STORM WATER QUALITY MANAGEMENT PLAN

HOME DEPOT MISSION VALLEY

PREPARED FOR

HOME DEPOT U.S.A., INC.
4000 W. METROPOLITAN DR., SUITE 100
ORANGE, CA 92868
C/O BOB BURNSIDE
(714) 940-3549

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

BRYAN D. SMITH P.E.
PROJECT MANAGER

DATE PREPARED: 08.12.2020

PROJECT#: 0128-020-01

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

Home Depot-Scottish Rite (Mission Valley)

PTS 657591

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

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

Engineer of Work:

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

Bryan D. Smith RCE 75822 EXP 06-30-22
Provide Wet Signature and Stamp Above Line

Prepared For:

Home Depot U.S.A., Inc.
4000 W. Metropolitan Dr., Suite 100
Orange, CA 92868
c/o Bob Burnside (714) 940-3549

Prepared By:

Fuscoe Engineering, Inc.
6390 Greenwich Drive, Suite 170
San Diego, CA 92122
(858) 554-1500

Date:

August 12, 2020

Approved by: City of San Diego Date

The City of SAN DIEGO

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Table of Contents

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

Acronyms

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

Certification Page

Project Name:
Permit Application

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

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

Engineer of Work's Signature

75822

PE#

6-30-2022

Expiration Date

Bryan D. Smith

Print Name

Fuscoe Engineering, Inc.

Company

5/26/20

Date

Engineer's Stamp

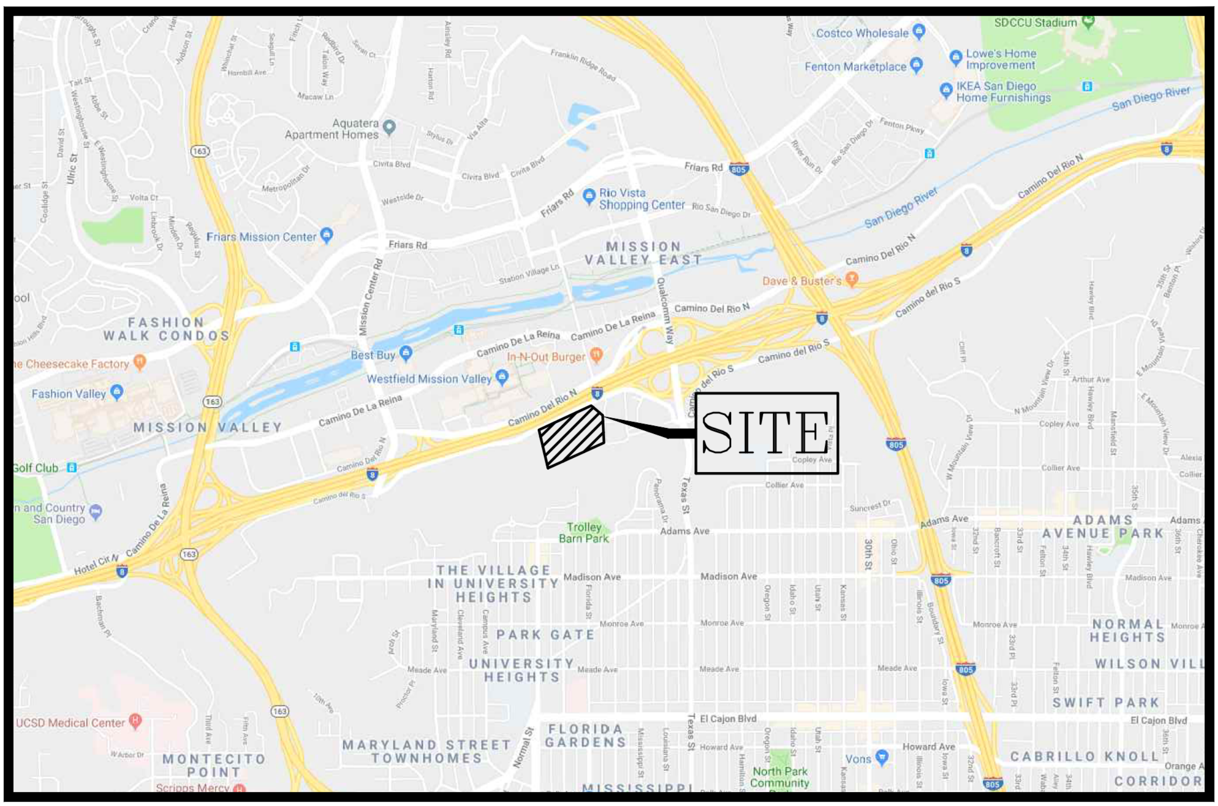
Submittal Record

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

Submittal Number	Date	Project Status	Changes
1	01/31/2020	☑ Preliminary Design/Planning/CEQA ☐ Final Design	**Initial Submittal**
2	04/22/2020	☑ Preliminary Design/Planning/CEQA ☐ Final Design	2nd Submittal
3		☐ Preliminary Design/Planning/CEQA ☐ Final Design	
4		☐ Preliminary Design/Planning/CEQA ☐ Final Design	

Project Vicinity Map

Project Name: Home Depot-Scottish Rite (Mission Valley)
Permit Application 657591



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

Attach DS-560 form.

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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: 1895 Camino del Rio South, San Diego CA 92108	Project Number: 657591

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

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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	Sr. Project Manager (Civil)
Name of Owner or Agent *(Please Print)*	Title
BDS	05/26/2020
Signature	Date

Clear Page 4

Clear Form

Applicability of Permanent, Post-Construction Storm Water BMP Requirements		Form I-1
Project Identification		
Project Name: Home Depot- Scottish Rite (Mission Valley)		
Permit Application Number: 657591		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:		

Form I-1 Page 2 of 2		
Step	**Answer**	**Progression**
Step 3. Is the project subject to earlier PDP requirements due to a prior lawful approval? See Section 1.10 of the manual (Part 1 of Storm Water Standards) for guidance.	☐ Yes	Consult the City Engineer to determine requirements. Provide discussion and identify requirements below. Go to **Step 4**.
	☑ No	BMP Design Manual PDP requirements apply. Go to **Step 4**.
Discussion / justification of prior lawful approval, and identify requirements (<u>not required if prior lawful approval does not apply</u>):		
Step 4. Do hydromodification control requirements apply? See Section 1.6 of the manual (Part 1 of Storm Water Standards) for guidance.	☐ Yes	PDP structural BMPs required for pollutant control (Chapter 5) and hydromodification control (Chapter 6). Go to **Step 5**.
	☑ No	**Stop**. PDP structural BMPs required for pollutant control (Chapter 5) only. Provide brief discussion of exemption to hydromodification control below.
Discussion / justification if hydromodification control requirements do <u>not</u> apply: This project connects to a City of San Diego storm drain system (MS4) which directly discharges in to a San Diego River reach (downstream of confluence with San Vicente Creek) which is exempt from hydromodification requirements per the WMAA.		
Step 5. Does protection of critical coarse sediment yield areas apply? See Section 6.2 of the manual (Part 1 of Storm Water Standards) for guidance.	☐ Yes	Management measures required for protection of critical coarse sediment yield areas (Chapter 6.2). **Stop**.
	☑ No	Management measures not required for protection of critical coarse sediment yield areas. Provide brief discussion below. **Stop**.
Discussion / justification if protection of critical coarse sediment yield areas does <u>not</u> apply: CCSYs are located on the undeveloped slopes adjacent to the project to the south but CCSYs are not located in the proposed construction areas of the project.		

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.

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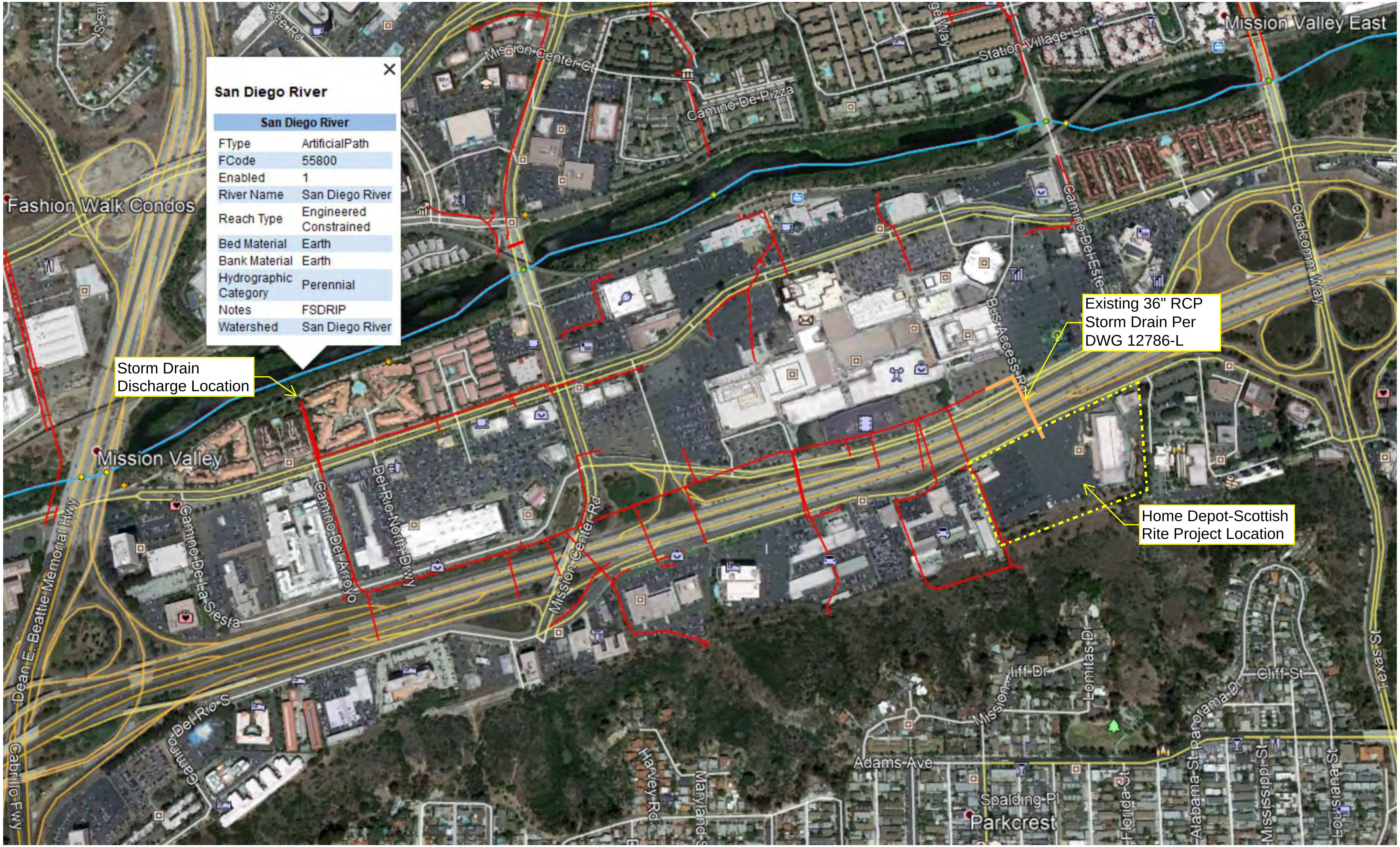
THE HOME DEPOT-SCOTTISH RITE
HMP -EXEMPTION EXHIBIT
(Direct discharge into San Diego River)



Constructed storm drain per City GIS records. Hmod-exempt storm drain system per WMAA.

Constructed storm drain per As-Builts. Ties into exempt storm drain system.

Project Name: Home Depot-Scottish Rite (Mission Valley)

Site Information Checklist For PDPs	Form I-3B
Project Summary Information	
Project Name	Home Depot-Scottish Rite Project
Project Address	Home Depot: 1895 Camino del Rio South, San Diego CA, 92108 Scottish Rite: 1561 Camino del Rio South, San Diego CA, 92108
Assessor's Parcel Number(s) (APN(s))	43809033000 and 4380903400
Permit Application Number	657591
Project Watershed	Select One: ☐ San Dieguito River ☐ Penasquitos ☐ Mission Bay ☑ San Diego River ☐ San Diego Bay ☐ Tijuana River
Hydrologic subarea name with Numeric Identifier up to two decimal places (9XX.XX)	Mission San Diego 907.11
Project Area (total area of Assessor's Parcel(s) associated with the project or total area of the right-of-way)	14.05 Acres (612018 Square Feet)
Area to be disturbed by the project (Project Footprint)	11.95 Acres (520823 Square Feet)
Project Proposed Impervious Area (subset of Project Footprint)	7.99 Acres (348198 Square Feet)
Project Proposed Pervious Area (subset of Project Footprint)	3.96 Acres (172625 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	-7 %

Project Name: Home Depot-Scottish Rite (Mission Valley)

Form I-3B Page 2 of 11
Description of Existing Site Condition and Drainage Patterns
Current Status of the Site (select all that apply): ☑Existing development ☐Previously graded but not built out ☐Agricultural or other non-impervious use ☐Vacant, undeveloped/natural Description / Additional Information: The site of the proposed Home Depot is the current Scottish Rite Center. The site of the proposed Scottish Rite Center is currently an auto dealership.
Existing Land Cover Includes (select all that apply): ☑Vegetative Cover ☐Non-Vegetated Pervious Areas ☑Impervious Areas Description / Additional Information: The main area to be redeveloped includes buildings and parking areas. There are vegetated terraced slopes at the southern border of the project.
Underlying Soil belongs to Hydrologic Soil Group (select all that apply): ☑NRCS Type A ☐NRCS Type B ☐NRCS Type C ☐NRCS Type D
Approximate Depth to Groundwater: ☐Groundwater Depth < 5 feet ☐5 feet < Groundwater Depth < 10 feet ☐10 feet < Groundwater Depth < 20 feet ☑Groundwater Depth > 20 feet
Existing Natural Hydrologic Features (select all that apply): ☑Watercourses ☐Seeps ☐Springs ☐Wetlands ☑None Description / Additional Information: The proposed redeveloped area does not consist of any natural hydrologic features. The slope areas upstream and along the southern border of the site indicate a natural watercourses.

Project Name: Home Depot-Scottish Rite (Mission Valley)

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
Drainage relative to this site generally consists of three discharge locations into the Camino del Rio South drainage system. WEST Discharge: this discharge includes an offsite upstream area which is both developed and natural slope. Runoff from this offsite area is collected in a storm drain and routed along the western boundary and ties into the existing storm drain at Camino del Rio South. Drainage from the existing auto dealership is collected locally and directly discharged into Camino del Rio South until it is collected by curb inlet about 60 feet west of the project boundary. In addition, a portion of the current Scottish Rite parking lot sheet flows and drains into Camino del Rio South and is also captured by the aforementioned curb inlet. CENTRAL Discharge: this discharge includes an offsite upstream area which is both developed and natural slope. Runoff from this offsite area is collected in a storm drain along the rear of the current Scottish Rite parking lot and conveyed through the site. Drainage from the Scottish Rite roof and parking lot are collected locally by grates and tie into this same storm drain. Additional Scottish Rite parking lot areas sheet flow towards Camino del Rio South and are collected by an existing curb inlet in front of the site. EAST Discharge: Runoff from the rear roof and rear parking area for the Scottish Rite Center drains northerly and discharges into Camino del Sur South. Offsite areas consisting of a building and slopes also comingle with onsite flow prior to exiting the site at the northeast corner of the site. Runoff from this area drains east along the Camino del Sur South gutter before getting collected by the existing curb inlet about 480 feet east of the site.

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 project will consist of a Home Depot building and parking garage structure, a Scottish Rite Center, associated parking throughout, and pockets of landscaped areas. The drainage boundaries will be modified slightly as outlined in the Hydrology Exhibits within this SWQMP and within the project specific Drainage Study. The general drainage patterns will remain the same as existing. However, treatment BMPs will be added throughout for treating runoff prior to exiting the site. The area of the project consisting of the 10' ROW dedication for bike lane and sidewalk meets PDP exemption and does not include treatment control BMPs. The site is exempt from Hydromodification Requirements.
List/describe proposed impervious features of the project (e.g., buildings, roadways, parking lots, courtyards, athletic courts, other impervious features): The overall site covered under this report consists of both the Home Depot store and Scottish Rite Center. The proposed impervious surfaces to be constructed will consist of building roofs, parking lots, driveways, sidewalks, and a parking garage structure.
List/describe proposed pervious features of the project (e.g., landscape areas): The site consists of both the Home Depot and Scottish Rite Center. The proposed pervious features will consist of landscaped areas, and surface level biofiltration basin areas.
Does the project include grading and changes to site topography? ☑ Yes ☐ No Description / Additional Information: The proposed site will minimally alter the existing grades of the site and revise the current drainage boundaries. Onsite mitigation is proposed to offset any area and/ or flow increases as detailed in the project's drainage study.

Form I-3B Page 5 of 11
Does the project include changes to site drainage (e.g., installation of new storm water conveyance systems)? ☑ Yes ☐ No If yes, provide details regarding the proposed project site drainage conveyance network, including storm drains, concrete channels, swales, detention facilities, storm water treatment facilities, natural and constructed channels, and the method for conveying offsite flows through or around the proposed project site. Identify all discharge locations from the proposed project site along with a summary of the conveyance system size and capacity for each of the discharge locations. Provide a summary of pre and post-project drainage areas and design flows to each of the runoff discharge locations. Reference the drainage study for detailed calculations. Description / Additional Information: The existing Scottish Rite Center consists of a building and parking lot. Drainage from a portion of the parking lot is collected by an onsite grate and storm drain then conveyed towards the storm drain system along Camino del Rio South. The eastern portion of the site overland flows towards Camino del Rio South where it then flows east within the gutter and is ultimately collected by an existing curb inlet about 500 feet east of the site. The remaining portion of the site sheet flows towards Camino del Rio South before getting collected by the existing curb inlet in front of the site. The existing auto dealer site will be the future location of the new Scottish Rite Center. Drainage from the backside of the lot is collected via onsite grates and storm pipe which discharge at the from the lot and drain onto Camino del Rio South. The front of the lot sheet flows over land towards Camino del Rio South. Runoff along the Camino del Rio South gutter at this location empties into an existing curb inlet located approximately 70 feet west of the western site boundary.

Project Name: Home Depot-Scottish Rite (Mission Valley)

Form I-3B Page 6 of 11

Identify whether any of the following features, activities, and/or pollutant source areas will be present (select all that apply):

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

Description/Additional Information:

The site as a whole consisting of both the Home Depot and Scottish Rite Center sites will include the above listed potential pollutant sources due to the expected land use and activities as a retail store and community use building.

Project Name: Home Depot-Scottish Rite (Mission Valley)

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) All storm water runoff generated by the impervious surfaces at the site will be routed through a water quality treatment BMPs prior exiting the site. All runoff from the site will enter the City of San Diego MS4 storm drain system which is conveyed downstream where it will discharge into the San Diego River which is located approximately 1,700 yards due west of the site. From the MS4 discharge point into the San Diego River, flow heads west and empties into the Pacific Ocean about 5.62 miles west. The 10' ROW dedication for bike lane and sidewalk will be considered exempt under exemption section "1.4.3 Local PDP Exemptions" of the City of San Diego Storm Water Standards Exemption Category 1, criteria (a).
Provide a summary of all beneficial uses of receiving waters downstream of the project discharge locations ACR, GOLD, IND, MUN, RARE, REC1, REC2, WARM, WILD
Identify all ASBS (areas of special biological significance) receiving waters downstream of the project discharge locations None.
Provide distance from project outfall location to impaired or sensitive receiving waters The San Diego River is located about 0.3 miles north of the site. The San Diego River is listed on the most recently released 2014/2016 Integrated active CWA Section 303d list with the following impairments: Bethnic Community Effects, Cadmium, Nitrogen, Oxygen (Dissolved), Phosphorus, TDS, Toxicity and Indicator Bacteria. The San Diego River has a TMDL for Indicator Bacteria. Please note, this updated list differs from the list provided in Appendix K of the October 2018 BMP Design Manual (See Form I-3B on following page for Pollutants/Stressors listed on Appendix K).
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 The site proposes numerous onsite treatment (biofiltration) BMPs. Multi-Habitat Planning Areas and sensitive area have been identified upstream of the site along the slopes on the rear side of the project. In addition, the SanGIS website identifies the San Diego River (0.3 miles feet due north) as a MHPA.

