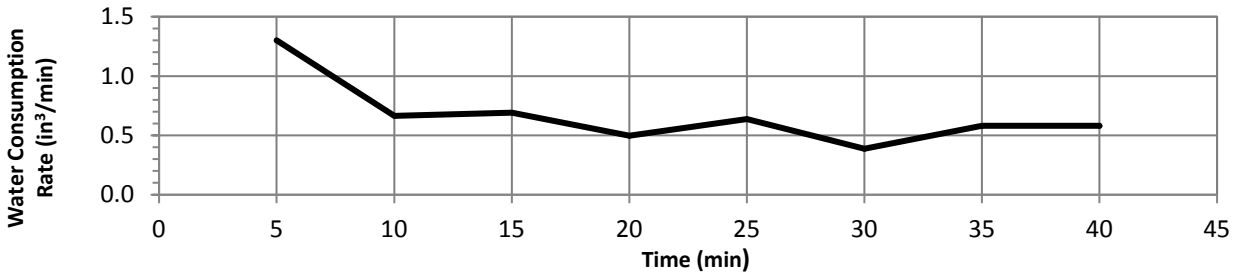
Steady Flow Rate, Q (in^3/min): 0.581



Field-Saturated Hydraulic Conductivity (Infiltration Rate)

Case 1: L/h > 3 K_{sat} = 3.30E-03 in/min 0.198 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	408 Hollister Street	Date:	4/26/2017
Project Number:	G2129-52-01	By:	JML
Borehole Location:	P-6	Ref. EL (feet, MSL):	20.0
		Bottom EL (feet, MSL):	16.0

Borehole Diameter, d (in.):	4.50
Borehole Depth, **H** (feet):	4.00
Distance Between Reservoir & Top of Borehole (in.):	30.00
Depth to Water Table, **s** (feet):	10.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Wetted Area, **A** (in^2): 86.59

Distance Between Resevoir and APM Float, **D** (in.):	72.25
Head Height Calculated, **h** (in.):	4.74
Head Height Recorded, **h** (in.):	5.00
Distance Between Constant Head and Water Table, **L** (in.):	76.74

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0			20.995			
2	5	5.00		20.930	0.065	0.065	0.360
3	10	5.00		20.845	0.085	0.150	0.471
4	15	5.00		20.770	0.075	0.225	0.415
5	20	5.00		20.690	0.080	0.305	0.443
6	25	5.00		20.615	0.075	0.380	0.415
7	30	5.00		20.550	0.065	0.445	0.360
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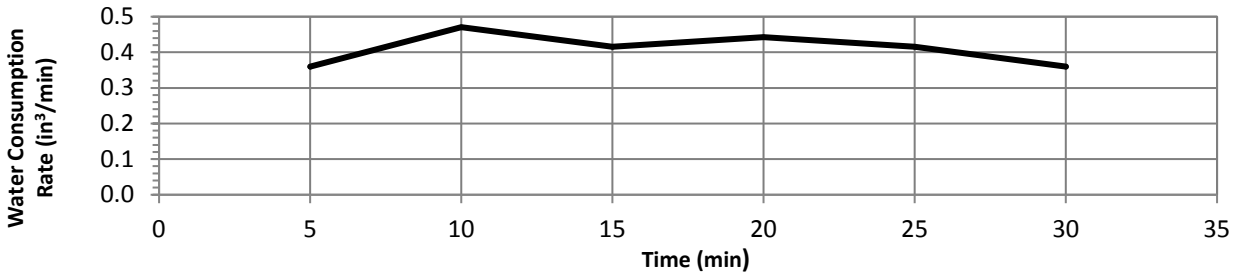
Steady Flow Rate, Q (in^3/min): 0.360



Field-Saturated Hydraulic Conductivity (Infiltration Rate)

Case 1: L/h > 3 K_{sat} = 2.04E-03 in/min 0.123 in/hr

APPENDIX F

APPENDIX F

RECOMMENDED GRADING SPECIFICATIONS

FOR

BELLA MAR
408 HOLLISTER STREET
SAN DIEGO, CALIFORNIA

PROJECT NO. G2129-52-03

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9

and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

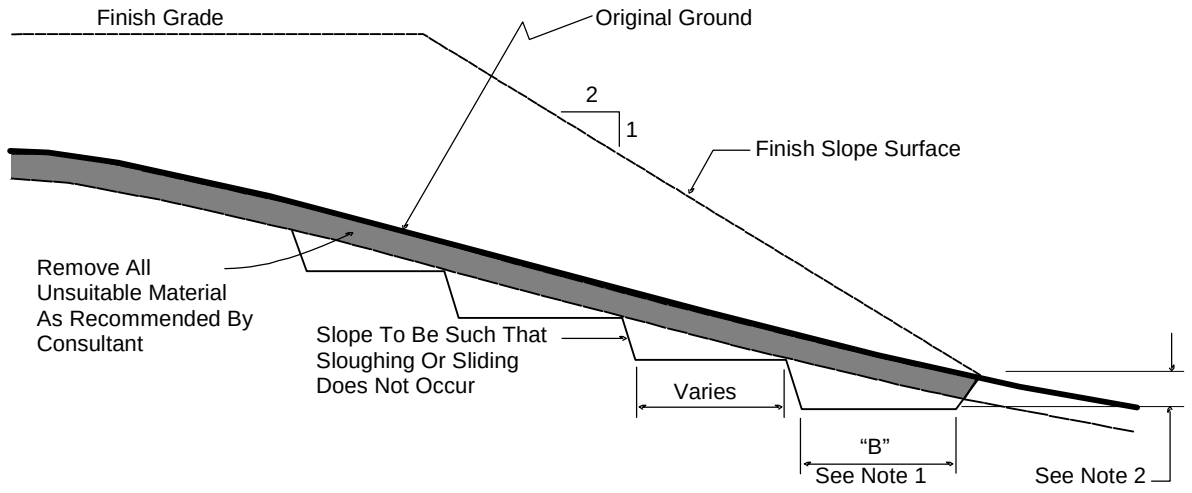
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



No Scale

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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection

variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

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

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

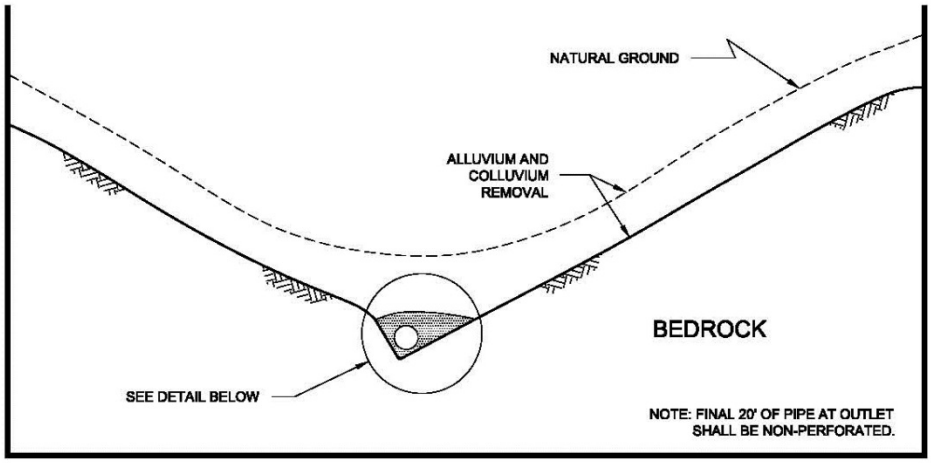
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

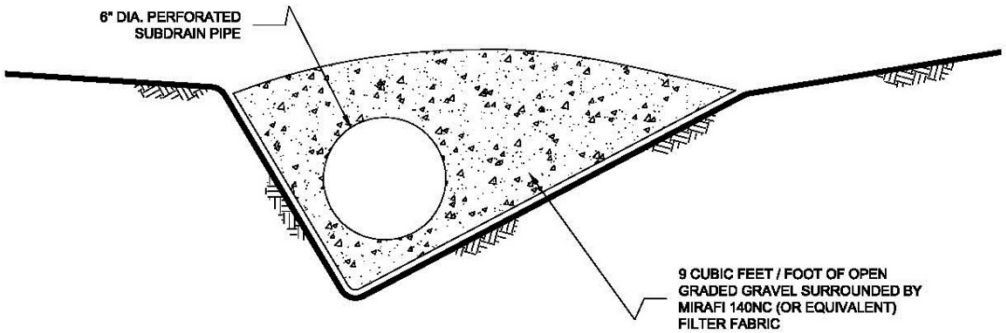
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