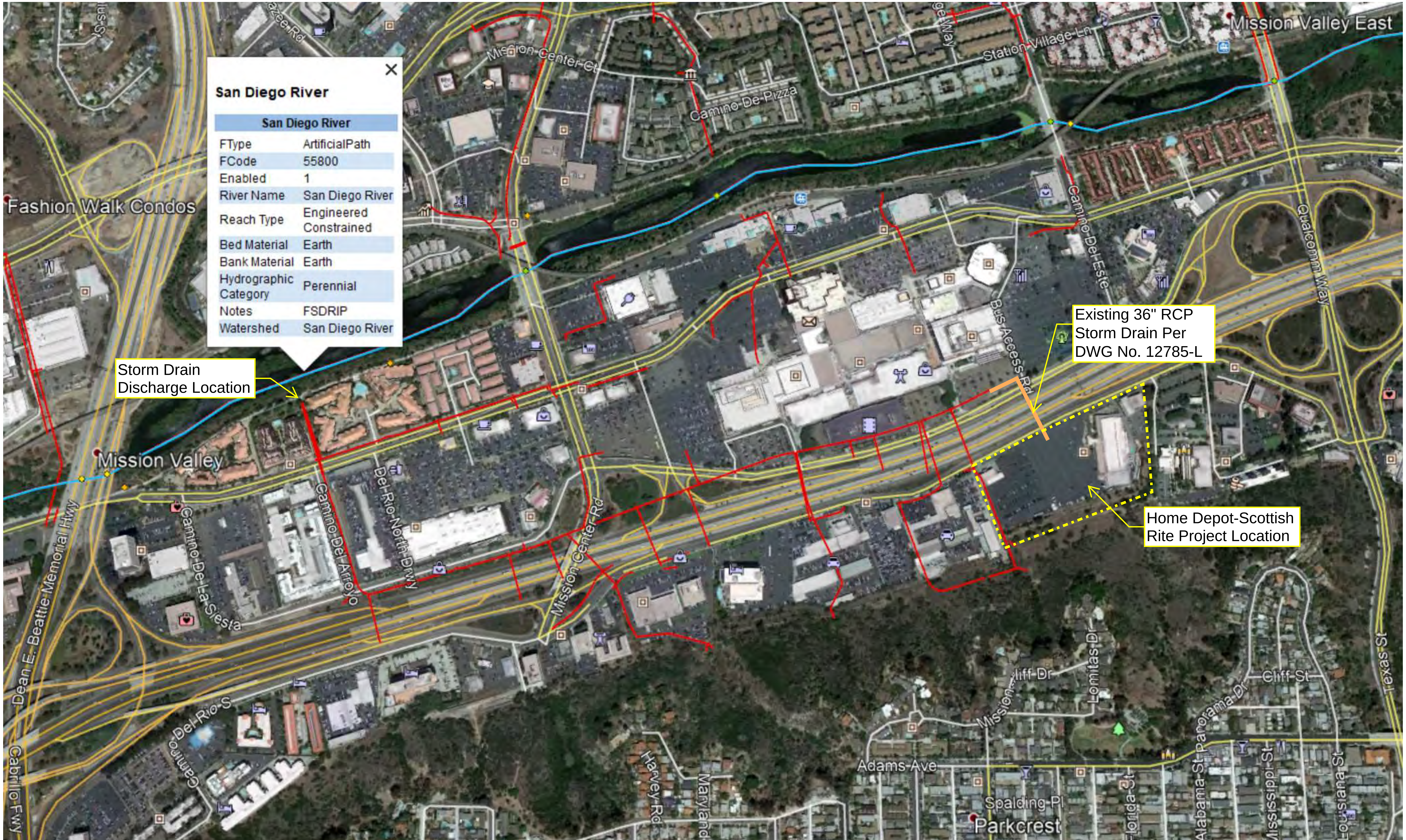
Project Name: Home Depot-Scottish Rite (Mission Valley)

Form I-3B Page 8 of 11		
Identification of Receiving Water Pollutants of Concern		
List any 303(d) impaired water bodies within the path of storm water from the project site to the Pacific Ocean (or bay, lagoon, lake or reservoir, as applicable), identify the pollutant(s)/stressor(s) causing impairment, and identify any TMDLs and/or Highest Priority Pollutants from the WQIP for the impaired water bodies:		
303(d) Impaired Water Body (Refer to Appendix K)	Pollutant(s)/Stressor(s) (Refer to Appendix K)	TMDLs/WQIP Highest Priority Pollutant (Refer to Table 1-4 in Chapter 1)
San Diego River	Enteroccus, Fecal Coliform, Low Dissolved	Indicator Bacteria
	Oxygen, Manganese, Nitrogen, Phosphorus,	
	Total Dissolved Solids, Toxicity	
Pacific Ocean Shoreline, San Diego HU at the San Diego River outlet at Dog Beach	Indicator Bacteria	Indicator Bacteria

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): This site connects to the MS4 storm drain system which directly discharges into the San Diego River which has been identified as a Homdromodification-exempt river reach. See Map exhibit on following sheet showing the site and the exempt system from the site to the San Diego River per the WMAA and As-Builts for the 36" RCP Storm Drain in DWG 12785-L. Note: If "No" answer has been selected the SWQMP must include an exhibit that shows the storm water conveyance system from the project site to an exempt water body. The exhibit should include details about the conveyance system and the outfall to the exempt water body.
Critical Coarse Sediment Yield Areas* ***This Section only required if hydromodification management requirements apply**
Based on Section 6.2 and Appendix H does CCSYA exist on the project footprint or in the upstream area draining through the project footprint? ☐ Yes ☐ No Discussion / Additional Information: N/A, HMP -exempt

THE HOME DEPOT-SCOTTISH RITE
HMP -EXEMPTION EXHIBIT
(Direct discharge into San Diego River)



Constructed storm drain per City GIS records. Hmod-exempt storm drain system per WMAA.

Constructed storm drain per As-Builts, DWG 12785-L. Ties into exempt storm drain system.

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, HMP-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:
Discussion / Additional Information: (optional)

Form I-3B Page 11 of 11
Other Site Requirements and Constraints
When applicable, list other site requirements or constraints that will influence storm water management design, such as zoning requirements including setbacks and open space, or local codes governing minimum street width, sidewalk construction, allowable pavement types, and drainage requirements. The project is a redevelopment project and as such, the existing offsite downstream storm drain infrastructure has been set along Camino del Rio South. Therefore, any onsite storm drain piping will be constrained vertically to be able to tie in to the existing system. Also, the placement of the main Home Depot building will be located directly over the existing storm drain conveying offsite runoff through towards Camino del Rio South storm drain system. This storm drain will need to be re-routed around the proposed building footprint.
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.

Project Name: Home Depot-Scottish Rite (Mission Valley)

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:			

Project Name: Home Depot-Scottish Rite (Mission Valley)

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 <u>all</u> "No" answers shown above.	

Project Name: Home Depot-Scottish Rite (Mission Valley)

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: The existing site is currently developed with buildings and parking lots. The project will alter internal drainage patterns but will attempt to mimic existing drainage areas and mitigate flows at all discharge points where feasible.			
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:			

Form I-5B Page 2 of 4			
Site Design Requirement	Applied?		
4.3.3 Minimize Impervious Area	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.3.3 not implemented: The project will be redeveloped as two commercial sites. Compared to existing conditions, the site will increase the amount of landscaping and minimize the amount of impervious area where feasible.			
4.3.4 Minimize Soil Compaction	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.3.4 not implemented: Soil within dispersion areas and at proposed biofiltration basin areas will minimize soil compaction and/or will be re-tilled prior to its construction.			
4.3.5 Impervious Area Dispersion	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.3.5 not implemented: A portion of the projects' roof and parking areas will drain towards landscaped areas acting as dispersion areas.			
5-1 Is the pervious area receiving runon from impervious area identified on the site map?	☑ Yes	☐ No	☐ N/A
5-2 Does the pervious area satisfy the design criteria in 4.3.5 Fact Sheet in Appendix E (e.g. maximum slope, minimum length, etc.)	☑ Yes	☐ No	☐ N/A
5-3 Is impervious area dispersion credit volume calculated using Appendix B.2.1.1 and 4.3.5 Fact Sheet in Appendix E?	☑ Yes	☐ No	☐ N/A

Form I-5B Page 3 of 4			
Site Design Requirement	Applied?		
4.3.6 Runoff Collection	☑ Yes	☐ No	☐ N/A
Discussion / justification if 4.3.6 not implemented: The proposed local area drain system is proposed in portions of the site to collect runoff to minimize transporting of pollutants.			
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: Landscaped areas will be with native or drought tolerant species.			
4.3.8 Harvest and Use Precipitation	☐ Yes	☐ No	☑ N/A
Discussion / justification if 4.3.8 not implemented:			
8-1 Are rain barrels implemented in accordance with design criteria in 4.3.8 Fact Sheet? If yes, are they shown on the site map?	☐ Yes	☐ No	☑ N/A
8-2 Is the rain barrel credit volume calculated using Appendix B.2.2.2 and 4.3.8 Fact Sheet in Appendix E?	☐ Yes	☐ No	☑ N/A

Project Name: Home Depot-Scottish Rite (Mission Valley)

Form I-5B Page 4 of 4

Insert Site Map with all site design BMPs identified:

BMPDM SECTION AND ELEMENT		SYMBOL/LOCATION
4.3.2	CONSERVE NATURAL AREAS, SOILS, AND VEGETATION	
4.3.3	MINIMIZE IMPERVIOUS AREA	
4.3.4	MINIMIZE SOIL COMPACTION	
4.3.5	IMPERVIOUS AREA DISPERSION	
4.3.6	RUNOFF COLLECTION	,
4.3.7	LANDSCAPING WITH NATIVE OR DROUGHT TOLERANT SPECIES	

Project Name: Home Depot-Scottish Rite (Mission Valley)

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.	
In accordance with Section 5.1 of the BMP Design Manual, the following steps were taken in selecting the site's BMPs: 1. Based on the drainage subareas delineated on the hydrology analysis, DMA areas were established. The DMAs which were considered to be self-mitigating or deminimis were then identified. The DCVs for each remaining DMAs were then calculated. 2. A 'harvest and use feasibility analysis' was then conducted and determined that harvest and use measures were infeasible for this site. 3. A soil infiltration feasibility analysis was then performed by the geotechnical engineer and civil engineer to determine whether there was an opportunity for placement of infiltration facilities at the site based on it's infiltration potential. Based on soil data and perc tests performed at the site, it was determined that the site has minimal, if any infiltration potential due to its shallow clay layers. 4. Based on the DMA subareas and infiltration capacities, BMPs were proposed. Where possible, above ground biofiltration areas (basins) were placed. (Continue on page 2 as necessary.)	

Form I-6 Page 2 of 12
(Continued from page 1) Where areas were restrictive for placement of above ground facilities, modular flow-based proprietary biofiltration devices are proposed. In those instances, Target volume reduction amounts were calculated for each DMA. Amended soils placed throughout the site were then proposed to achieve adequate volume retention. 5. Flow-thru treatment Control BMPs were not necessary since Steps 1A and 3 above were able to achieve treatment of the DCV and volume retention requirements. 6. This SWQMP including calculations for proposed BMPs is being prepared as Step 6 of achieving stormwater compliance. 7. Attachment 3 of this SWQMP provides maintenance information. Form I-6 (Structural BMP Summary Information) includes the maintenance responsibility and funding mechanism. Maintenance agreements for the site will be addressed during the final engineering phases of the project.

Project Name: Home Depot-Scottish Rite (Mission Valley)

Form I-6 Page 1 of 12 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BF-1-1	
Construction Plan Sheet No.	
Type of Structural BMP: ☐ Retention by harvest and use (e.g. HU-1, cistern) ☐ Retention by infiltration basin (INF-1) ☐ Retention by bioretention (INF-2) ☐ Retention by permeable pavement (INF-3) ☐ Partial retention by biofiltration with partial retention (PR-1) ☐ Biofiltration (BF-1) ☐ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide BMP type/description in discussion section below) ☐ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below) ☐ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion section below) ☐ Detention pond or vault for hydromodification management ☐ Other (describe in discussion section below)	
Purpose: ☐ Pollutant control only ☐ Hydromodification control only ☐ Combined pollutant control and hydromodification control ☐ Pre-treatment/forebay for another structural BMP ☐ Other (describe in discussion section below)	
Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Bryan D. Smith, P.E. Fuscoe Engineering, Inc. 6390 Greenwich Drive, Suite 170, San Diego, CA 92122 858.554.1500
Who will be the final owner of this BMP?	Scottish Rite: 1561 Camino del Rio South, San Diego CA, 92108
Who will maintain this BMP into perpetuity?	Scottish Rite: 1561 Camino del Rio South, San Diego CA, 92108
What is the funding mechanism for maintenance?	Privately funded by Owner

Form I-6 Page 2 of 12 (Copy as many as needed)
Structural BMP ID No. BF-1-1
Construction Plan Sheet No.
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs):

Project Name: Home Depot-Scottish Rite (Mission Valley)

Form I-6 Page 3 of 12 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BF-3-1	
Construction Plan Sheet No.	
Type of Structural BMP: ☐ Retention by harvest and use (e.g. HU-1, cistern) ☐ Retention by infiltration basin (INF-1) ☐ Retention by bioretention (INF-2) ☐ Retention by permeable pavement (INF-3) ☐ Partial retention by biofiltration with partial retention (PR-1) ☐ Biofiltration (BF-1) ☐ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide BMP type/description in discussion section below) ☐ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below) ☐ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion section below) ☐ Detention pond or vault for hydromodification management ☐ Other (describe in discussion section below)	
Purpose: ☐ Pollutant control only ☐ Hydromodification control only ☐ Combined pollutant control and hydromodification control ☐ Pre-treatment/forebay for another structural BMP ☐ Other (describe in discussion section below)	
Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Bryan D. Smith, P.E. Fuscoe Engineering, Inc. 6390 Greenwich Drive, Suite 170, San Diego, CA 92122 858.554.1500
Who will be the final owner of this BMP?	Scottish Rite: 1561 Camino del Rio South, San Diego CA, 92108
Who will maintain this BMP into perpetuity?	Scottish Rite: 1561 Camino del Rio South, San Diego CA, 92108
What is the funding mechanism for maintenance?	Privately funded by Owner

Form I-6 Page 4 of 12 (Copy as many as needed)
Structural BMP ID No. BF-3-1
Construction Plan Sheet No.
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): This BMP is a flow based proprietary modular biofiltration device which qualifies as a biofiltration facility since it is TAPE-certified and complies with the requirements in Appendix F of the BMP Design Manual.

Project Name: Home Depot-Scottish Rite (Mission Valley)

Form I-6 Page 5 of 12 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BF-1-2	
Construction Plan Sheet No.	
Type of Structural BMP: ☐ Retention by harvest and use (e.g. HU-1, cistern) ☐ Retention by infiltration basin (INF-1) ☐ Retention by bioretention (INF-2) ☐ Retention by permeable pavement (INF-3) ☐ Partial retention by biofiltration with partial retention (PR-1) ☑ Biofiltration (BF-1) ☐ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide BMP type/description in discussion section below) ☐ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below) ☐ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion section below) ☐ Detention pond or vault for hydromodification management ☐ Other (describe in discussion section below)	
Purpose: ☑ Pollutant control only ☐ Hydromodification control only ☐ Combined pollutant control and hydromodification control ☐ Pre-treatment/forebay for another structural BMP ☐ Other (describe in discussion section below)	
Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Bryan D. Smith, P.E. Fuscoe Engineering, Inc. 6390 Greenwich Drive, Suite 170, San Diego, CA 92122 858.554.1500
Who will be the final owner of this BMP?	Home Depot USA, Inc.: 1895 Camino del Rio South, San Diego CA, 9108
Who will maintain this BMP into perpetuity?	Home Depot USA, Inc: 1895 Camino del Rio South, San Diego CA, 9108
What is the funding mechanism for maintenance?	Privately funded by Owner

Form I-6 Page 6 of 12 (Copy as many as needed)
Structural BMP ID No. BF-1-2
Construction Plan Sheet No.
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs):

Form I-6 Page 7 of 12 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BF-3-2	
Construction Plan Sheet No.	
Type of Structural BMP: ☐ Retention by harvest and use (e.g. HU-1, cistern) ☐ Retention by infiltration basin (INF-1) ☐ Retention by bioretention (INF-2) ☐ Retention by permeable pavement (INF-3) ☐ Partial retention by biofiltration with partial retention (PR-1) ☐ Biofiltration (BF-1) ☐ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide BMP type/description in discussion section below) ☐ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below) ☐ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion section below) ☐ Detention pond or vault for hydromodification management ☐ Other (describe in discussion section below)	
Purpose: ☐ Pollutant control only ☐ Hydromodification control only ☐ Combined pollutant control and hydromodification control ☐ Pre-treatment/forebay for another structural BMP ☐ Other (describe in discussion section below)	
Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Bryan D. Smith, P.E. Fuscoe Engineering, Inc. 6390 Greenwich Drive, Suite 170, San Diego, CA 92122 858.554.1500
Who will be the final owner of this BMP?	Home Depot USA, Inc.: 1895 Camino del Rio South, San Diego CA, 9108
Who will maintain this BMP into perpetuity?	Home Depot USA, Inc.: 1895 Camino del Rio South, San Diego CA, 9108
What is the funding mechanism for maintenance?	Privately funded by Owner

Form I-6 Page 8 of 12 (Copy as many as needed)
Structural BMP ID No. BF-3-2
Construction Plan Sheet No.
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): This BMP is a flow based proprietary modular biofiltration device which qualifies as a biofiltration facility since it is TAPE-certified and complies with the requirements in Appendix F of the BMP Design Manual.

Project Name: Home Depot-Scottish Rite (Mission Valley)

Form I-6 Page 9 of 12 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BF-3-3	
Construction Plan Sheet No.	
Type of Structural BMP: ☐ Retention by harvest and use (e.g. HU-1, cistern) ☐ Retention by infiltration basin (INF-1) ☐ Retention by bioretention (INF-2) ☐ Retention by permeable pavement (INF-3) ☐ Partial retention by biofiltration with partial retention (PR-1) ☐ Biofiltration (BF-1) ☐ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide BMP type/description in discussion section below) ☐ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below) ☐ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion section below) ☐ Detention pond or vault for hydromodification management ☐ Other (describe in discussion section below)	
Purpose: ☐ Pollutant control only ☐ Hydromodification control only ☐ Combined pollutant control and hydromodification control ☐ Pre-treatment/forebay for another structural BMP ☐ Other (describe in discussion section below)	
Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Bryan D. Smith, P.E. Fuscoe Engineering, Inc. 6390 Greenwich Drive, Suite 170, San Diego, CA 92122 858.554.1500
Who will be the final owner of this BMP?	Home Depot USA, Inc.: 1895 Camino del Rio South, San Diego CA, 9108
Who will maintain this BMP into perpetuity?	Home Depot USA, Inc.: 1895 Camino del Rio South, San Diego CA, 9108
What is the funding mechanism for maintenance?	Privately funded by Owner

Form I-6 Page 10 of 12 (Copy as many as needed)
Structural BMP ID No. BF-3-3
Construction Plan Sheet No.
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs): This BMP is a flow based proprietary modular biofiltration device which qualifies as a biofiltration facility since it is TAPE-certified and complies with the requirements in Appendix F of the BMP Design Manual.

Form I-6 Page 11 of 12 (Copy as many as needed)	
Structural BMP Summary Information	
Structural BMP ID No. BF-3-4	
Construction Plan Sheet No.	
Type of Structural BMP: ☐ Retention by harvest and use (e.g. HU-1, cistern) ☐ Retention by infiltration basin (INF-1) ☐ Retention by bioretention (INF-2) ☐ Retention by permeable pavement (INF-3) ☐ Partial retention by biofiltration with partial retention (PR-1) ☐ Biofiltration (BF-1) ☐ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide BMP type/description in discussion section below) ☐ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below) ☐ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion section below) ☐ Detention pond or vault for hydromodification management ☐ Other (describe in discussion section below)	
Purpose: ☐ Pollutant control only ☐ Hydromodification control only ☐ Combined pollutant control and hydromodification control ☐ Pre-treatment/forebay for another structural BMP ☐ Other (describe in discussion section below)	
Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Bryan D. Smith, P.E. Fuscoe Engineering, Inc. 6390 Greenwich Drive, Suite 170, San Diego, CA 92122 858.554.1500
Who will be the final owner of this BMP?	Home Depot USA, Inc.: 1895 Camino del Rio South, San Diego CA, 9108
Who will maintain this BMP into perpetuity?	Home Depot USA, Inc.: 1895 Camino del Rio South, San Diego CA, 9108
What is the funding mechanism for maintenance?	Privately funded by Owner

Form I-6 Page 12 of 12 (Copy as many as needed)
Structural BMP ID No. BF-3-4
Construction Plan Sheet No.
Discussion (as needed; must include worksheets showing BMP sizing calculations in the SWQMPs):

This BMP is a flow based proprietary modular biofiltration device which qualifies as a biofiltration facility since it is TAPE-certified and complies with the requirements in Appendix F of the BMP Design Manual.