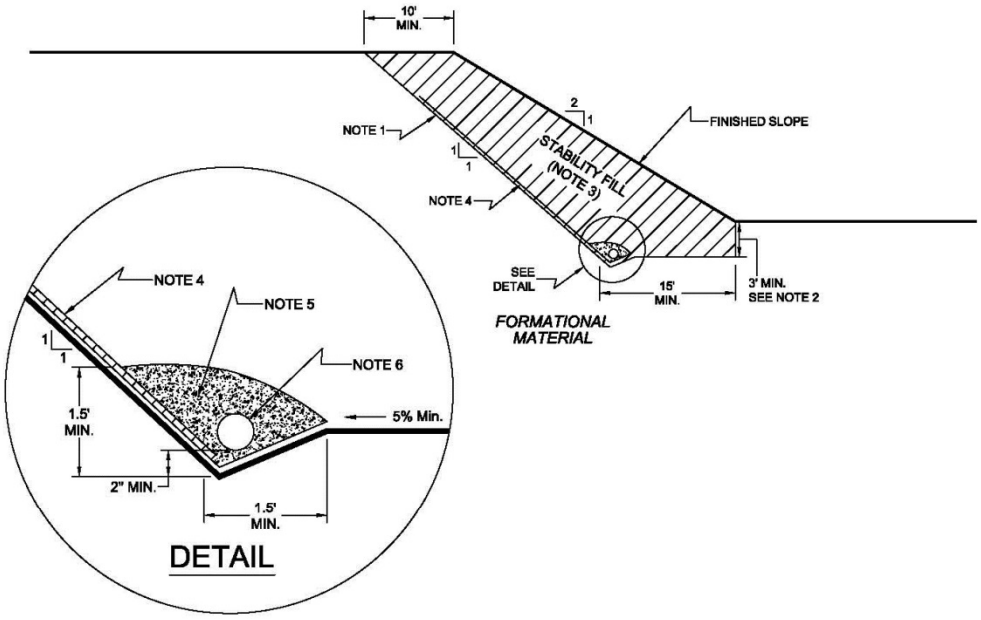
1.....6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS
IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

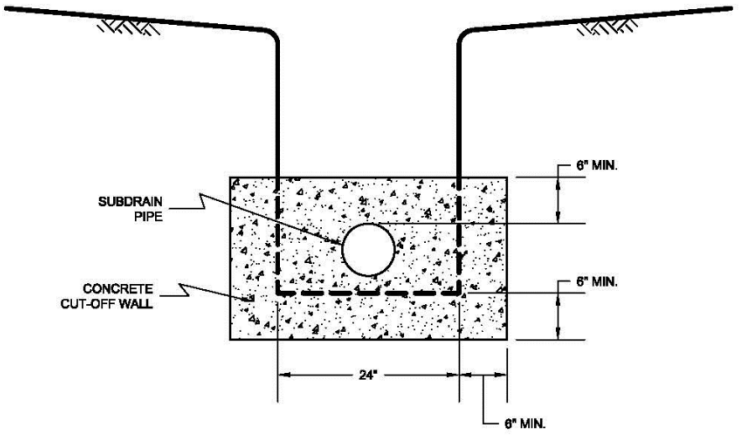
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

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

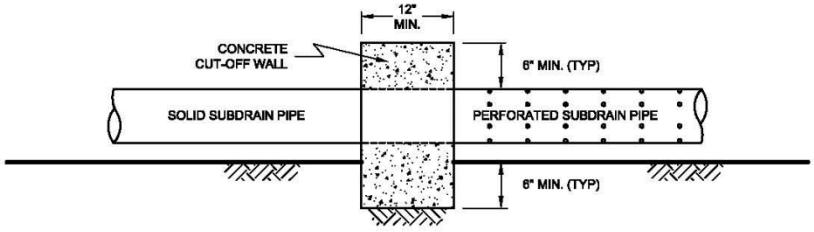
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW

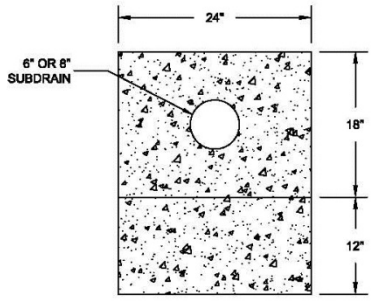


NO SCALE

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

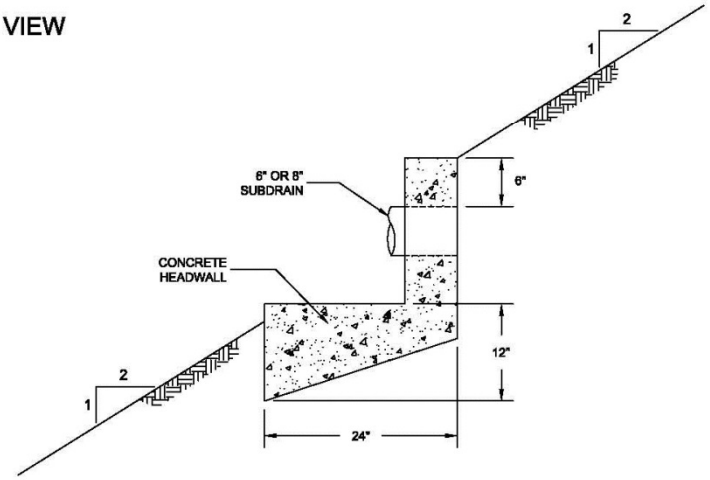
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



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

NO SCALE

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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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

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