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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: Home Depot-Scottish Rite (Mission Valley)

Indicate which Items are Included:

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

Project Name: Home Depot-Scottish Rite (Mission Valley)

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

The DMA Exhibit must identify:

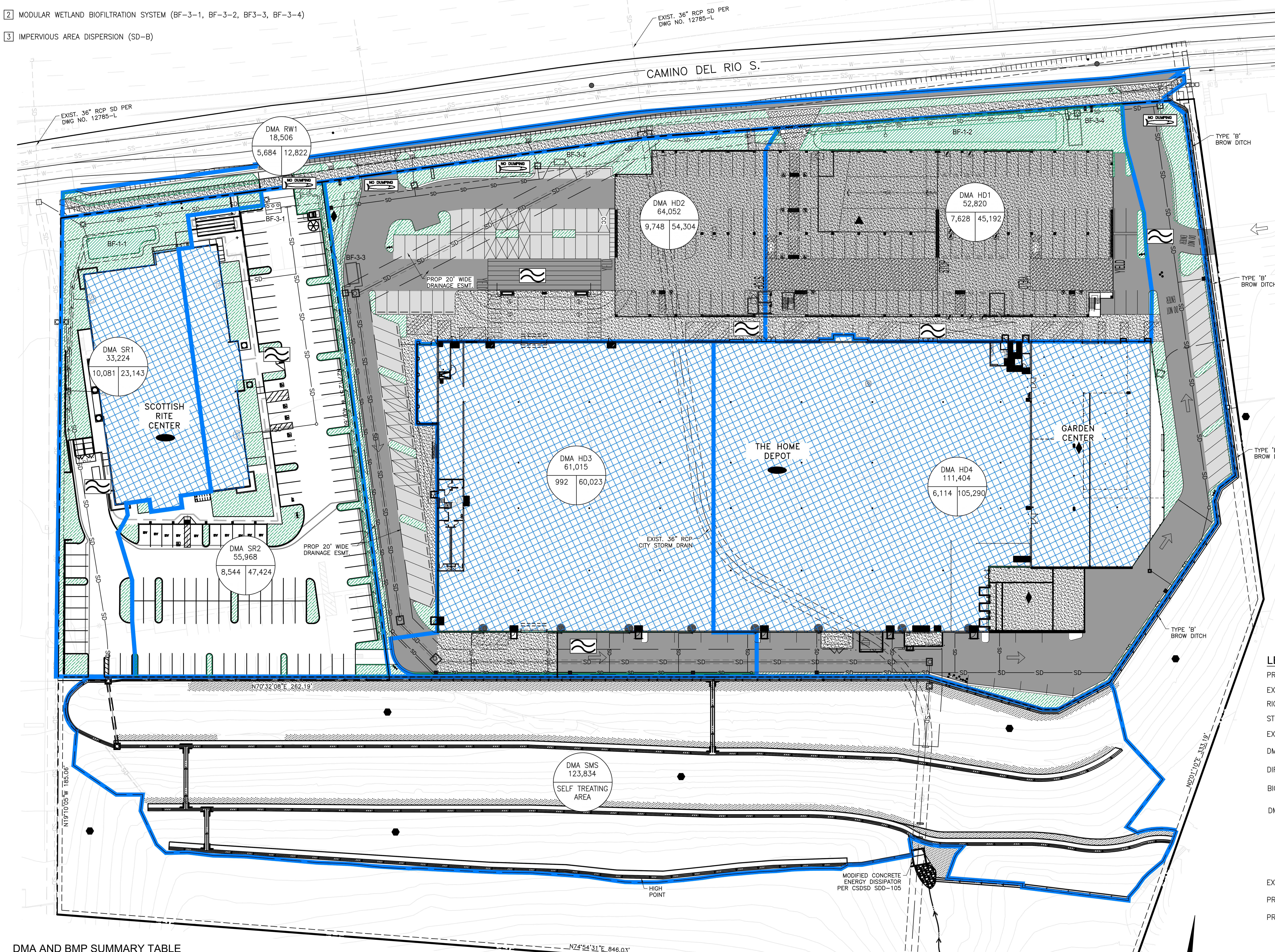
- ☑ Underlying hydrologic soil group
- ☑ Approximate depth to groundwater
- ☑ Existing natural hydrologic features (watercourses, seeps, springs, wetlands)
- ☑ Critical coarse sediment yield areas to be protected
- ☑ Existing topography and impervious areas
- ☑ Existing and proposed site drainage network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

Attachment 1a

DMA Exhibits

PERMANENT STORM WATER BMP NOTES

1. POLLUTANT CONTROL BMP (PR-1, BIOFILTRATION W/PARTIAL INFILTRATION)
2. MODULAR WETLAND BIOFILTRATION SYSTEM (BF-3-1, BF-3-2, BF3-3, BF-3-4)
3. IMPERVIOUS AREA DISPERSION (SD-B)



PROJECT SITE INFO

UNDERLYING HYDROLOGIC SOIL: DEEMED D DUE TO SHALLOW CLAY LAYERS
APPROXIMATE DEPTH TO GROUNDWATER: 18-21 FEET
EXISTING NATURAL HYDROLOGIC FEATURES (WATERCOURSES, SEEPS, SPRINGS, WETLANDS): NONE
INFILTRATION FEASIBILITY: PARTIAL INFILTRATION
CRITICAL COARSE SEDIMENT YEILD AREAS TO BE PROTECTED: NONE
EXISTING IMPERVIOUS AREA: 385,201 SF
DISTURBED AREA: 11.95 AC (520,823 SF)
PROPOSED/REPLACED IMPERVIOUS AREA: 348,198 SF
PROPOSED PERVIOUS AREA (INCLUDES LANDSCAPING): 172,625 SF

HYDROMODIFICATION EXEMPTION

THE PROJECT SITE DISCHARGES TO AN EXEMPT CONVEYANCE SYSTEM PER THE WMAA AND THEREFORE, HYDROMODIFICATION DESIGN AND CONTROLS DO NOT APPLY TO THE PROJECT SITE.

SOURCE CONTROL BMP LEGEND — SYMBOL/LOCATION

BMP DRAINAGE MANUAL SECTION AND ELEMENT

- 4.2.1 PREVENTION OF ILLICIT DISCHARGE IN THE MS4 — NOT PLOTTABLE*
- 4.2.2 STORM DRAIN STENCILING AND SIGNAGE — NO DUMPING
- 4.2.3 PROTECT OUTDOOR MATERIALS STORAGE AREAS
- 4.2.4 PROTECT MATERIALS STORED IN OUTDOOR WORK AREAS
- 4.2.5 PROTECT TRASH STORAGE AREAS
- 4.2.6 ADDITIONAL BMPS BASED ON POTENTIAL SOURCES OF RUNOFF POLLUTANTS:
  - ONSITE STORM DRAIN INLETS — NO DUMPING
  - INTERIOR FLOOR DRAINS & ELEVATOR SHAFT SUMP PUMPS
  - INTERIOR PARKING GARAGES
  - NEED FOR FUTURE INDOOR & STRUCTURAL PEST CONTROL
  - LANDSCAPE/OUTDOOR PESTICIDE USE
  - REFUSE AREAS
  - OUTDOOR STORAGE OF EQUIPMENT OR MATERIALS
  - LOADING DOCKS
  - FIRE SPRINKLER TEST WATER
  - MISCELLANEOUS DRAIN OR WASH WATER
  - PLAZAS, SIDEWALKS, AND PARKING LOTS
  - PLANT NURSERIES AND GARDEN CENTERS

SITE DESIGN BMP LEGEND

BMP DRAINAGE MANUAL SECTION AND ELEMENT

- 4.3.2 CONSERVE NATURAL AREAS, SOILS, AND VEGETATION
- 4.3.3 MINIMIZE IMPERVIOUS AREA
- 4.3.4 MINIMIZE SOIL COMPACTION
- 4.3.5 IMPERVIOUS AREA DISPERSION
- 4.3.6 RUNOFF COLLECTION
- 4.3.7 LANDSCAPING WITH NATIVE OR DROUGHT TOLERANT SPECIES

LEGEND

- PROPERTY LINE
- EXISTING EASEMENT
- RIGHT-OF-WAY
- STREET CENTERLINE
- EXISTING STORM DRAIN — SD
- DMA LIMITS
- DIRECTION OF FLOW
- BIOFILTRATION BASIN — BF-##
- DMA DESIGNATION — DMA ID / TOTAL AREA / #### | #### (PERVIOUS AREA | IMPERVIOUS AREA)
- EXISTING CONTOUR — 260
- PROPOSED CONTOUR — 280
- PROPOSED STORM DRAIN — SD

LEGEND (CONT.)

- PROPOSED PERVIOUS AREA
- PROPOSED IMPERVIOUS ROOF
- PROPOSED IMPERVIOUS AC PAVING
- PROPOSED IMPERVIOUS PARKING
- PROPOSED IMPERVIOUS HARDSCAPE
- PROPOSED IMPERVIOUS CONCRETE
- PROPOSED BIOFILTRATION BASIN
- MODULAR WETLAND UNITS — BMP 8(MWS)

DMA AND BMP SUMMARY TABLE

BASIN (BMP ID)	TOTAL DEPTH (FT)	BOTTOM SURFACE AREA (SF)	RISER HEIGHT (FT)	RISER RIM OPENING (FT)	UNDERDRAIN ORIFICE NO. OF ORIFICES	UNDERDRAIN ORIFICE DIAM. (IN)	BASIN SIDE SLOPES (H:V)	SURFACE PONDING DEPTH (IN)	SOIL MEDIA THICKNESS (IN)	GRAVEL STORAGE DEPTH (IN)	STORAGE BELOW UNDERDRAIN (IN)	MEDIA FILTRATION RATE (IN/HR)
BF-1-1	1	785	0.5	3 X 3	1	4	3:1	6	18	12	3	5
BF-1-2	1.5	2,282	0.5	3 X 3	1	4	3:1	6	18	12	3	5

DMA ID	BMP ID	RUNOFF FACTOR	INTENSITY (IN/HR)	AREA TREATED (SF)	AREA TREATED (AC)	TREATMENT FACTOR	Q (TREAT) REQ'D (CFS)	TREATMENT MODEL SIZE	MODEL CAPACITY (CFS)	MODEL NUMBER
SR2	BF-3-1	0.78	0.2	55,968	1.28	1.5	0.30	0.346	0.2	MWS L-8-12-V
HD2	BF-3-2	0.78	0.2	64,052	1.47	1.5	0.34	0.462	0.2	MWS L-8-16-V
HD3	BF-3-3	0.89	0.2	61,015	1.40	1.5	0.37	0.577	0.2	MWS L-8-20-V
HD4	BF-3-4	0.86	0.2	111,404	2.5	1.5	0.645	0.693	0.2	MWS L-10-20-V

VOLUME REDUCTION SUMMARY

VOLUME (CF)	HOME DEPOT	SCOTTISH RITE
TARGET	597	307
ACHIEVED	1727	1069

N

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

NO.	DATE	REVISION

HOME DEPOT MISSION VALLEY

ATTACHMENT 1A & 1B - DRAINAGE MANAGEMENT AREAS (DMA) EXHIBIT

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

JOB NO. 128-020
DRAWN BY: E.S.
DATE: 8/12/2020
SHEET 1 of 2

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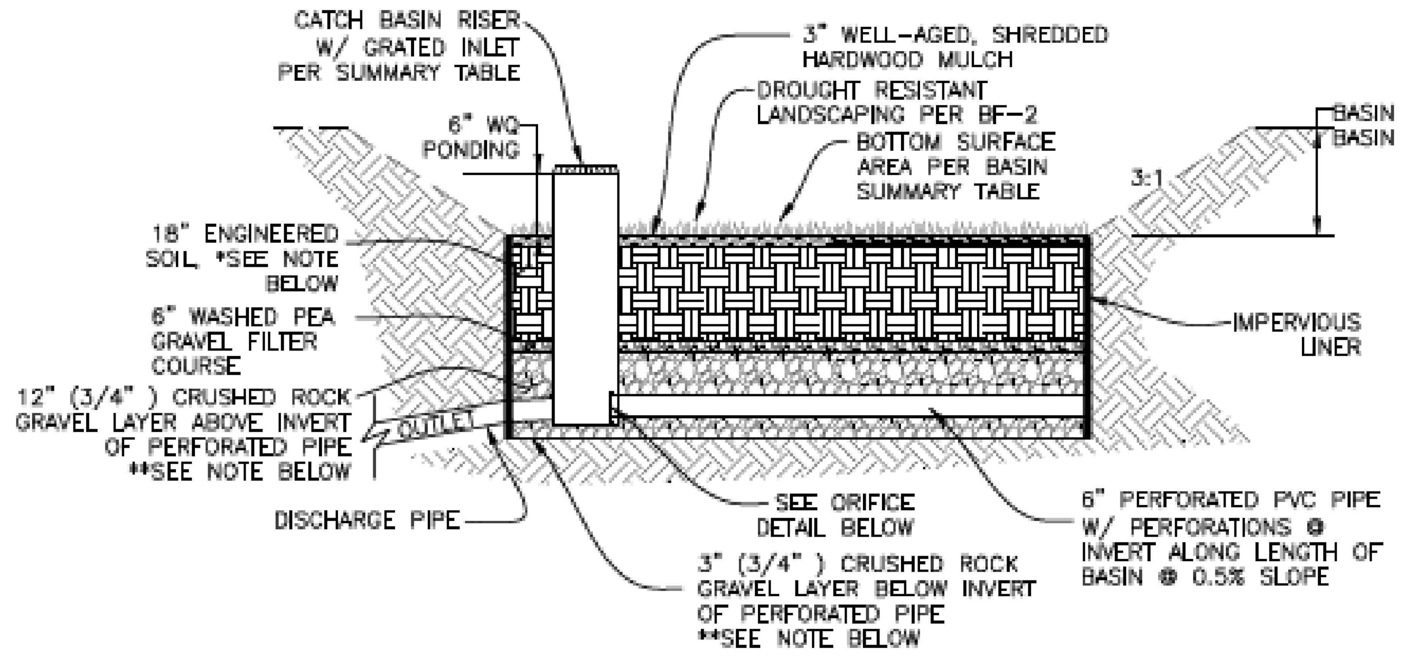


*ENGINEERED SOIL NOTE: LAYER SHALL BE 18" DEEP AND SHALL HAVE VOLUME AND WEIGHT AS FOLLOWS (PER COSD LID MANUAL G.1.1) AND MINIMUM INFILTRATION RATE OF 5.0 IN/HR.:

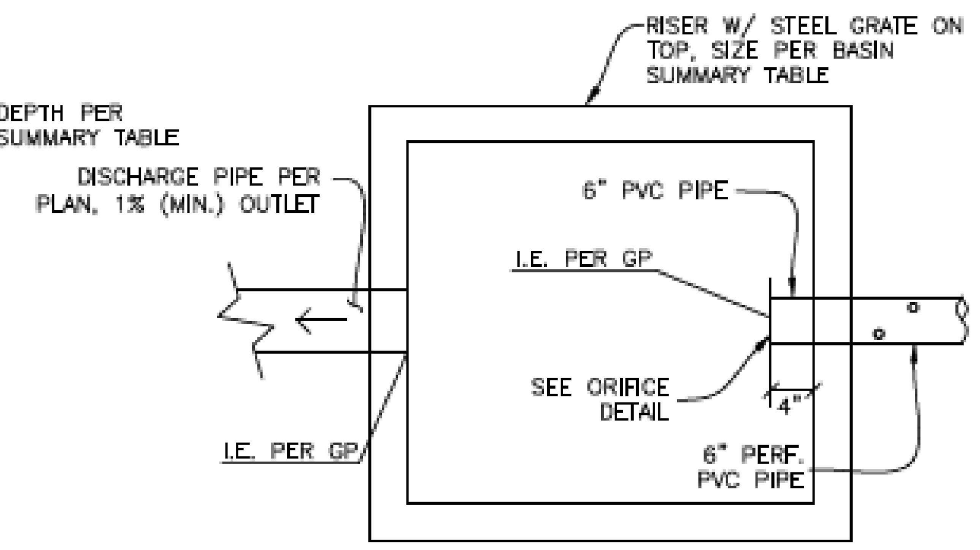
SOIL		SANDY LOAM			
	SAND	SAND	SILT	CLAY	COMPOST
VOLUME	65%	20%			15%
WEIGHT	75–80%		10% MAX	3% MAX	9% MAX

**12" CRUSHED ROCK LAYER, CL. II PERMEABLE ROCK WITH MIN. 40% POROSITY PER CALTRANS SPEC 68-202F(3).

NUTRIENT SENSITIVE MEDIA NOTE: ALL MEDIA SHALL COMPLY WITH THE NUTRIENT SENSITIVE MEDIA PER BF-2.

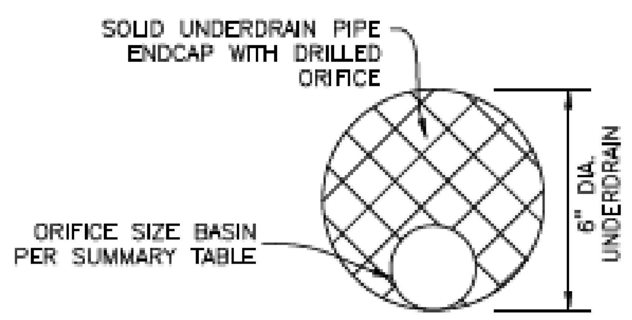
BIOFILTRATION BASIN DETAIL

NOT TO SCALE



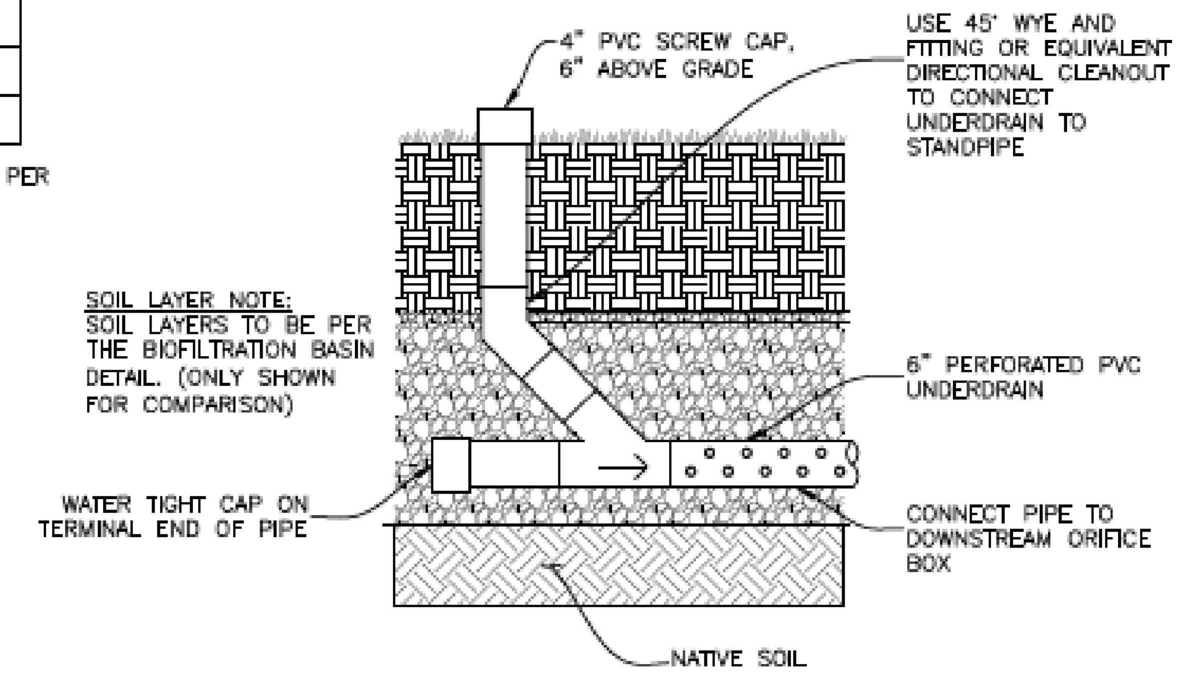
ORIFICE BOX DETAIL–PLAN VIEW

NOT TO SCALE



ORIFICE DETAIL

NOT TO SCALE



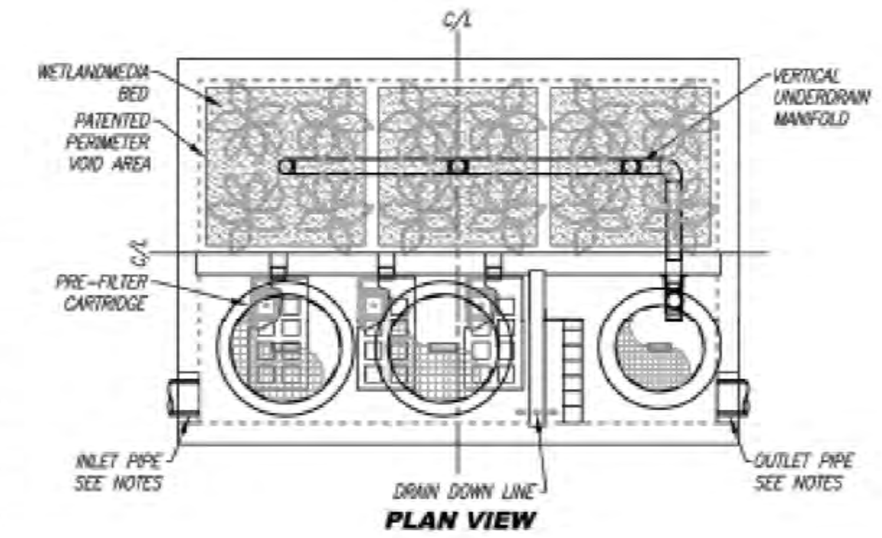
BIOFILTRATION BASIN UNDERDRAIN CLEANOUT DETAIL

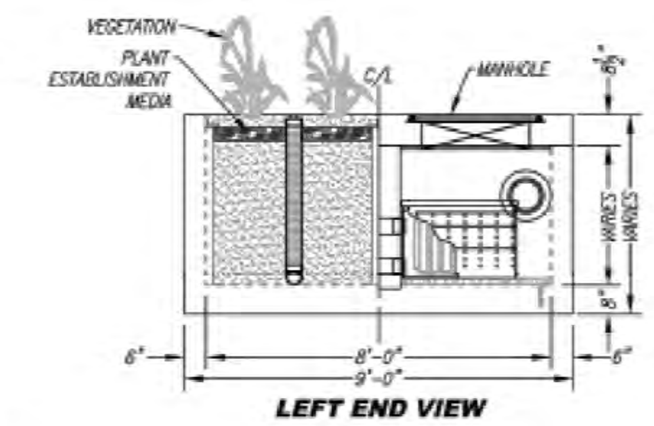
NOT TO SCALE

SITE SPECIFIC DATA

PROJECT NUMBER			
PROJECT NAME			
PROJECT LOCATION			
STRUCTURE ID			
TREATMENT REQUIRED			
VOLUME BASED (CF)		FLOW BASED (CFS)	
N/A		0.346	
PEAK BYPASS REQUIRED (CFS) – IF APPLICABLE			OFFLINE
PIPE DATA	I.E.	MATERIAL	DIAMETER
INLET PIPE 1			
INLET PIPE 2	N/A	N/A	N/A
OUTLET PIPE			
	PRETREATMENT	BIOFILTRATION	DISCHARGE
RIM ELEVATION			
SURFACE LOAD	PEDESTRIAN		
FRAME & COVER	2EA ø30"	OPEN PLANTER	ø24"
NOTES:			

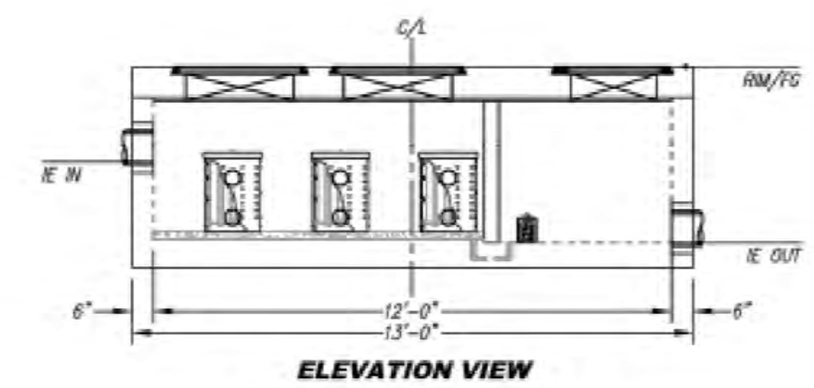
* PRELIMINARY NOT FOR CONSTRUCTION

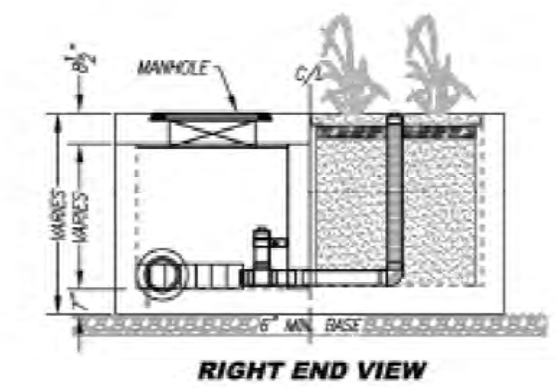




INSTALLATION NOTES

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





GENERAL NOTES

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

TREATMENT FLOW (CFS)	0.346
OPERATING HEAD (FT)	3.4
PRETREATMENT LOADING RATE (GPM/SF)	2.0
WETLAND MEDIA LOADING RATE (GPM/SF)	1.0

MODULAR WETLANDS

PROPRIETARY AND CONFIDENTIAL:

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Bio Clean A Forterra Company

MWS-L-8-12-V
STORMWATER BIOFILTRATION SYSTEM
STANDARD DETAIL

TYPICAL PROPRIETARY MODULAR BIOFILTRATION FACILITY

TYPICAL STORM DRAIN SIGNAGE

NO.	DATE	REVISION

HOME DEPOT MISSION VALLEY

ATTACHMENT 1A & 1B - DRAINAGE MANAGEMENT AREAS (DMA) EXHIBIT

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www.fuscoe.com

JOB NO. 128–020

DRAWN BY: E.S.

SHEET 2 of 2

Attachment 1b

Tabular Summary of DMAs

Project Name: Home Depot-Scottish Rite (Mission Valley)

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)
SR1	0.763	0.531	69.65	D	0.72	1,042	BF-1-1	Biofiltration	N/A
SR2	1.285	1.088	84.73	D	0.81	2,002	BF-3-1	Biofiltration	N/A
HD1	1.212	1.037	85.56	D	0.81	1,890	BF-1-2	Biofiltration	N/A
HD2	1.470	1.266	84.78	D	0.81	2,291	BF-3-2	Biofiltration	N/A
HD3	1.401	1.378	98.37	D	0.89	2,398	BF-3-3	Biofiltration	N/A
HD4	2.557	2.417	94.51	D	0.87	4,231	BF-3-4	Biofiltration	N/A
RW1	0.425	0.294	69.29	D	0.71	580	EXEMPT	EXEMPT	
SMS	2.84	0	0	D	.3		SELF MTGTNG		

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
8	11.956	7.994	66.86		0.701	14,434	11.956		N/A

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 1c

Form I-7, Harvest and Use Feasibility Screening Checklist

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]

Office Use assumes 5 employees and 10 non-employees = 15 users
7 gals/users per day = 7 x 15 = 105 gals = 14.04 CF
Landscape = ETWU = (2.8in/mo)x {[(PFxHA/IE]+SLA} x 0.015 = 2.8x{[(0.5x18,169)/ 0.9]+0}x0.015 = 424 gals
Total gals = 105 + 424 = 529 gals/day = 71 CF/day; Therfore 36 hr demand = 107 CF.

3. Calculate the DCV using worksheet B-2.1.
DCV = 2,898 (cubic feet)
[Provide a summary of calculations here]

See Attachment 1e for DCV calculations for each respective site (i.e. the Home Depot and Scottish Rite Center.

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.

Attachment 1d

Infiltration Feasibility Information

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]	Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
HD1	Home Depot
Criteria 1: Infiltration Rate Screening	

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

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

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

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

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

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

Soil Conditions encountered in the nine borings and three down hole percolation tests completed along the frontage area designated for infiltration indicated the subsurface conditions are highly variable with soil types ranging from silty and clayey sands to sandy lean clays and sandy silts. So infiltration is expected to be highly variable.

The depth to groundwater at the time of our field investigations ranged from 26 to 30 feet, and it is expected that groundwater could rise to the 20 foot level for short periods.

More Detailed descriptions of the soils and groundwater Conditions are presented in the Preliminary Geotechnical Investigation Report, dated January 10, 2020 prepared by Moore Twining Associates, Inc.

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

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

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions</th><th colspan="2">Worksheet C.4-1: Form I-8A[2]</th></tr>
<tr><td>2C</td><td>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.</td><td>◉ Yes</td><td>○ No</td></tr>
<tr><td>Criteria 2 Result</td><td>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?</td><td>◉ Yes</td><td>○ No</td></tr>
<tr><td colspan="4">Summarize findings and basis; provide references to related reports or exhibits.

Based on the current planned location for infiltration facilities, setbacks for structures (30 feet) are recommended in the Preliminary Geotechnical Investigation Report, dated January 10, 2020 prepared by Moore Twining Associates, Inc. The Geotechnical Report includes further evaluation of each of the categories responded to in this section.</td></tr>
<tr><th colspan="2">Part 1 Result – Full Infiltration Geotechnical Screening [4]</th><th colspan="2">Result</th></tr>
<tr><td colspan="2">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.</td><td colspan="2">○ Full infiltration Condition

◉ Complete Part 2</td></tr>
</table>

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
HD1		Home Depot
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). The soils encountered in the test borings drilled for the three percolation tests comprised silty sands and clayey sands. The estimated infiltration rates of the materials tested ranged from 0.1 to 0.5 inches per hour. Considering that clays with less infiltration likely exist, stormwater infiltration systems should be designed for an average un-factored infiltration rate of 0.1 inches per hour. The depths, locations, and percolation result details are provided in the Preliminary Geotechnical Investigation Report, dated January 10, 2020 prepared by Moore Twining Associates, Inc.		

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits. The soils encountered in the test borings drilled for the three percolation tests comprised silty sands and clayey sands. The estimated infiltration rates of the materials tested ranged from 0.1 to 0.5 inches per hour. Considering that clays with less infiltration likely exist, stormwater infiltration systems should be designed for an average un-factored infiltration rate of 0.1 inches per hour. The depths, locations, and percolation result details are provided in the Preliminary Geotechnical Investigation Report, dated January 10, 2020 prepared by Moore Twining Associates, Inc.		
Part 2 – Partial Infiltration Geotechnical Screening Result[5]	**Result**	
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	◉ Partial Infiltration Condition ○ No Infiltration Condition	

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions[1]		Worksheet C.4-1: Form I-8A[2]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
HD2 through HD4		Home Depot
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[3]? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ○ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ◉ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ○ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ◉ Yes; Continue to Step 1C. ○ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ○ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ◉ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ○ Yes; continue to Step 1E. ○ No; select an appropriate infiltration testing method.	

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

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

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

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

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

Soil Conditions encountered in the nine borings and three down hole percolation tests completed along the frontage area designated for infiltration indicated the subsurface conditions are highly variable with soil types ranging from silty and clayey sands to sandy lean clays and sandy silts. So infiltration is expected to be highly variable.

The depth to groundwater at the time of our field investigations ranged from 26 to 30 feet, and it is expected that groundwater could rise to the 20 foot level for short periods.

More Detailed descriptions of the soils and groundwater Conditions are presented in the Preliminary Geotechnical Investigation Report, dated January 10, 2020 prepared by Moore Twining Associates, Inc.

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

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

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

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

Based on the current planned location for infiltration facilities, setbacks for structures (30 feet) are recommended in the Preliminary Geotechnical Investigation Report, dated January 10, 2020 prepared by Moore Twining Associates, Inc. The Geotechnical Report includes further evaluation of each of the categories responded to in this section.

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
HD2 through HD4	Home Depot
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).

The soils encountered in the test borings drilled for the three percolation tests comprised silty sands and clayey sands. The estimated infiltration rates of the materials tested ranged from 0.1 to 0.5 inches per hour. Considering that clays with less infiltration likely exist, stormwater infiltration systems should be designed for an average un-factored infiltration rate of 0.1 inches per hour.

The depths, locations, and percolation result details are provided in the Preliminary Geotechnical Investigation Report, dated January 10, 2020 prepared by Moore Twining Associates, Inc.

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[2]
Summarize findings and basis; provide references to related reports or exhibits. The soils encountered in the test borings drilled for the three percolation tests comprised silty sands and clayey sands. The estimated infiltration rates of the materials tested ranged from 0.1 to 0.5 inches per hour. Considering that clays with less infiltration likely exist, stormwater infiltration systems should be designed for an average un-factored infiltration rate of 0.1 inches per hour. The depths, locations, and percolation result details are provided in the Preliminary Geotechnical Investigation Report, dated January 10, 2020 prepared by Moore Twining Associates, Inc.	

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

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

Attachment 1e

Pollutant Control BMP Design

Scottish Rite Center

Worksheets

SCOTTISH RITE ADJUSTED RUNOFF FACTORS

	Runoff Factor	DMA SR1 (SF)	Summation RFxA	DMA SR2 (SF)	Summation RFxA
Roof, Sidewalk, Pavement	0.90	23,143	20,828.70	47,424	42,681.60
Landscape, Basin, Amended Soils	0.10	10,081	1,008.10	8,544	854.40
		33,224	21,836.80	55,968.00	43,536.00
			0.66		0.78

		DMA SR2
Proprietary BMP Q x 1.5	Treatment Flow Required=	0.31 CFS
	Treatment Unit Model	MWS-L-8-12
	Treatment Unit Capacity=	0.346

SCOTTISH RITE VOLUME REDUCTION SUMMARY

	DMA SR1	DMA SR2	Total for SRC Site
Target Volume Retention	103 cf	204 cf	307
Volume Retention Achieved	132 cf	937 cf	1069
		Balance=	762

The City of SAN DIEGO		**Project Name**	Scottish Rite
		BMP ID	BF-1-1 (DMA SR-1)
Sizing Method for Pollutant Removal Criteria			**Worksheet B.5-1**
1	Area draining to the BMP	33224	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.66	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	968	cu. ft.
BMP Parameters			
5	Surface ponding [6 inch minimum, 12 inch maximum]	6	inches
6	Media thickness [18 inches minimum], also add mulch layer and washed ASTM 33 fine aggregate sand thickness to this line for sizing calculations	18	inches
7	Aggregate storage (also add ASTM No 8 stone) above underdrain invert (12 inches typical) – use 0 inches if the aggregate is not over the entire bottom surface area	12	inches
8	Aggregate storage below underdrain invert (3 inches minimum) – use 0 inches if the aggregate is not over the entire bottom surface area	3	inches
9	Freely drained pore storage of the media	0.2	in/in
10	Porosity of aggregate storage	0.4	in/in
11	Media filtration rate to be used for sizing (maximum filtration rate of 5 in/hr. with no outlet control; if the filtration rate is controlled by the outlet use the outlet controlled rate (includes infiltration into the soil and flow rate through the outlet structure) which will be less than 5 in/hr.)	5	in/hr.
Baseline Calculations			
12	Allowable routing time for sizing	6	hours
13	Depth filtered during storm [Line 11 x Line 12]	30	inches
14	Depth of Detention Storage [Line 5 + (Line 6 x Line 9) + (Line 7 x Line 10) + (Line 8 x Line 10)]	15.6	inches
15	Total Depth Treated [Line 13 + Line 14]	45.6	inches
Option 1 – Biofilter 1.5 times the DCV			
16	Required biofiltered volume [1.5 x Line 4]	1453	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12	382	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]	726	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12	559	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]	658	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)	658	sq. ft.
23	Provided BMP Footprint	785	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**	

The City of SAN DIEGO	Project Name	Scottish Rite	
	BMP ID	BF-1-1 (DMA SR1)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	

1	Area draining to the BMP	33224	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.66	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	968	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	0.1	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.05	in/hr.
8	Average annual volume reduction target (Figure B.5-2) When Line 7 > 0.01 in/hr. = Minimum (40, 166.9 x Line 7 +6.62) When Line 7 ≤ 0.01 in/hr. = 3.5%	15.0	%
9	Fraction of DCV to be retained (Figure B.5-3) When Line 8 > 8% = 0.0000013 x Line 8^3 - 0.000057 x Line 8^2 + 0.0086 x Line 8 - 0.014 When Line 8 ≤ 8% = 0.023	0.106	
10	Target volume retention [Line 9 x Line 4]	103	cu. ft.

The City of SAN DIEGO	Project Name	Scottish Rite	
	BMP ID	BF-1-1 (DMA SR1)	
Volume Retention from Biofiltration with Partial Retention BMPs		**Worksheet B.5-3**	

1	Area draining to the BMP	33224	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.66	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	968	cu. ft.
BMP Parameters			
5	Footprint of the BMP	785	sq. ft.
6	Media thickness [18 inches minimum], also add mulch layer and washed ASTM 33 fine aggregate sand thickness to this line for sizing calculations	18	inches
7	Media retained pore space [50% of (Field Capacity-Wilting Point)]	0.05	in/in
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	Porosity of aggregate storage	0.4	in/in
10	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	0.1	in/hr.
11	Factor of safety	2	
12	Reliable infiltration rate, for biofiltration BMP sizing [Line 10/ Line 11]	0.05	in/hr.
Evapotranspiration: Average Annual Volume Retention			
13	Effective evapotranspiration depth [Line 6 x Line 7]	0.9	inches
14	Retained Pore Volume [(Line 13 x Line 5)/12]	59	cu. ft.
15	Fraction of DCV retained in pore spaces [Line 14/Line 4]	0.06	
16	Evapotranspiration average annual capture [ET nomographs in Figure B.5-5]	4.5	%
Infiltration: Average Annual Volume Retention			
17	Drawdown for infiltration storage [(Line 8 x Line 9)/Line 12]	24	hours
18	Equivalent DCV fraction from evapotranspiration (use Line 16 and Line 17 in Figure B.4-1; Refer to Appendix B.4.2.2)	0.02	
19	Infiltration volume storage [(Line 5 x Line 8 x Line 9)/12]	79	cu. ft.
20	Infiltration Storage Fraction of DCV [Line 19/Line 4]	0.08	
21	Total Equivalent Fraction of DCV [Line 18 + Line 20]	0.10	
22	Biofiltration BMP average annual capture [use Line 21 and 17 in Figure B.4-1]	18.83	%
23	Fraction of DCV retained (Figure B.5-3) $0.0000013 \times \text{Line } 22^3 - 0.000057 \times \text{Line } 22^2 + 0.0086 \times \text{Line } 22 - 0.014$	0.136	
24	Volume retention achieved by biofiltration BMP [Line 23 x Line 4]	132	cu. ft.

Volume Retention = 132 cubic feet

The City of SAN DIEGO	Project Name	Scottish Rite	
	BMP ID	BF-3-1 (DMA SR2)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	
1	Area draining to the BMP	55968	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.78	
3	85^{th} percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	1928	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	0.1	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.05	in/hr.
8	Average annual volume reduction target (Figure B.5-2) When Line 7 > 0.01 in/hr. = Minimum (40, 166.9 x Line 7 +6.62) When Line 7 ≤ 0.01 in/hr. = 3.5%	15.0	%
9	Fraction of DCV to be retained (Figure B.5-3) When Line 8 > 8% = 0.0000013 x Line 8^3 - 0.000057 x Line 8^2 + 0.0086 x Line 8 - 0.014 When Line 8 ≤ 8% = 0.023	0.106	
10	Target volume retention [Line 9 x Line 4]	204	cu. ft.

The City of SAN DIEGO	Project Name	Scottish Rite	
	BMP ID	BF-3-1 (DMA SR2)	
Volume Retention From Amended Soils		**Worksheet B.5-7**	

1	Impervious area draining to the pervious area	55968	sq. ft.
2	Pervious area (must meet the requirements in SD-B and SD-F Fact Sheets)	8544	sq. ft.
3	Dispersion Ratio [Line 1/Line 2] Note: This worksheet is not applicable when Line 3 > 50 or Line 3 < 0.25	6.55	
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	0.53	inches
6	Design capture volume [(Line 1 + Line 2) x Line 4 x (Line 5/12)]	2251	cu. ft.
7	Amendment Depth (Choose from 3", 6", 9", 12", 15" and 18")	3	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]	534	cu. ft.
10	Fraction of DCV [Line 9 / Line 6]	0.24	
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	0.1	in/hr.
12	Factor of Safety	2	
13	Reliable Infiltration Rate [Line 11/Line 12]	0.05	in/hr.
14	Dispersion Credit (Based on Figures B.5.6 to B.5.11; Line 10 and Line 13)	0.379	
15	Volume retention due to amendment [Line 1 * (Line 5/12) * Line 14]	937	cu. ft.

Home Depot

Worksheets

THE HOME DEPOT ADJUSTED RUNOFF FACTORS

	Runoff Factor	DMA HD1 (SF)	Summation RFxA	DMA HD2 (SF)	Summation RFxA	DMA HD3 (SF)	Summation RFxA	DMA HD4 (SF)	Summation RFxA
Roof, Sidewalk, Pavement	0.90	45,192	40,672.80	54,304	48,873.60	60,023	54,020.70	105,290	94,761.00
Landscape, Basin, Amended Soils	0.10	7,628	762.80	9,748	974.80	992	99.20	6,114	611.40
		52,820	41,435.60	64,052	49,848.40	61,015	54,119.90	111,404	95,372.40
			0.78		0.78		0.89		0.86

		DMA HD2	DMA HD3	DMA HD4
Proprietary BMP Q x 1.5	Treatment Flow Required=	0.39 CFS	0.47 CFS	0.62 CFS
	Treatment Unit Model	MWS-L-8-16	MWS-L-8-20	MWS-L-10-20
	Treatment Unit Capacity=	0.462 CFS	0.577 CFS	0.693 CFS

THE HOME DEPOT VOLUME REDUCTION SUMMARY

	DMA HD11	DMA HD2	DMA HD3	DMA HD4	Total THD Site
Target Volume Retention	193 cf	234 cf	55 cf	97 cf	579
Volume Retention Achieved	378 cf	632 cf	121 cf	596 cf	1727
				Balance=	1148

The City of SAN DIEGO		Project Name	Home Depot	
		BMP ID	BF-1-2 (DMA HD1)	
Sizing Method for Pollutant Removal Criteria			**Worksheet B.5-1**	
1	Area draining to the BMP		52820	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)		0.78	
3	85^{th} percentile 24-hour rainfall depth		0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]		1820	cu. ft.
BMP Parameters				
5	Surface ponding [6 inch minimum, 12 inch maximum]		6	inches
6	Media thickness [18 inches minimum], also add mulch layer and washed ASTM 33 fine aggregate sand thickness to this line for sizing calculations		18	inches
7	Aggregate storage (also add ASTM No 8 stone) above underdrain invert (12 inches typical) – use 0 inches if the aggregate is not over the entire bottom surface area		12	inches
8	Aggregate storage below underdrain invert (3 inches minimum) – use 0 inches if the aggregate is not over the entire bottom surface area		3	inches
9	Freely drained pore storage of the media		0.2	in/in
10	Porosity of aggregate storage		0.4	in/in
11	Media filtration rate to be used for sizing (maximum filtration rate of 5 in/hr. with no outlet control; if the filtration rate is controlled by the outlet use the outlet controlled rate (includes infiltration into the soil and flow rate through the outlet structure) which will be less than 5 in/hr.)		5	in/hr.
Baseline Calculations				
12	Allowable routing time for sizing		6	hours
13	Depth filtered during storm [Line 11 x Line 12]		30	inches
14	Depth of Detention Storage [Line 5 + (Line 6 x Line 9) + (Line 7 x Line 10) + (Line 8 x Line 10)]		15.6	inches
15	Total Depth Treated [Line 13 + Line 14]		45.6	inches
Option 1 – Biofilter 1.5 times the DCV				
16	Required biofiltered volume [1.5 x Line 4]		2729	cu. ft.
17	Required Footprint [Line 16/ Line 15] x 12		718	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]		1365	cu. ft.
19	Required Footprint [Line 18/ Line 14] x 12		1050	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]		1236	sq. ft.
22	Footprint of the BMP = Maximum(Minimum(Line 17, Line 19), Line 21)		1236	sq. ft.
23	Provided BMP Footprint		2282	sq. ft.
24	Is Line 23 ≥ Line 22?	**Yes, Performance Standard is Met**		

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	BF-1-2 (DMA HD1)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	

1	Area draining to the BMP	52820	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.78	
3	85^{th} percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	1820	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	0.1	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.05	in/hr.
8	Average annual volume reduction target (Figure B.5-2) When Line 7 > 0.01 in/hr. = Minimum (40, 166.9 x Line 7 +6.62) When Line 7 ≤ 0.01 in/hr. = 3.5%	15.0	%
9	Fraction of DCV to be retained (Figure B.5-3) When Line 8 > 8% = 0.0000013 x Line 8^3 - 0.000057 x Line 8^2 + 0.0086 x Line 8 - 0.014 When Line 8 ≤ 8% = 0.023	0.106	
10	Target volume retention [Line 9 x Line 4]	193	cu. ft.

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	BF-1-2 (DMA HD1)	
Volume Retention from Biofiltration with Partial Retention BMPs		**Worksheet B.5-3**	
1	Area draining to the BMP	52820	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.78	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	1820	cu. ft.
BMP Parameters			
5	Footprint of the BMP	2282	sq. ft.
6	Media thickness [18 inches minimum], also add mulch layer and washed ASTM 33 fine aggregate sand thickness to this line for sizing calculations	18	inches
7	Media retained pore space [50% of (Field Capacity-Wilting Point)]	0.05	in/in
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	Porosity of aggregate storage	0.4	in/in
10	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	0.1	in/hr.
11	Factor of safety	2	
12	Reliable infiltration rate, for biofiltration BMP sizing [Line 10/ Line 11]	0.05	in/hr.
Evapotranspiration: Average Annual Volume Retention			
13	Effective evapotranspiration depth [Line 6 x Line 7]	0.9	inches
14	Retained Pore Volume [(Line 13 x Line 5)/12]	171	cu. ft.
15	Fraction of DCV retained in pore spaces [Line 14/Line 4]	0.09	
16	Evapotranspiration average annual capture [ET nomographs in Figure B.5-5]	6.6	%
Infiltration: Average Annual Volume Retention			
17	Drawdown for infiltration storage [(Line 8 x Line 9)/Line 12]	24	hours
18	Equivalent DCV fraction from evapotranspiration (use Line 16 and Line 17 in Figure B.4-1; Refer to Appendix B.4.2.2)	0.04	
19	Infiltration volume storage [(Line 5 x Line 8 x Line 9)/12]	228	cu. ft.
20	Infiltration Storage Fraction of DCV [Line 19/Line 4]	0.13	
21	Total Equivalent Fraction of DCV [Line 18 + Line 20]	0.17	
22	Biofiltration BMP average annual capture [use Line 21 and 17 in Figure B.4-1]	27.67	%
23	Fraction of DCV retained (Figure B.5-3) 0.0000013 x Line 22^3 - 0.000057 x Line 22^2 + 0.0086 x Line 22 - 0.014	0.208	
24	Volume retention achieved by biofiltration BMP [Line 23 x Line 4]	378	cu. ft.
Volume Retention = 378 cubic feet			

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	BF-3-2 (DMA HD2)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	
1	Area draining to the BMP	64052	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.78	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	2207	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	0.1	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.05	in/hr.
8	Average annual volume reduction target (Figure B.5-2) When Line 7 > 0.01 in/hr. = Minimum (40, 166.9 x Line 7 +6.62) When Line 7 ≤ 0.01 in/hr. = 3.5%	15.0	%
9	Fraction of DCV to be retained (Figure B.5-3) When Line 8 > 8% = $0.0000013 \times \text{Line } 8^3 - 0.000057 \times \text{Line } 8^2 + 0.0086 \times \text{Line } 8 - 0.014$ When Line 8 ≤ 8% = 0.023	0.106	
10	Target volume retention [Line 9 x Line 4]	234	cu. ft.

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	BF-3-2 (DMA HD2)	
Volume Retention From Amended Soils		**Worksheet B.5-7**	

1	Impervious area draining to the pervious area	28239	sq. ft.
2	Pervious area (must meet the requirements in SD-B and SD-F Fact Sheets)	7551	sq. ft.
3	Dispersion Ratio [Line 1/Line 2] Note: This worksheet is not applicable when Line 3 > 50 or Line 3 < 0.25	3.74	
4	Adjusted runoff factor [(Line 1 * 0.9 + Line 2 * 0.1) / (Line 1 + Line 2)]	0.73	
5	85th percentile 24-hour rainfall depth	0.53	inches
6	Design capture volume [(Line 1 + Line 2) x Line 4 x (Line 5/12)]	1154	cu. ft.
7	Amendment Depth (Choose from 3", 6", 9", 12", 15" and 18")	3	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]	472	cu. ft.
10	Fraction of DCV [Line 9 / Line 6]	0.41	
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	0.1	in/hr.
12	Factor of Safety	2	
13	Reliable Infiltration Rate [Line 11/Line 12]	0.05	in/hr.
14	Dispersion Credit (Based on Figures B.5.6 to B.5.11; Line 10 and Line 13)	0.507	
15	Volume retention due to amendment [Line 1 * (Line 5/12) * Line 14]	632	cu. ft.

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	BF-3-3 (DMA HD3)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	
1	Area draining to the BMP	61015	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.89	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	2398	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	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0	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%	3.5	%
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	0.023	
10	Target volume retention [Line 9 x Line 4]	55	cu. ft.

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	BF-3-3 (DMA HD3)	
Volume Retention From Amended Soils		**Worksheet B.5-7**	
1	Impervious area draining to the pervious area	8867	sq. ft.
2	Pervious area (must meet the requirements in SD-B and SD-F Fact Sheets)	992	sq. ft.
3	Dispersion Ratio [Line 1/Line 2] Note: This worksheet is not applicable when Line 3 > 50 or Line 3 < 0.25	8.94	
4	Adjusted runoff factor [(Line 1 * 0.9 + Line 2 * 0.1) / (Line 1 + Line 2)]	0.82	
5	85th percentile 24-hour rainfall depth	0.53	inches
6	Design capture volume [(Line 1 + Line 2) x Line 4 x (Line 5/12)]	357	cu. ft.
7	Amendment Depth (Choose from 3", 6", 9", 12", 15" and 18")	3	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]	62	cu. ft.
10	Fraction of DCV [Line 9 / Line 6]	0.17	
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	0.1	in/hr.
12	Factor of Safety	2	
13	Reliable Infiltration Rate [Line 11/Line 12]	0.05	in/hr.
14	Dispersion Credit (Based on Figures B.5.6 to B.5.11; Line 10 and Line 13)	0.309	
15	Volume retention due to amendment [Line 1 * (Line 5/12) * Line 14]	121	cu. ft.

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	BF-3-4 (DMA HD4)	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	
1	Area draining to the BMP	111404	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.86	
3	85^{th} percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	4231	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	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0	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%	3.5	%
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	0.023	
10	Target volume retention [Line 9 x Line 4]	97	cu. ft.

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	BF-3-4 (DMA HD4)	
Volume Retention From Amended Soils		**Worksheet B.5-7**	

1	Impervious area draining to the pervious area	31078	sq. ft.
2	Pervious area (must meet the requirements in SD-B and SD-F Fact Sheets)	6114	sq. ft.
3	Dispersion Ratio [Line 1/Line 2] Note: This worksheet is not applicable when Line 3 > 50 or Line 3 < 0.25	5.08	
4	Adjusted runoff factor [(Line 1 * 0.9 + Line 2 * 0.1) / (Line 1 + Line 2)]	0.77	
5	85th percentile 24-hour rainfall depth	0.53	inches
6	Design capture volume [(Line 1 + Line 2) x Line 4 x (Line 5/12)]	1265	cu. ft.
7	Amendment Depth (Choose from 3", 6", 9", 12", 15" and 18")	3	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]	382	cu. ft.
10	Fraction of DCV [Line 9 / Line 6]	0.3	
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	0.1	in/hr.
12	Factor of Safety	2	
13	Reliable Infiltration Rate [Line 11/Line 12]	0.05	in/hr.
14	Dispersion Credit (Based on Figures B.5.6 to B.5.11; Line 10 and Line 13)	0.434	
15	Volume retention due to amendment [Line 1 * (Line 5/12) * Line 14]	596	cu. ft.

The City of SAN DIEGO	Project Name	Home Depot	
	BMP ID	Sidewalk/Bikelane RW1 - EXEMPT	
Sizing Method for Volume Retention Criteria		**Worksheet B.5-2**	

1	Area draining to the BMP	18506	sq. ft.
2	Adjusted runoff factor for drainage area (Refer to Appendix B.1 and B.2)	0.65	
3	85th percentile 24-hour rainfall depth	0.53	inches
4	Design capture volume [Line 1 x Line 2 x (Line 3/12)]	531	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	0.1	in/hr.
6	Factor of safety	2	
7	Reliable infiltration rate, for biofiltration BMP sizing [Line 5 / Line 6]	0.05	in/hr.
8	Average annual volume reduction target (Figure B.5-2) When Line 7 > 0.01 in/hr. = Minimum (40, 166.9 x Line 7 +6.62) When Line 7 ≤ 0.01 in/hr. = 3.5%	15.0	%
9	Fraction of DCV to be retained (Figure B.5-3) When Line 8 > 8% = 0.0000013 x Line 8^3 - 0.000057 x Line 8^2 + 0.0086 x Line 8 - 0.014 When Line 8 ≤ 8% = 0.023	0.106	
10	Target volume retention [Line 9 x Line 4]	56	cu. ft.

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.

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Attachment 3
Structural BMP Maintenance Information

This is the cover sheet for Attachment 3.

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Project Name: Home Depot-Scottish Rite (Mission Valley)

Indicate which Items are Included:

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

Not applicable at Peliminary Design Phase

Maintenance Fact Sheet:

Biofiltration with Partial Retention

PR-1
Biofiltration with Partial Retention

BMP MAINTENANCE FACT SHEET
FOR
STRUCTURAL BMP PR-1 BIOFILTRATION WITH PARTIAL RETENTION

Biofiltration with partial retention facilities are vegetated surface water systems that filter water through vegetation and soil or engineered media prior to infiltrating into native soils, discharge via underdrain, or overflow to the downstream conveyance system. These BMPs have an elevated underdrain discharge point that creates storage capacity in the aggregate storage layer. Typical biofiltration with partial retention components include:

- Inflow distribution mechanisms (e.g., perimeter flow spreader or filter strips)
- Energy dissipation mechanism for concentrated inflows (e.g., splash blocks or riprap)
- Shallow surface ponding for captured flows
- Side slope and basin bottom vegetation selected based on climate and ponding depth
- Non-floating mulch layer
- Media layer (planting mix or engineered media) capable of supporting vegetation growth
- Filter course layer consisting of aggregate to prevent the migration of fines into uncompacted native soils or the aggregate storage layer
- Aggregate storage layer with underdrain(s)
- Uncompacted native soils at the bottom of the facility
- Overflow structure

Normal Expected Maintenance

Biofiltration with partial retention requires routine maintenance to: remove accumulated materials such as sediment, trash or debris; maintain vegetation health; maintain infiltration capacity of the media layer; replenish mulch; and maintain integrity of side slopes, inlets, energy dissipators, and outlets. A summary table of standard inspection and maintenance indicators is provided within this Fact Sheet.

Non-Standard Maintenance or BMP Failure

If any of the following scenarios are observed, the BMP is not performing as intended to protect downstream waterways from pollution and/or erosion. Corrective maintenance, increased inspection and maintenance, BMP replacement, or a different BMP type will be required.

- The BMP is not drained between storm events. Surface ponding longer than approximately 24 hours following a storm event may be detrimental to vegetation health, and surface ponding longer than approximately 96 hours following a storm event poses a risk of vector (mosquito) breeding. Poor drainage can result from clogging of the media layer, filter course, aggregate storage layer, underdrain, or outlet structure. The specific cause of the drainage issue must be determined and corrected.
- Sediment, trash, or debris accumulation greater than 25% of the surface ponding volume within one month. This means the load from the tributary drainage area is too high, reducing BMP function or clogging the BMP. This would require pretreatment measures within the tributary area draining to the BMP to intercept the materials. Pretreatment components, especially for sediment, will extend the life of components that are more expensive to replace such as media, filter course, and aggregate layers.

- Erosion due to concentrated storm water runoff flow that is not readily corrected by adding erosion control blankets, adding stone at flow entry points, or minor re-grading to restore proper drainage according to the original plan. If the issue is not corrected by restoring the BMP to the original plan and grade, the [City Engineer] shall be contacted prior to any additional repairs or reconstruction.

Other Special Considerations

Biofiltration with partial retention is a vegetated structural BMP. Vegetated structural BMPs that are constructed in the vicinity of, or connected to, an existing jurisdictional water or wetland could inadvertently result in creation of expanded waters or wetlands. As such, vegetated structural BMPs have the potential to come under the jurisdiction of the United States Army Corps of Engineers, SDRWQCB, California Department of Fish and Wildlife, or the United States Fish and Wildlife Service. This could result in the need for specific resource agency permits and costly mitigation to perform maintenance of the structural BMP. Along with proper placement of a structural BMP, **routine maintenance is key to preventing this scenario**.

SUMMARY OF STANDARD INSPECTION AND MAINTENANCE FOR PR-1 BIOFILTRATION WITH PARTIAL RETENTION		
The property owner is responsible to ensure inspection, operation and maintenance of permanent BMPs on their property unless responsibility has been formally transferred to an agency, community facilities district, homeowners association, property owners association, or other special district. Maintenance frequencies listed in this table are average/typical frequencies. Actual maintenance needs are site-specific, and maintenance may be required more frequently. Maintenance must be performed whenever needed, based on maintenance indicators presented in this table. The BMP owner is responsible for conducting regular inspections to see when maintenance is needed based on the maintenance indicators. During the first year of operation of a structural BMP, inspection is recommended at least once prior to August 31 and then monthly from September through May. Inspection during a storm event is also recommended. After the initial period of frequent inspections, the minimum inspection and maintenance frequency can be determined based on the results of the first year inspections.		
Threshold/Indicator	**Maintenance Action**	**Typical Maintenance Frequency**
Accumulation of sediment, litter, or debris	Remove and properly dispose of accumulated materials, without damage to the vegetation or compaction of the media layer.	• Inspect monthly. If the BMP is 25% full* or more in one month, increase inspection frequency to monthly plus after every 0.1-inch or larger storm event. • Remove any accumulated materials found at each inspection.
Obstructed inlet or outlet structure	Clear blockage.	• Inspect monthly and after every 0.5-inch or larger storm event. • Remove any accumulated materials found at each inspection.
Damage to structural components such as weirs, inlet or outlet structures	Repair or replace as applicable.	• Inspect annually. • Maintenance when needed.
Poor vegetation establishment	Re-seed, re-plant, or re-establish vegetation per original plans.	• Inspect monthly. • Maintenance when needed.
Dead or diseased vegetation	Remove dead or diseased vegetation, re-seed, re-plant, or re-establish vegetation per original plans.	• Inspect monthly. • Maintenance when needed.
Overgrown vegetation	Mow or trim as appropriate.	• Inspect monthly. • Maintenance when needed.
2/3 of mulch has decomposed, or mulch has been removed	Remove decomposed fraction and top off with fresh mulch to a total depth of 3 inches.	• Inspect monthly. • Replenish mulch annually, or more frequently when needed based on inspection.

*"25% full" is defined as ¼ of the depth from the design bottom elevation to the crest of the outflow structure (e.g., if the height to the outflow opening is 12 inches from the bottom elevation, then the materials must be removed when there is 3 inches of accumulation – this should be marked on the outflow structure).

SUMMARY OF STANDARD INSPECTION AND MAINTENANCE FOR PR-1 BIOFILTRATION WITH PARTIAL RETENTION (Continued from previous page)		
Threshold/Indicator	**Maintenance Action**	**Typical Maintenance Frequency**
Erosion due to concentrated irrigation flow	Repair/re-seed/re-plant eroded areas and adjust the irrigation system.	• Inspect monthly. • Maintenance when needed.
Erosion due to concentrated storm water runoff flow	Repair/re-seed/re-plant eroded areas, and make appropriate corrective measures such as adding erosion control blankets, adding stone at flow entry points, or minor re-grading to restore proper drainage according to the original plan. If the issue is not corrected by restoring the BMP to the original plan and grade, the [City Engineer] shall be contacted prior to any additional repairs or reconstruction.	• Inspect after every 0.5-inch or larger storm event. If erosion due to storm water flow has been observed, increase inspection frequency to after every 0.1-inch or larger storm event. • Maintenance when needed. If the issue is not corrected by restoring the BMP to the original plan and grade, the [City Engineer] shall be contacted prior to any additional repairs or reconstruction.
Standing water in BMP for longer than 24 hours following a storm event Surface ponding longer than approximately 24 hours following a storm event may be detrimental to vegetation health	Make appropriate corrective measures such as adjusting irrigation system, removing obstructions of debris or invasive vegetation, clearing underdrains, or repairing/replacing clogged or compacted soils.	• Inspect monthly and after every 0.5-inch or larger storm event. If standing water is observed, increase inspection frequency to after every 0.1-inch or larger storm event. • Maintenance when needed.
Presence of mosquitos/larvae For images of egg rafts, larva, pupa, and adult mosquitos, see http://www.mosquito.org/biology	If mosquitos/larvae are observed: first, immediately remove any standing water by dispersing to nearby landscaping; second, make corrective measures as applicable to restore BMP drainage to prevent standing water. If mosquitos persist following corrective measures to remove standing water, or if the BMP design does not meet the 96-hour drawdown criteria due to release rates controlled by an orifice installed on the underdrain, the [City Engineer] shall be contacted to determine a solution. A different BMP type, or a Vector Management Plan prepared with concurrence from the County of San Diego Department of Environmental Health, may be required.	• Inspect monthly and after every 0.5-inch or larger storm event. If mosquitos are observed, increase inspection frequency to after every 0.1-inch or larger storm event. • Maintenance when needed.
Underdrain clogged	Clear blockage.	• Inspect if standing water is observed for longer than 24-96 hours following a storm event. • Maintenance when needed.

References

American Mosquito Control Association.
http://www.mosquito.org/
California Storm Water Quality Association (CASQA). 2003. Municipal BMP Handbook.
https://www.casqa.org/resources/bmp-handbooks/municipal-bmp-handbook
County of San Diego. 2014. Low Impact Development Handbook.
http://www.sandiegocounty.gov/content/sdc/dpw/watersheds/susmp/lid.html
San Diego County Copermittees. 2016. Model BMP Design Manual, Appendix E, Fact Sheet PR-1.
http://www.projectcleanwater.org/index.php?option=com_content&view=article&id=250&Itemid=220

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PR-1
Biofiltration with Partial Retention

Date:	Inspector:	BMP ID No.:
Permit No.:	APN(s):	

Property / Development Name:	Responsible Party Name and Phone Number:
Property Address of BMP:	Responsible Party Address:

INSPECTION AND MAINTENANCE CHECKLIST FOR PR-1 BIOFILTRATION WITH PARTIAL RETENTION PAGE 1 of 5			
Threshold/Indicator	**Maintenance Recommendation**	**Date**	**Description of Maintenance Conducted**
Accumulation of sediment, litter, or debris Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Remove and properly dispose of accumulated materials, without damage to the vegetation ☐ If sediment, litter, or debris accumulation exceeds 25% of the surface ponding volume within one month (25% full*), add a forebay or other pre-treatment measures within the tributary area draining to the BMP to intercept the materials. ☐ Other / Comments:		
Poor vegetation establishment Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Re-seed, re-plant, or re-establish vegetation per original plans ☐ Other / Comments:		

*"25% full" is defined as ¼ of the depth from the design bottom elevation to the crest of the outflow structure (e.g., if the height to the outflow opening is 12 inches from the bottom elevation, then the materials must be removed when there is 3 inches of accumulation – this should be marked on the outflow structure).

Date:	Inspector:	BMP ID No.:
Permit No.:	APN(s):	

INSPECTION AND MAINTENANCE CHECKLIST FOR PR-1 BIOFILTRATION WITH PARTIAL RETENTION PAGE 2 of 5			
Threshold/Indicator	**Maintenance Recommendation**	**Date**	**Description of Maintenance Conducted**
Dead or diseased vegetation Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Remove dead or diseased vegetation, re-seed, re-plant, or re-establish vegetation per original plans ☐ Other / Comments:		
Overgrown vegetation Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Mow or trim as appropriate ☐ Other / Comments:		
2/3 of mulch has decomposed, or mulch has been removed Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Remove decomposed fraction and top off with fresh mulch to a total depth of 3 inches ☐ Other / Comments:		

Date:	Inspector:	BMP ID No.:
Permit No.:	APN(s):	

INSPECTION AND MAINTENANCE CHECKLIST FOR PR-1 BIOFILTRATION WITH PARTIAL RETENTION PAGE 3 of 5			
Threshold/Indicator	**Maintenance Recommendation**	**Date**	**Description of Maintenance Conducted**
Erosion due to concentrated irrigation flow Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Repair/re-seed/re-plant eroded areas and adjust the irrigation system ☐ Other / Comments:		
Erosion due to concentrated storm water runoff flow Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Repair/re-seed/re-plant eroded areas, and make appropriate corrective measures such as adding erosion control blankets, adding stone at flow entry points, or minor re-grading to restore proper drainage according to the original plan ☐ If the issue is not corrected by restoring the BMP to the original plan and grade, the [City Engineer] shall be contacted prior to any additional repairs or reconstruction ☐ Other / Comments:		

Date:	Inspector:	BMP ID No.:
Permit No.:	APN(s):	

INSPECTION AND MAINTENANCE CHECKLIST FOR PR-1 BIOFILTRATION WITH PARTIAL RETENTION PAGE 4 of 5			
Threshold/Indicator	**Maintenance Recommendation**	**Date**	**Description of Maintenance Conducted**
Obstructed inlet or outlet structure Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Clear blockage ☐ Other / Comments:		
Underdrain clogged (inspect underdrain if standing water is observed for longer than 24-96 hours following a storm event) Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Clear blockage ☐ Other / Comments:		
Damage to structural components such as weirs, inlet or outlet structures Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Repair or replace as applicable ☐ Other / Comments:		

Date:	Inspector:	BMP ID No.:
Permit No.:	APN(s):	

INSPECTION AND MAINTENANCE CHECKLIST FOR PR-1 BIOFILTRATION WITH PARTIAL RETENTION PAGE 5 of 5			
Threshold/Indicator	**Maintenance Recommendation**	**Date**	**Description of Maintenance Conducted**
Standing water in BMP for longer than 24 hours following a storm event* Surface ponding longer than approximately 24 hours following a storm event may be detrimental to vegetation health Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Make appropriate corrective measures such as adjusting irrigation system, removing obstructions of debris or invasive vegetation, clearing underdrains, or repairing/replacing clogged or compacted soils ☐ Other / Comments:		
Presence of mosquitos/larvae For images of egg rafts, larva, pupa, and adult mosquitos, see http://www.mosquito.org/biology Maintenance Needed? ☐ YES ☐ NO ☐ N/A	☐ Apply corrective measures to remove standing water in BMP when standing water occurs for longer than 24-96 hours following a storm event.** ☐ Other / Comments:		

*Surface ponding longer than approximately 24 hours following a storm event may be detrimental to vegetation health, and surface ponding longer than approximately 96 hours following a storm event poses a risk of vector (mosquito) breeding. Poor drainage can result from clogging of the media layer, filter course, aggregate storage layer, underdrain, or outlet structure. The specific cause of the drainage issue must be determined and corrected.

**If mosquitos persist following corrective measures to remove standing water, or if the BMP design does not meet the 96-hour drawdown criteria due to release rates controlled by an orifice installed on the underdrain, the [City Engineer] shall be contacted to determine a solution. A different BMP type, or a Vector Management Plan prepared with concurrence from the County of San Diego Department of Environmental Health, may be required.

Maintenance Fact Sheet:

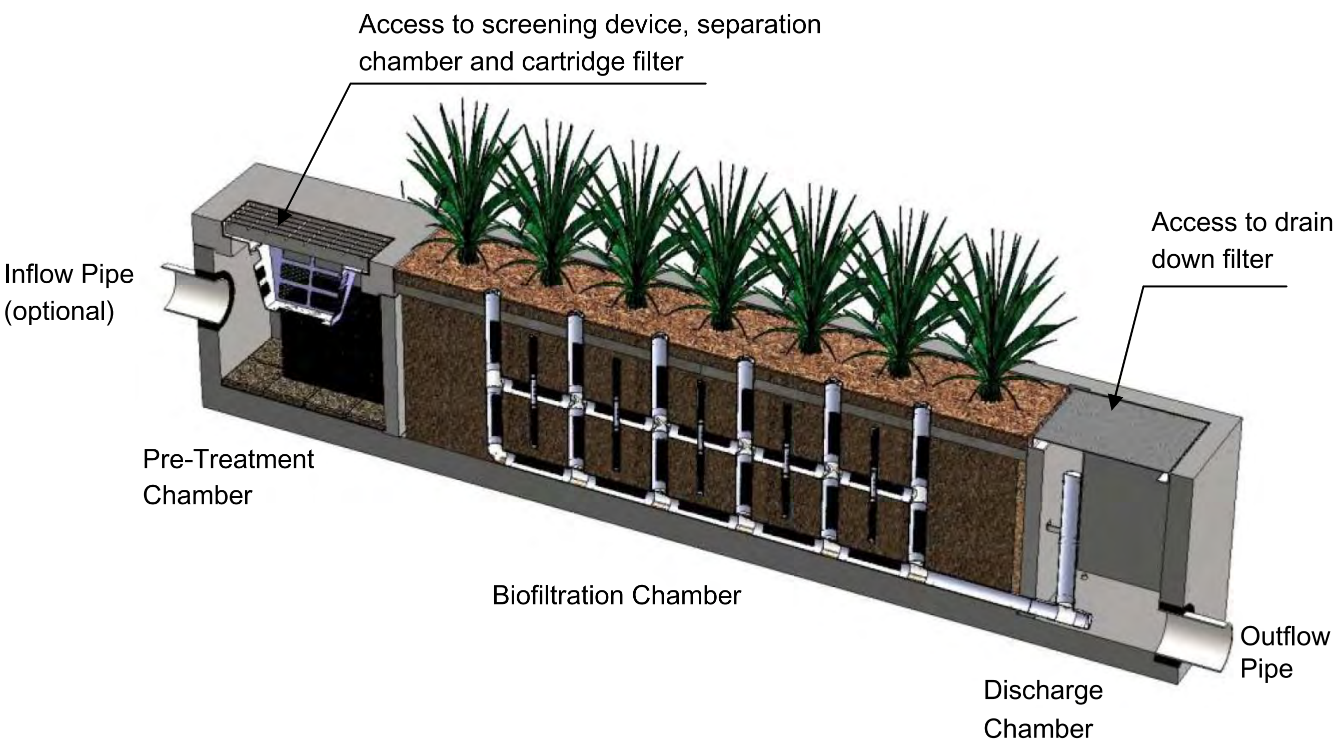
Proprietary Modular Biofiltration

Maintenance Guidelines for
Modular Wetland System - Linear

Maintenance Summary

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

System Diagram



Maintenance Procedures

Screening Device

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

Separation Chamber

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

Cartridge Filters

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

Drain Down Filter

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

Maintenance Notes

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

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

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

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

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

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

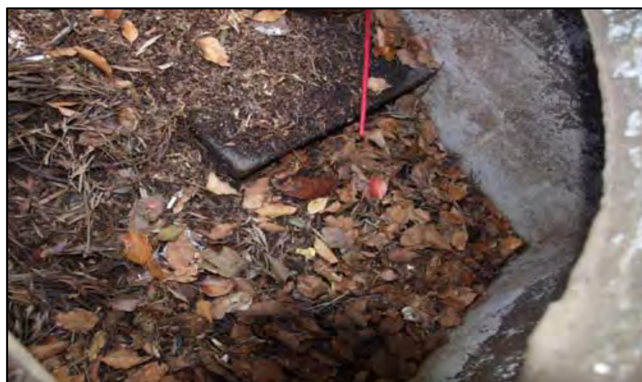
Maintenance Procedure Illustration

Screening Device

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

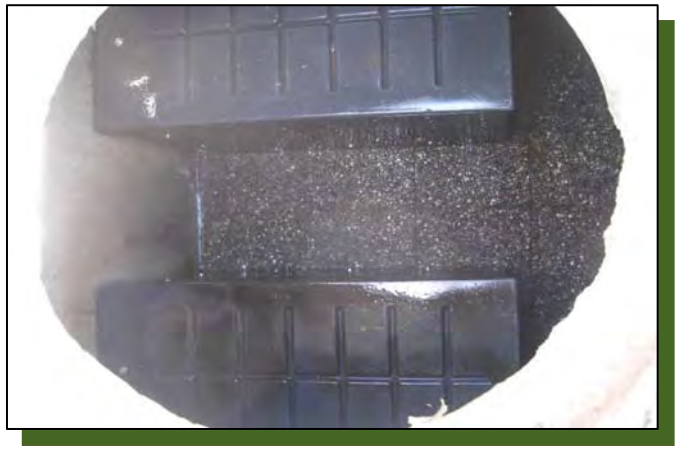
Separation Chamber

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



Cartridge Filters

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



Drain Down Filter

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

Trim Vegetation

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

Inspection Form

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

www.modularwetlands.com

Inspection Report
Modular Wetlands System

Project Name ______________________________

Project Address ______________________________
(city) (Zip Code)

Owner / Management Company ______________________________

Contact ______________________ Phone ( ) –

Inspector Name ______________________ Date ____/____/______ Time __________ AM / PM

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

Weather Condition ______________________ Additional Notes ______________________

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

Inspection Checklist

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

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

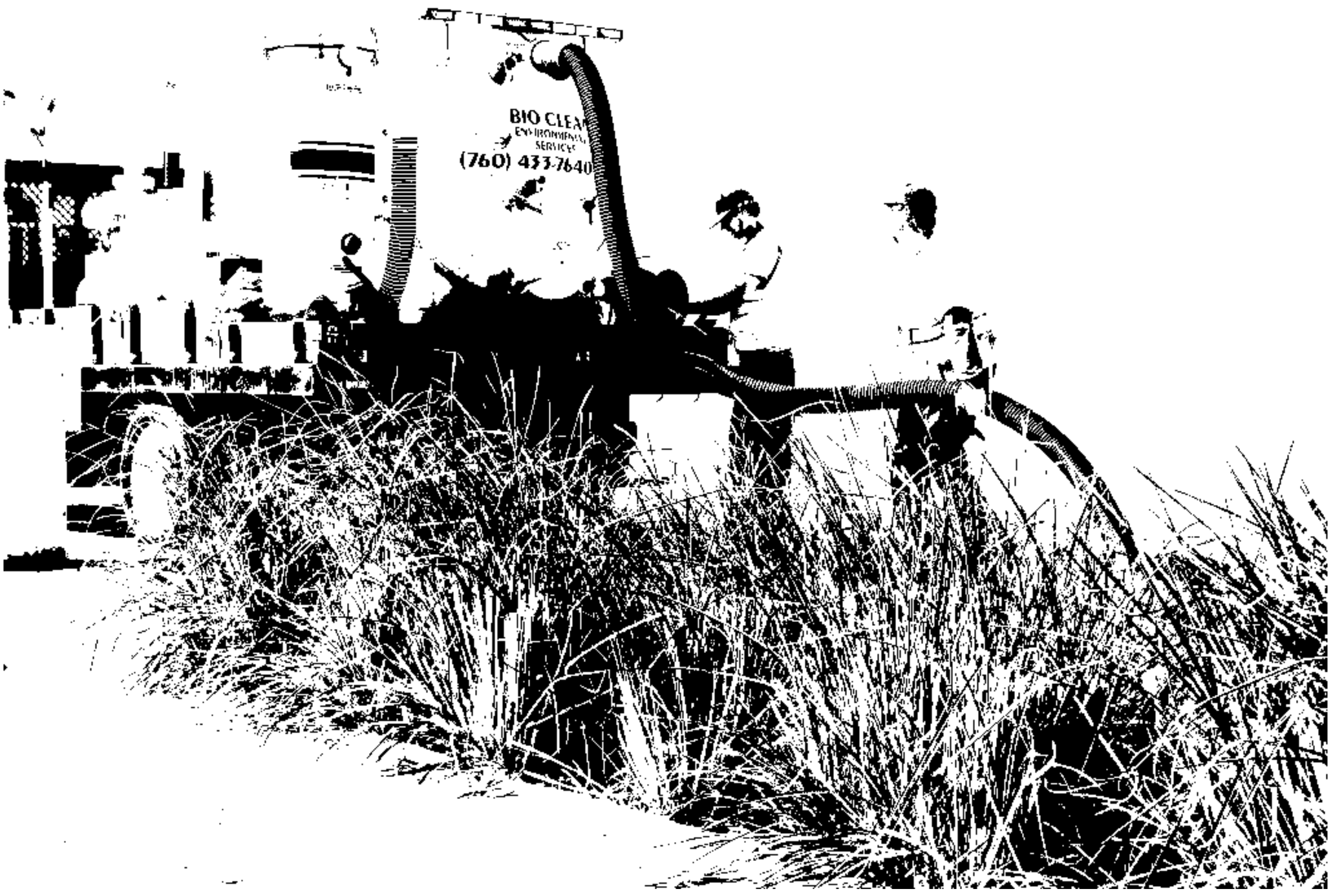
Waste:	Yes	No
Sediment / Silt / Clay		
Trash / Bags / Bottles		
Green Waste / Leaves / Foliage		

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

Plant Information	
Damage to Plants	
Plant Replacement	
Plant Trimming	

Additional Notes: ______________________________

Maintenance Report



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

www.modularwetlands.com

Cleaning and Maintenance Report
Modular Wetlands System

Project Name ______________________________

Project Address ______________________________
(city) (Zip Code)

Owner / Management Company ______________________________

Contact ______________________________ Phone ( ) –

Inspector Name ______________________________ Date ____/____/______ Time __________ AM / PM

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

Weather Condition ______________________________ Additional Notes ______________________________

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

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

Comments:

Attachment 4
Copy of Plan Sheets Showing Permanent Storm Water BMPs

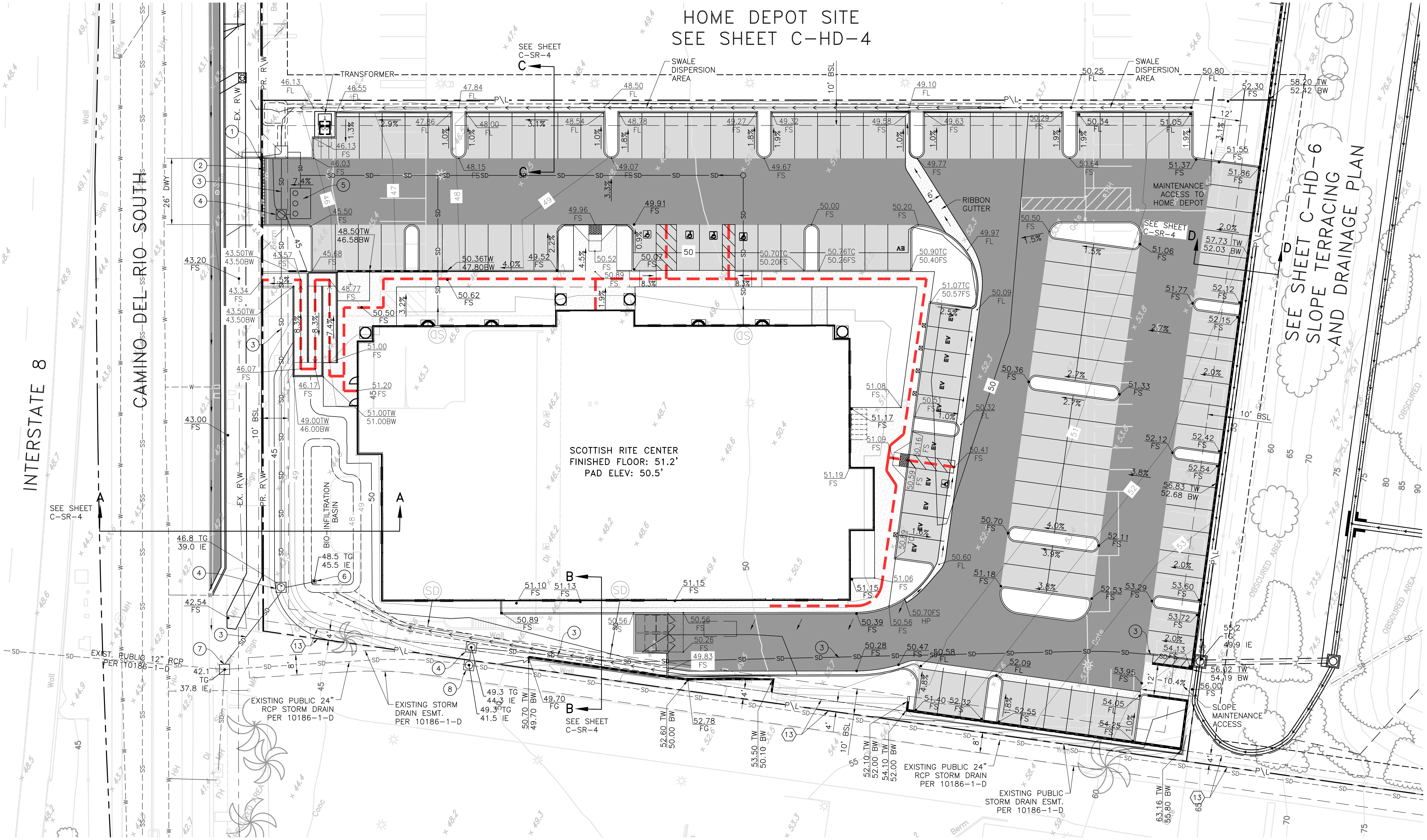
This is the cover sheet for Attachment 4.

Project Name: Home Depot-Scottish Rite (Mission Valley)

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.



BASIS OF BEARING

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QUOTED BEARINGS FROM REFERENCE MAPS OR DEEDS MAY OR MAY NOT BE IN TERMS OF SAID SYSTEM.

THE COMBINED SCALE FACTOR (CSF) AT POINT '970' = 1.0000055. GRID DISTANCE = GROUND DISTANCE X COMBINED SCALE FACTOR. ELEVATION AT POINT 'A' IS 40.96 (NGVD29). THE CONVERGENCE ANGLE AT POINT '970' = −0°29'29.024"

BASIS OF COORDINATES

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STATION #970
N. 1,861,366.50
E. 6,286,787.22
EL. = 40.96 (NGVD29)

BASIS OF ELEVATION

THE BASIS OF ELEVATIONS FOR THIS DRAWING IS THE NORTHWEST BRASS PLUG IN THE TOP OF INLET (NWBP TOP INLET) AT THE INTERSECTION OF CAMINO DEL RIO SOUTH AND MISSION CENTER ROAD AS PUBLISHED IN THE CITY OF SAN DIEGO VERTICAL CONTROL BENCHBOOK.
ELEV. = 52.712 (MSL)

CONSTRUCTION NOTES

1. MODIFIED TYPE B CURB INLET PER SDD-116 WITH F-OPENING ON BACK
2. 8" TRENCH DRAIN
3. 18" HDPE STORM DRAIN
4. STORM DRAIN CLEANOUT (TYPE A) PER D-09
5. 8'x12' MODULAR WETLAND SYSTEM (MWS)
6. 36"x36" BROOKS BOX. TOP OF GRATE SET 6" ABOVE BASIN BOTTOM
7. CONNECT 18" HDPE STORM DRAIN TO EXIST CLEANOUT
8. CONNECT TO EXISTING STORM DRAIN WITH (TYPE A) CLEANOUT PER D-09

LEGEND

- PROPERTY LINE
- CURB & GUTTER
- CURB
- RETAINING WALL
- BROW DITCH
- SWALE
- STORM DRAIN PIPE
- STORM DRAIN STRUCTURES
- ADA PATH

EARTHWORK QUANTITIES

CUT:	3,000 C.Y.
FILL:	4,800 C.Y.
IMPORT:	1,800 C.Y.
MAX CUT:	4.0 FT
MAX FILL:	5.5 FT
MAX SLOPE RATIO:	2:1 (H:V)

NOTE: GRADING QUANTITIES ARE PROVIDED FOR PERMIT PURPOSES ONLY, NOT TO BE USED FOR BIDDING.

SPECIAL NOTES

1. TW AND BW GRADES REPRESENT THE GROUND ELEVATION AT THE BACK AND FRONT OF THE WALL. THE ACTUAL TOP-OF-WALL AND TOP-OF-FOOTING AND WALL DIMENSIONS TO BE DETERMINED BY THE WALL DESIGNER.
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6. ALL STORM DRAINS ARE PRIVATE UNLESS OTHERWISE NOTED.

EASEMENT

13. AN EASEMENT FOR AERIAL AND UNDERGROUND PUBLIC UTILITIES AND INCIDENTAL PURPOSES, RECORDED JANUARY 07, 1957 AS BOOK 6407, PAGE 438 OF OFFICIAL RECORDS.
IN FAVOR OF: SAN DIEGO GAS AND ELECTRIC COMPANY
AFFECTS: AS DESCRIBED THEREIN

SCALE: 1"=20'-0"
0 10 20 40 60 80
NORTH

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

REGISTERED PROFESSIONAL ENGINEER
BRYAN D. SMITH
C 75822
EXP. 06-30-22
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WARE MALCOMB
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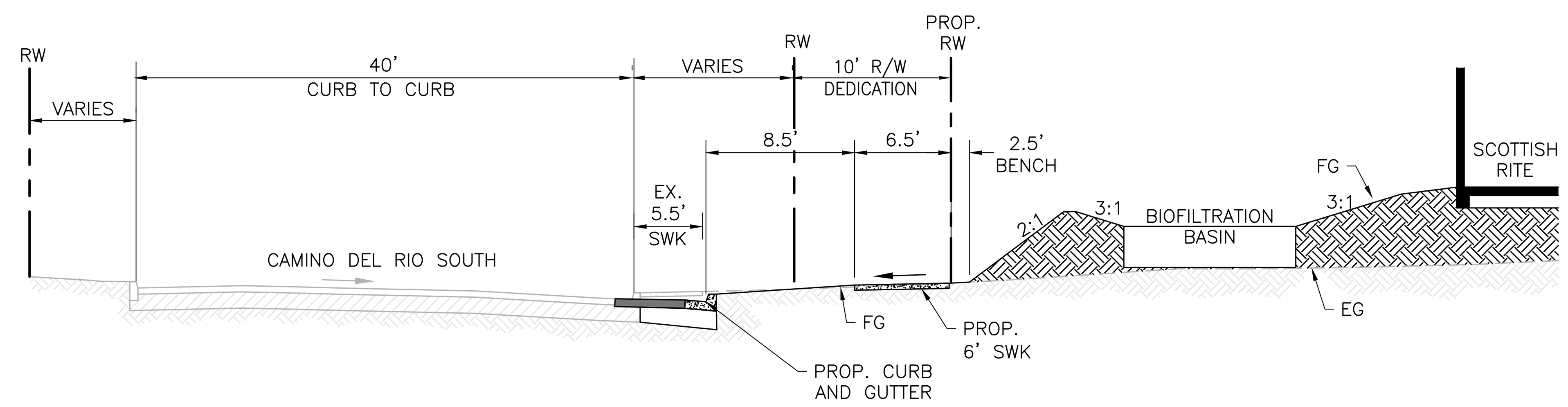
SCOTTISH RITE / HOME DEPOT
1895 & 1561 CAMINO DEL RIO S
SAN DIEGO, CA 92108

GRADING AND DRAINAGE PLAN
C-SR-2
05.27.2020 18134.00



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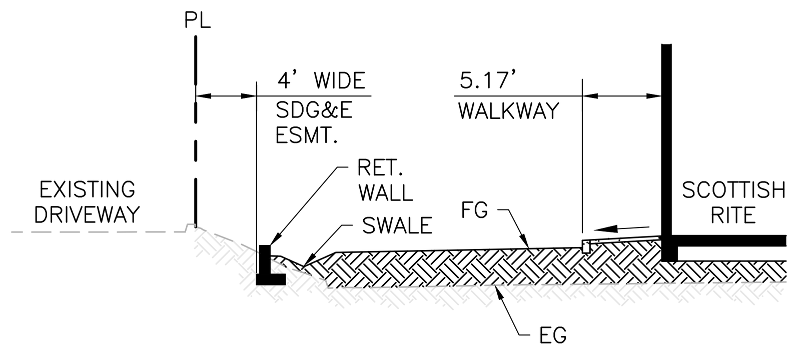

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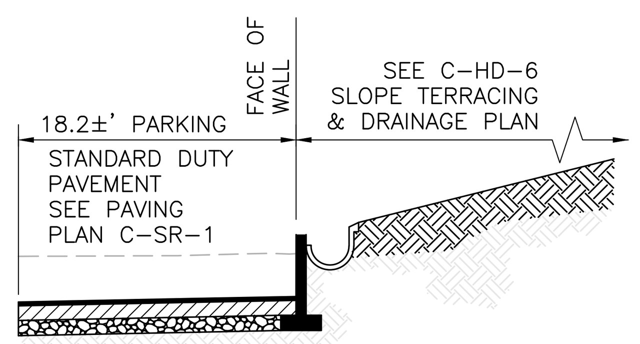

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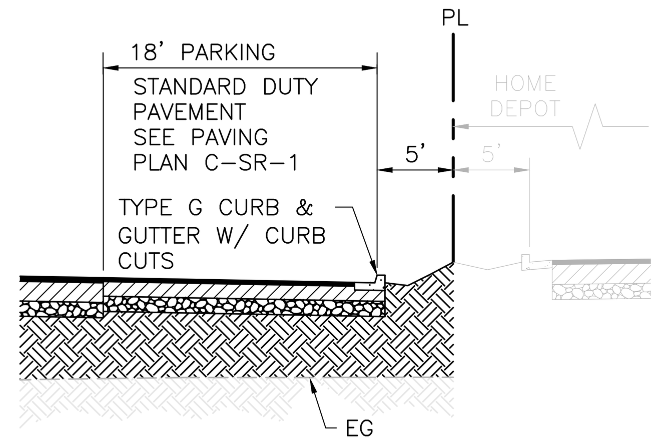

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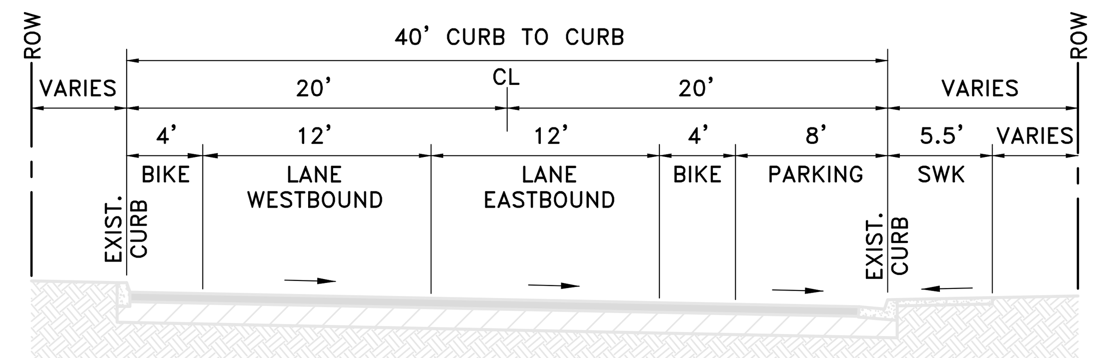
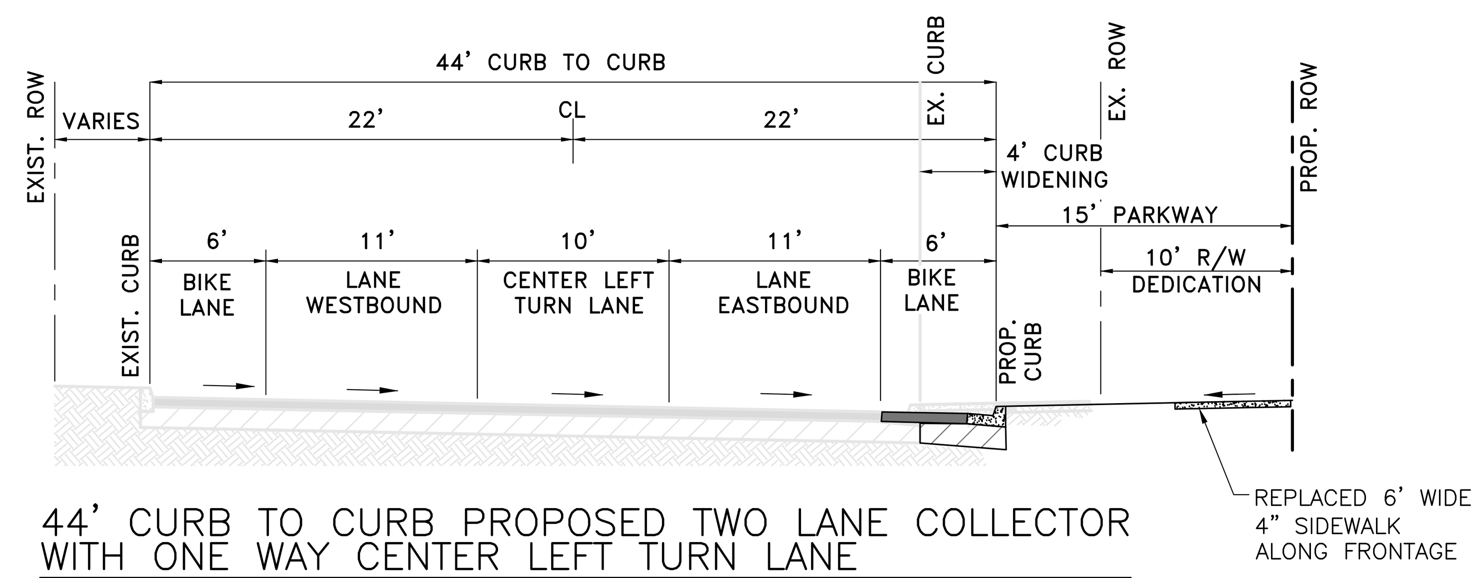


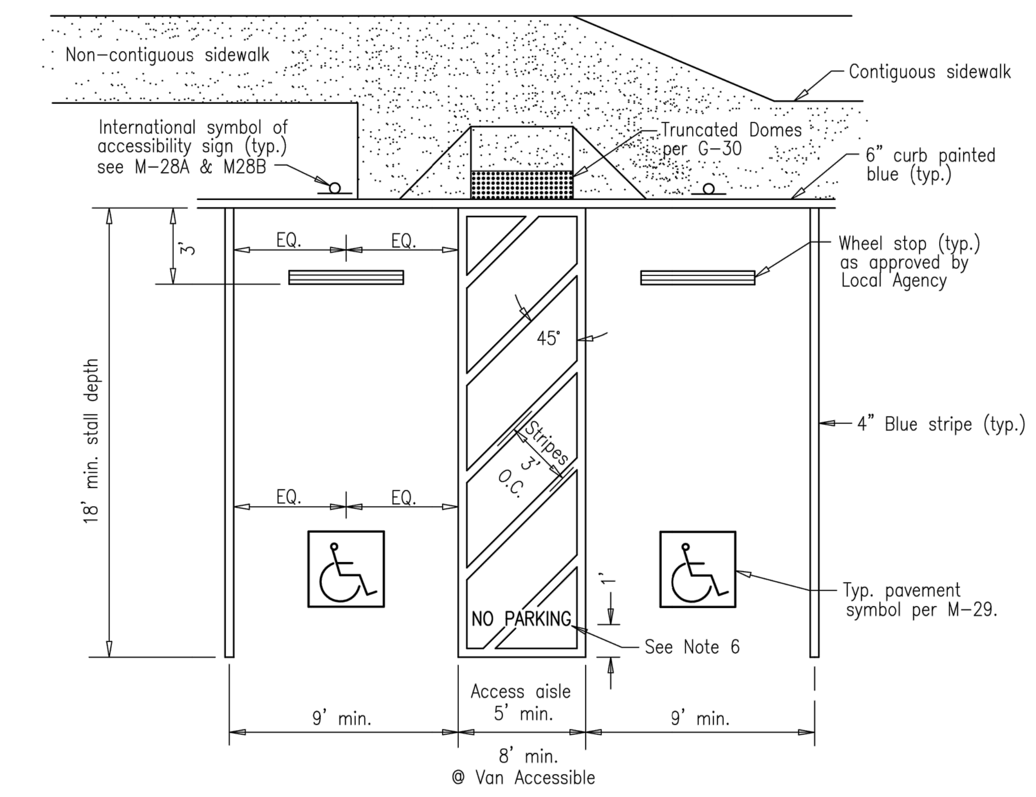
EXISTING TWO LANE COLLECTOR

NOT TO SCALE



44' CURB TO CURB PROPOSED TWO LANE COLLECTOR WITH ONE WAY CENTER LEFT TURN LANE

NOT TO SCALE



NOTES:

1. Provide for adequate drainage.
2. For appropriate ramp alternate to conform to topographical conditions, see standard drawings G–27 through G–32B.
3. Blue color should match color No. 15090 in the Federal Standard 595a as specified in Section 522(b)2.
4. If only one accessible parking stall is going to be provided, the access aisle shall be 8' (van accessible) and located on the passenger side.
5. Sidewalk cross slope shall not exceed 2.0%.
6. "NO PARKING" 12" high stencil marking, reflective white over blue stripes.

Revision	By	Approved	Date
ORIGINAL		Kercheval	12/75
Revised		D. Gerschoffer	09/15
Reviewed		K. Equipado	10/18

SAN DIEGO REGIONAL STANDARD DRAWING

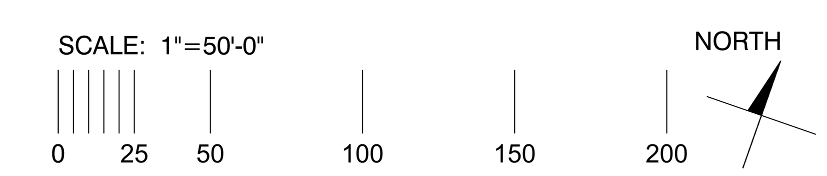
DISABLED PARKING STALL(S)

RECOMMENDED BY THE SAN DIEGO REGIONAL STANDARDS COMMITTEE

Chairperson R.C.E. 19246 Date 10/25/2018

DRAWING NUMBER M–27A

SIGHT DISTANCE DETAIL



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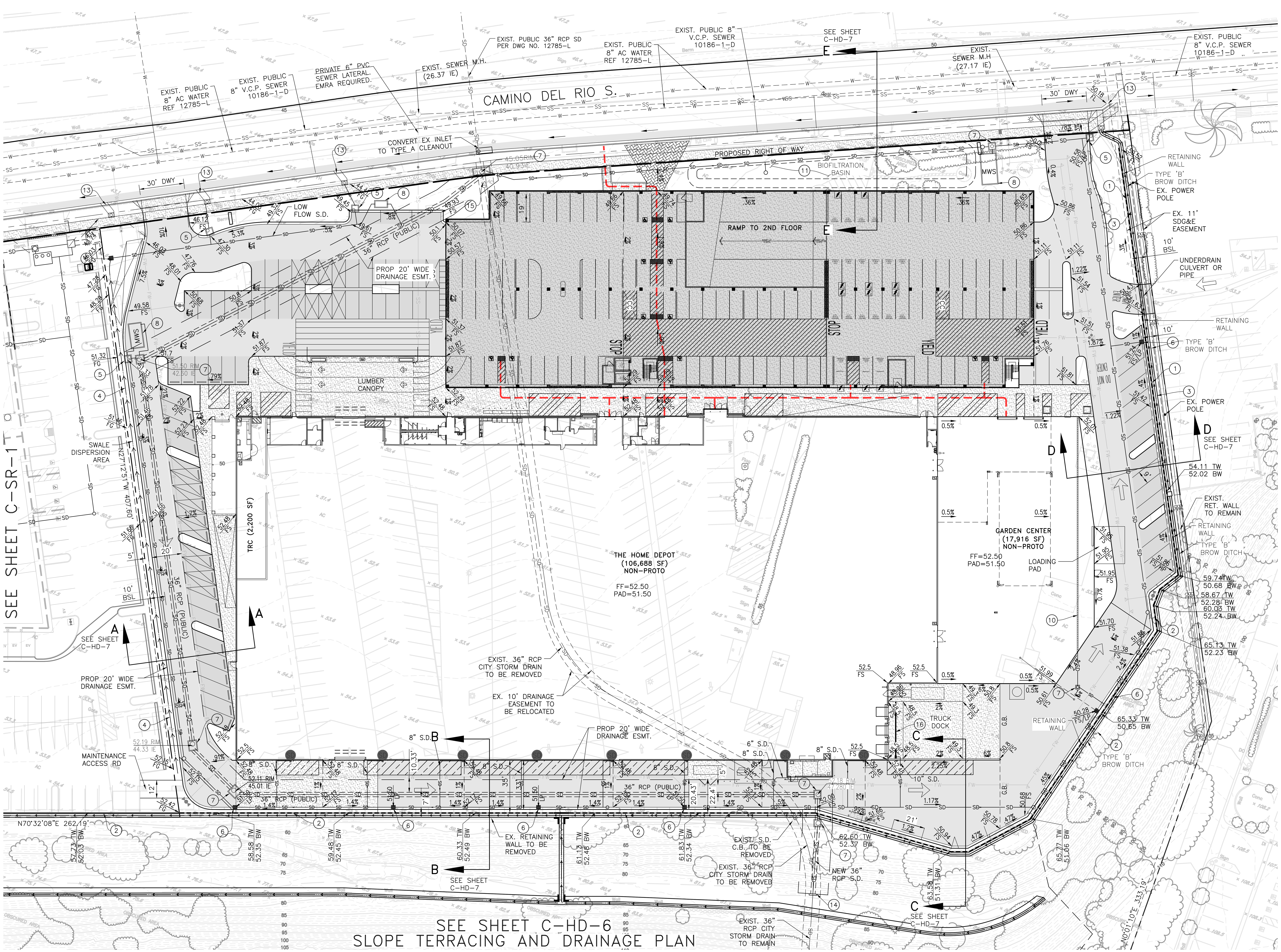
SAN DIEGO, CA 92108

SECTIONS AND DETAILS
C-SR-4
05.27.2020
18134.00



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ELEV. = 52.712 (MSL)

CONSTRUCTION NOTES

1. CONCRETE MASONRY RETAINING WALL, TYPE 5 PER C-5
2. CONCRETE CANTILEVER WALL
3. BROW DITCH PER SDD-106
4. SWALE DISPERSION AREA
5. STORM DRAIN CURB INLET, TYPE A & B PER SDD-115 & 116
6. STORM DRAIN CATCH BASIN, TYPE F PER SDD-119
7. STORM DRAIN CLEANOUT, TYPE A PER D-09
8. UNDERGROUND MWS STORM WATER TREATMENT VAULT
9. STORM DRAIN PIPE
10. TRENCH GRATE
11. BIOFILTRATION BASIN
12. CATCH BASIN RISER WITH GRATED INLET
13. CURB OUTLET PER D-25
14. POINT OF CONNECTION TO REMOVED EXISTING 36" RCP PIPE AT PCR AND CONNECT NEW 36" RCP PIPE.
15. POINT OF CONNECTION TO INSTALL STORM DRAIN CLEAN OUT AND REMOVED SOUTHERLY PORTION OF EXISTING 36" RCP.
16. TRENCH DRAIN WITH EJECTOR PUMP TO BACKBONE STORM DRAIN LINE

LEGEND

PROPERTY LINE
CURB & GUTTER
CURB
RETAINING WALL
BROW DITCH
SWALE
STORM DRAIN PIPE
STORM DRAIN STRUCTURES
ADA PATH

EARTHWORK QUANTITIES

CUT:	41,000 C.Y.
FILL:	35,500 C.Y.
EXPORT:	5,500 C.Y.
MAX CUT:	14.5 FT
MAX FILL:	4.1 FT
MAX SLOPE RATIO:	1.75:1 (H:V)

NOTE: GRADING QUANTITIES ARE PROVIDED FOR PERMIT PURPOSES ONLY, NOT TO BE USED FOR BIDDING.

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SCALE: 1"=30'-0"
0 15 30 60 90 120
NORTH

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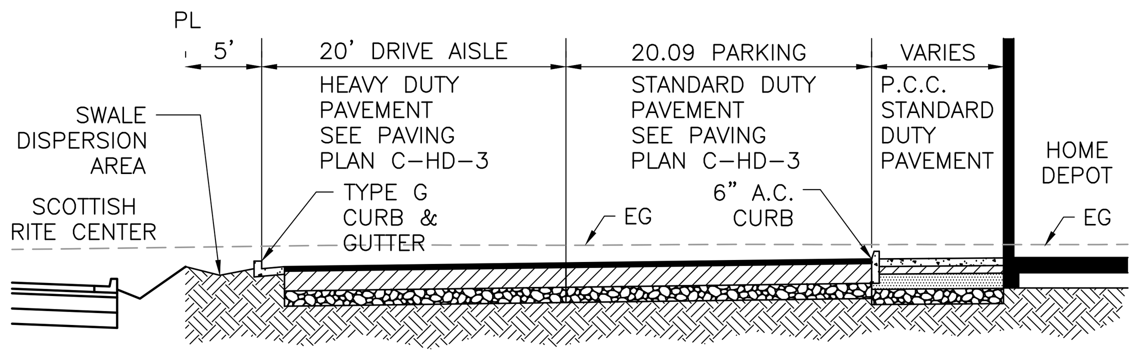
C-HD-4
05.27.2020
18134.00



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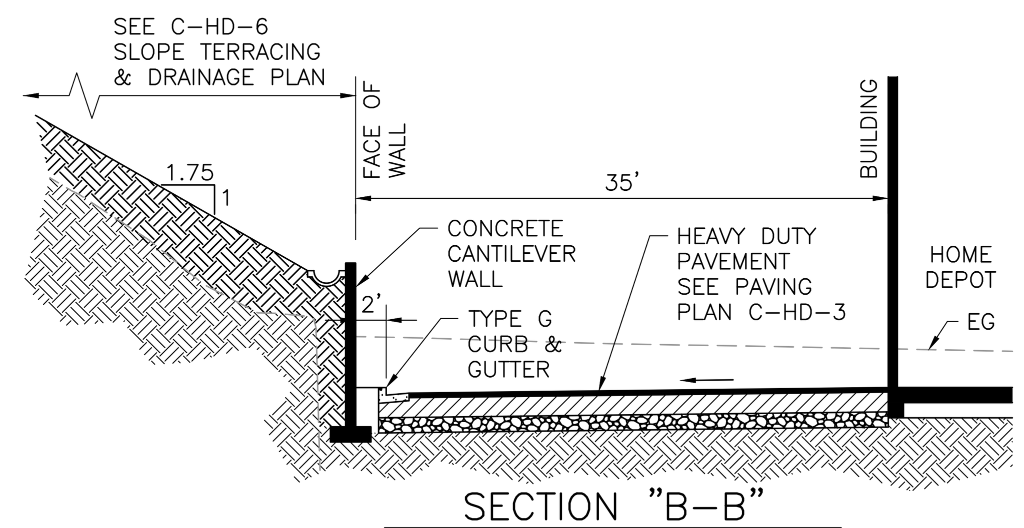

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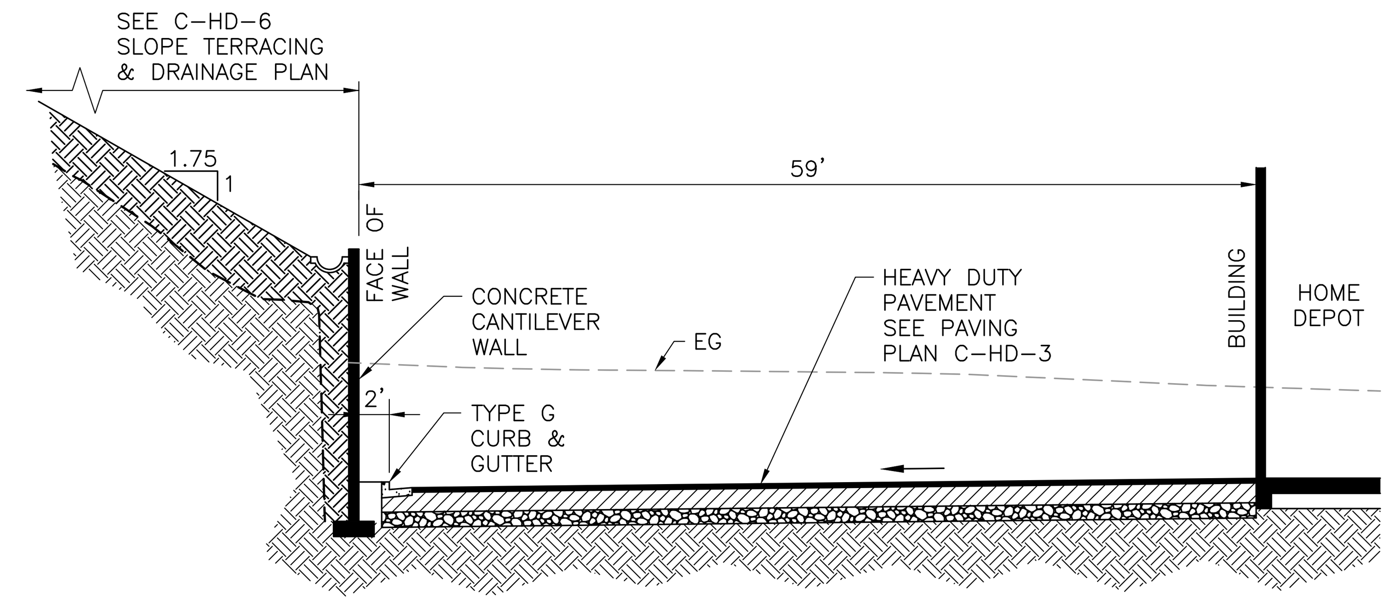

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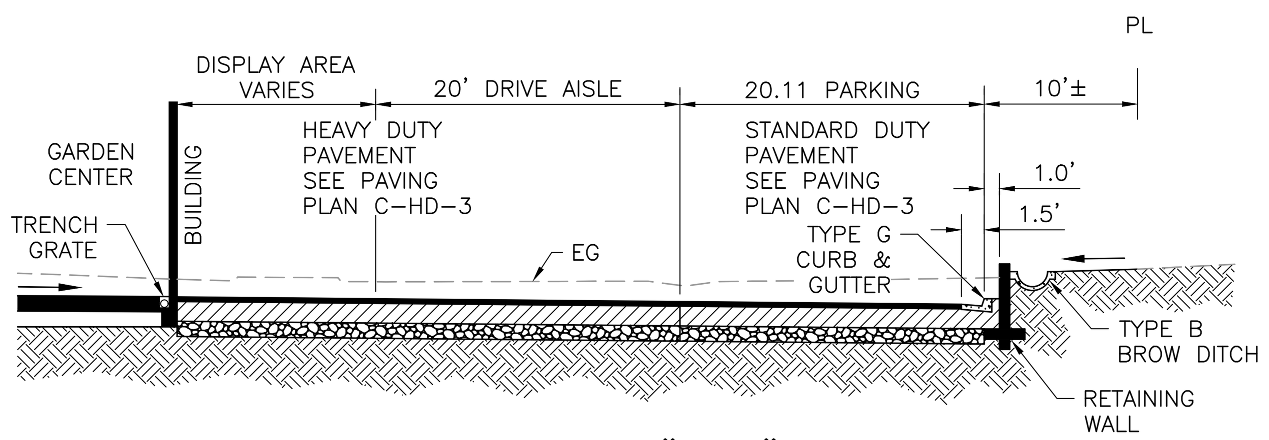

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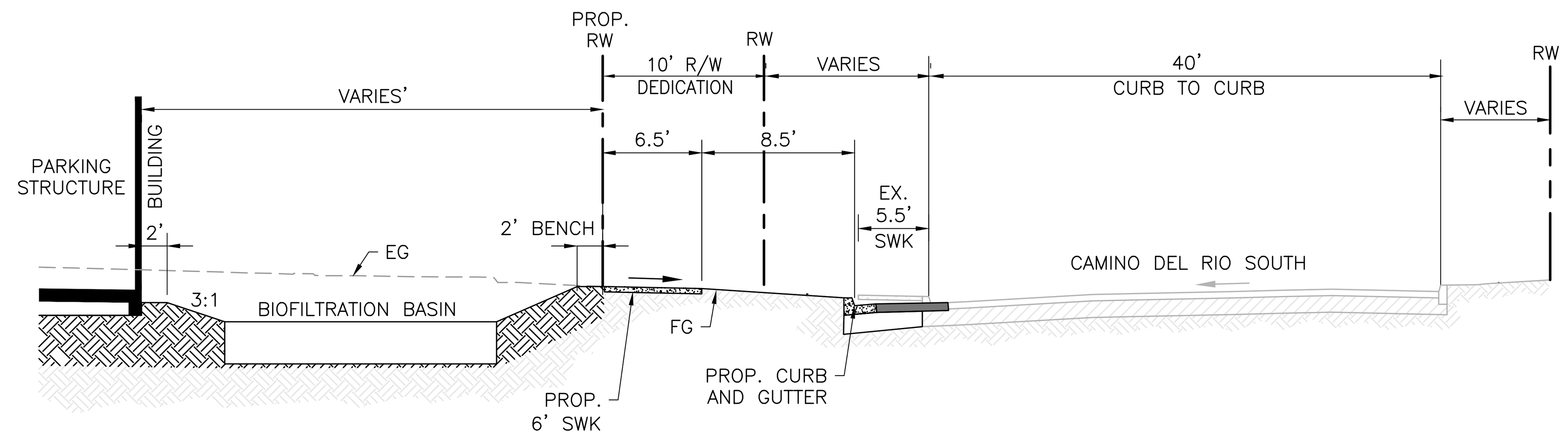

SECTION "E-E"

NOT TO SCALE

EASTERN TRANSITION

SCALE: 1"=20'-0"

0 10 20 40 60 80

NORTH

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SAN DIEGO, CA 92108

SECTIONS AND DETAILS
C-HD-7
05.27.2020
18134.00

Attachment 5
Drainage Report

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

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FUSCOE ENGINEERING

DRAINAGE STUDY

HOME DEPOT-SCOTTISH RITE (MISSION VALLEY)

PREPARED FOR

HOME DEPOT U.S.A., INC.
C/O BOB BURNSIDE
4000 W. METROPOLITAN DR. SUITE 100
ORANGE, CA 92868

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

PROJECT MANAGER
BRYAN D. SMITH, P.E.

DATE PREPARED: APRIL 2020

full circle thinking®

PRELIMINARY DRAINAGE STUDY

THE HOME DEPOT AND SCOTTISH RITE

SAN DIEGO, CA
PTS 657591

Prepared by Brianne VanGorder Under the Responsible Charge of:

Bryan Smith, PE 75822 EXP: 06-30-2022

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

MAY 27, 2020

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

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Table of Contents

Introduction

The purpose of this drainage study is to present the preliminary drainage design to support the entitlement process for the proposed Scottish Rite and the Home Depot sites located in the Mission Valley area of San Diego, CA. The criteria used for this drainage study is in accordance with the City of San Diego Drainage Design Manual 2017 (Hydraulics) and the County of San Diego Hydrology Manual 2003 (Hydrology).

This study examines the existing and proposed hydrology of the including any associated offsite drainage areas considered part of the local watershed and presents preliminary design for the project drainage facilities.

Fuscoe Engineering has taken over as the Engineer of Work from a previous preliminary Drainage Study prepared by San Dieguito Engineering (SDE) dated Jan. 14, 2020. This study has utilized information from the previous report and updated it to address plan check comments and design changes.

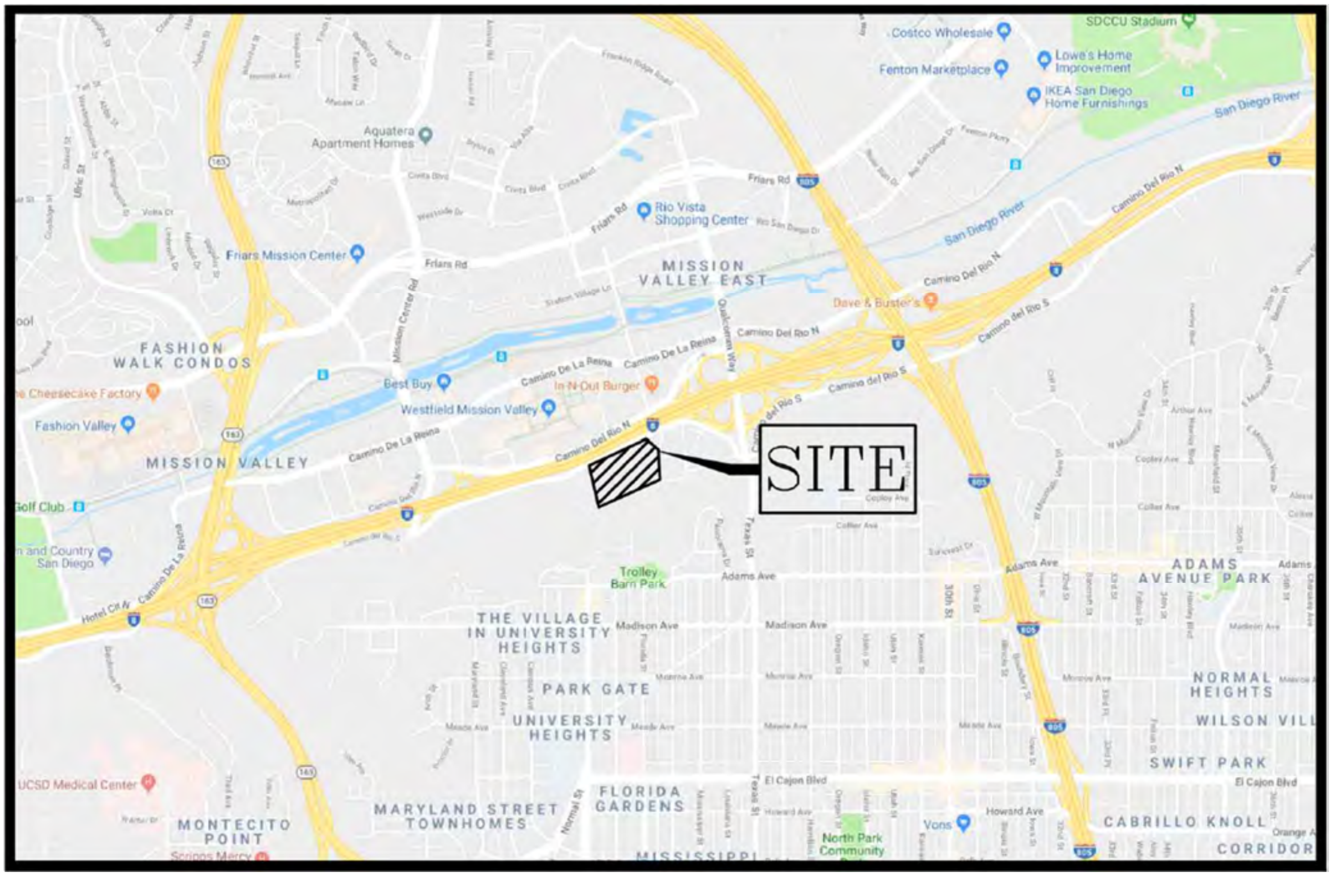
Section 1 – Project Information

1.1 Project Description

The site analyzed within this drainage study encompasses the development of the future the Scottish Rite Center (SRC) and the Home Depot (THD) located within the Mission Valley area of the City of San Diego, CA. The project is located along the southern side of Camino del Rio South, approximately 0.5 miles east of Mission Center Road and 0.3 miles west of Texas Street. See next page for a vicinity map of the site.

The project proposes to construct two building structures for the Scottish Rite Center and a Home Depot. Both sites will also include associated parking and landscaped areas. In addition, THD will include a two-level garage structure located adjacent to Camino del Rio South. Driveways from Camino del Rio South will provide access to the sites.

None of the proposed activities or structures require Regional Water Quality Control Board 401 Certification, U.S. Army Corps of Engineers 404 permit or approval by California Department of Fish and Game.


SITE
MISSION VALLEY EAST
MISSION VALLEY
FASHION WALK CONDOS
THE VILLAGE IN UNIVERSITY HEIGHTS
PARK GATE
UNIVERSITY HEIGHTS
FLORIDA GARDENS
MARYLAND STREET TOWNHOMES
MONTECITO POINT
ADAMS AVENUE PARK
NORMAL HEIGHTS
SWIFT PARK
CABRILLO KNOLL
CORRIDOR
Camino Del Rio N
Camino del Rio S
Friars Rd
San Diego River
Adams Ave
Texas St
El Cajon Blvd
Madison Ave

1.2 Hydrologic Setting

The project site is located in the Mission San Diego Hydrologic Sub-Area (907.11) which is within the Lower San Diego Hydrologic Area within the San Diego Hydrologic Unit Watershed. Exhibit 2 illustrates the project site in the context of the watershed.

1.2.1 Topography

The site is located on the south side of Camino del Rio South and drains from the rear of site towards Camino del Rio South. A grate inlet within the existing Scottish Rite Center parking lot collects a portion of the site runoff. The remaining site runoff sheet flows towards Camino del Rio South. The highest point of the property is located at the rear of the lot (southern boundary) with an elevation of about 57 feet. The property drains over a distance of approximately 475 feet to Camino del Rio South with an average grade of approximately 2.5 percent.

1.2.2 Current and Adjacent Land Use

The eastern portion of the project site is current Scottish Rite Center while the western portion of the site is a car dealership. There are slopes along the entire rear of the site which slope towards the site but whose runoff is collected via terraced channels and catch basins. The existing site is accessed by driveways along Camino del Rio South.

The general plan designation for the area is Commercial Employment, Retail, & Services. The property is zoned as Commercial (Office and limited Industrial with an auto orientation). The assessor's parcel numbers (APNs) are 4380903300 and 4380903400.

Exhibit 3 illustrates the County of San Diego General Plan and Zoning near the project site.

1.2.3 Soil and Vegetation Conditions

A preliminary report of soil investigation was prepared for the project by Moore Twining titled Geotechnical Engineering Investigation- Proposed Home Depot Store- Mission Valley (June 3, 2019). The Soil Survey for the San Diego Area by the United States Department of Agriculture Soil Conservation Service (1973) was also used for reference.

Infiltration testing indicated a vertical infiltration rate ranging from 0.1 to 0.5 inches per hour without any factor of safety applied.

The USDA Soil Survey classifies the site soils as Urban land which is assumed to be soil type 'D'. This is consistent with the clay soils identified in the geotechnical report over the project site. Exhibit 4 illustrates the soil types and limits. The flat portions of the site contain minimal if any vegetation. However, the slopes located along the rear of the lot are Terrace escarpments with natural vegetation consisting of grass and bushes.

1.2.4 Existing Drainage Patterns and Facilities

The existing Scottish Rite Center consists of a building and parking lot. Drainage from a portion of the parking lot is collected by an onsite grate and storm drain then conveyed towards the storm drain system along Camino del Rio South. The eastern portion of the site overland flows towards Camino del Rio South where it then flows east within the gutter and is ultimately collected by an existing curb inlet about 500 feet east of the site. The remaining portion of the site sheet flows towards Camino del Rio South before getting collected by the existing curb inlet in front of the site.

The existing auto dealer site will be the future location of the new Scottish Rite Center. Drainage from the backside of the lot is collected via onsite grates and storm pipe which discharge at the from the lot and drain onto Camino del Rio South. The front of the lot sheet flows over land towards Camino del Rio South. Runoff along the Camino del Rio South gutter at this location empties into an existing curb inlet located approximately 70 feet west of the western site boundary.

1.2.5 Floodplain Mapping

The Federal Emergency Management Agency (FEMA) has mapped Special Flood Hazard Areas (SFHAs) for the project site. The project site is within an area on the FEMA Flood Insurance Rate Map designated as an area of minimal flood hazard (Zone X). A Firmette was produced from the FEMA website taken from FIRM Map Panel Number 06073C1619G which is included in **Exhibit 5**.

1.2.6 Downstream Conditions

Analysis of downstream conditions has not been performed as part of this drainage study, as the project site plan has been designed to mitigate storm water flows to below pre-development levels.

1.2.7 Impervious Cover

The site is almost entirely covered with impervious surfaces consisting of roofs, sidewalks, parking lot, and driveway. There are minimal areas with vegetated areas consisting of landscaping around the existing Scottish Rite Center. The project will remove all of the impervious areas and replace them with new impervious and pervious surfaces. However, it is expected that the proposed site will ultimately reduce the amount of impervious surfaces relative to existing condition.

Section 2 – Methodology and Design Criteria

The design criteria and methodology for Hydrology follow the County of San Diego Hydrology Manual (June 2003) and the Hydraulics for Underground Storm Drain Design follow the City of San Diego Drainage Design Manual (2017) as appropriate for the project site.

2.1 Rational Method

Rational Method Peak Flows were calculated using methodology in the County of San Diego Hydrology Manual for the rational method via AES software. These calculations were performed for both the existing and proposed conditions to quantify differences in the peak rate of discharge. Runoff coefficients were based upon researched soils data and Table 3-1 of the County Hydrology Manual for the existing condition. The proposed condition assumed a Type D Soil, and calculated the weighted runoff coefficient based on the percentage of impervious for each subarea. Time of concentration was calculated per Section 3.1.4 of the County Hydrology Manual and corresponding runoff intensities for the 100-year storm were based upon a 6-hour precipitation of 2.6 inches. See Appendix C for Pre-project and Post-project rational method hydrology calculations.

The peak runoff rates in proposed conditions result in a slight decrease in peak runoff due to a decrease in impervious areas. The facilities have been designed to meet flow control and treatment control criteria per calculations in the SWQMP.

Storm water treatment to mitigate for pollutants will be provided through the various BMPs throughout the site. This mitigation will be provided through biofiltration with either basin-type structures or with modular proprietary devices. Narrative and calculations relative to stormwater treatment are included in the project's Storm Water Quality Management Plan (SWQMP).

2.2 Hydraulic Analysis

The proposed 36in public storm drain is designed in accordance with Section 4.1 of the City of San Diego Drainage Design Manual. StormCAD by Bentley Systems was used to analyze the hydraulic capacity of this storm drain. Output tables and profiles are provided in Appendix D of this report. At the connection point to the existing 36in public storm drain, a tailwater was assumed in order to model the downstream existing hydraulic grade line for the 50-year storm event. Because downstream HGL data was not available, this tailwater was set at the existing top of curb elevation as a conservative measure. Assuming the downstream tailwater and flows generated at the nodes described above, the 36in public storm drain will convey the design flow while maintaining at least 1 foot of freeboard from the hydraulic grade line below the ground surface in accordance with Section 4.1.1 of the Design Manual.

Node 219, 220, and 262 represents the rerouted public storm drainage system that goes through the existing Scottish Rite site. Node 262 represents the ultimate downstream condition prior to connecting into the existing 36in RCP culvert per drawing number 12785-L. The offsite drainage and mitigated onsite flow rates are less than the existing condition at node 262 per **Table 3-1.**

Section 3 – Hydrologic Effect of Project

This section summarizes the quantities and location of storm water runoff from the project site. Discussion of the water quality aspects of the project can be found in the PDP-SWQMP, which is under separate cover from this report.

3.1 Drainage Patterns

The grading and lot line revisions associated with the proposed site will affect the drainage patterns relative to the existing condition drainage. Therefore, the proposed drainage facilities for the site will reconfigure the existing drainage patterns to best mimic and match existing peak flows at the site's three compliance points which are designated on the hydrology maps.

3.2 Impervious Cover

As in existing condition, the majority of the combined THD/SRC site will consist of impervious surfaces such as of roof, sidewalk, parking lot, driveway, and parking garage structure. However, the amount of pervious surfaces for landscaping will be increased as indicated in the site plan and proposed condition hydrology map in **Appendix A**.

3.3 Peak Runoff

The project will not increase the peak 100-year storm discharge from the site at all three compliance points along Camino del Rio South. Refer to Nodes 145, 262, and 310 on the proposed conditions hydrology maps in **Appendix A.**

Table 3-1 on the next page summarizes the hydrologic effects in terms of calculated peak runoff from the project watershed under both existing and proposed conditions. Nodes at points of drainage discharge from the project pre- and post-development (corresponding with **Table 3-1**) are labeled on the hydrology maps in **Appendix A.**

Table 3-1 Summary of Hydrology Analysis

	Existing Condition			Proposed Condition			Summary	
Location	Node	Area (ac)	Q_{100} (cfs)	Node	Area (cfs)	Q_{100} (cfs)	Area Delta (ac)	Q_{100} Delta
West	132	15.0	37.20	145	15.8	36.85	+0.8	-0.35
Central	237	35.6	71.31	262	36.6	70.37	+1.0	-0.94
East	302	3.0	10.15	310	1.6	4.94	-1.4	-5.21
TOTALS:	-	53.52	118.66	-	53.55	112.16	0.4	-6.50

3.4 Project Erosion and Sedimentation

Because runoff over erodible surfaces will be restricted to flows over the individual lots and vegetated cut and fill slopes, and because the proposed grading will limit the flows and velocities of runoff generated, neither erosion or sedimentation are anticipated. Velocities over the proposed lots will be decreased from the existing condition to non-erosive levels. Once flows have exited each lot, the flows are conveyed via impervious surfaces (gutters and storm drain pipes) not subject to erosion.

Section 4 – Summary and Conclusions

This section provides a summary discussion of the potential effects of the proposed project on local water resources in terms of quantity and location.

- The proposed project will not increase the calculated 100-year peak flows towards any of the sites three compliance points.
- There are no City of San Diego Master Plan drainage facilities shown in the approved General Plan that would affect the project.
- The project will not affect the capacity of existing offsite drainage facilities. The project will remove or replace any existing onsite drainage improvements and all storm drainage pipes and facilities will be designed during the Final Engineering phase to convey the 50-year peak flows without causing flooding of proposed structures.
- The rerouted public storm drainage pipe has been designed for 50-year peak flows and meets the City of San Diego design guidelines.

Section 5 – CEQA Summary

This section summarizes the results of the hydrology, hydraulics and drainage analysis in the context of CEQA significance guidelines.

5.1 Drainage

5.1.1 Erosion and/or Sedimentation

Would the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, in a manner that would result in substantial erosion or siltation on- or off-site?

The project will not alter existing drainage patterns of the site area in a manner that would result in substantial erosion or sedimentation. The project does not alter the course of a stream or river.

- Flows may be concentrated at certain locations, including storm drain outfalls, however, all proposed outfalls will be to non-erosive surfaces. Other storm water Best Management Practices (BMPs) will help preclude significant erosion and/or siltation on- and off-site.

5.1.2 Flooding

Does the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river, or substantially increase the rate or amount of surface runoff in a manner that would result in flooding on- or off-site?

The project will not alter existing drainage patterns of the site area in a manner that would result in flooding on- or off-site. The project does not alter the course of a stream or river.

- This drainage study demonstrates that the project will not increase the 100-year peak storm discharge, as compared with existing conditions.

5.1.3 Drainage System Capacity

Does the project create or contribute runoff water that would exceed the capacity of existing or planned storm water drainage systems?

The project will not create or contribute runoff water that would exceed the capacity of existing or planned storm water drainage systems.

- The project will not affect any City master-planned drainage facilities
- All proposed drainage facilities will be designed to accommodate the 50-Year storm

5.2 Flood Hazards

5.2.1 Residential Flood Hazard

Would the project place housing within a 100-year flood hazard area as mapped on a federal Flood Hazard Boundary or Flood Insurance Rate Map or other flood hazard delineation map, including County Floodplain Maps?

The project does not propose to locate any housing within the 100-year flood hazard area.

- The project does not propose any development within the 100-year floodplain or other Special Flood Hazard Area (SFHA) designated by FEMA

5.2.2 Flood Flow

Does the project place within a 100-year flood hazard area structures that would impede or redirect flood flows?

The project does not propose to locate any structures or grading in the floodplain that would impede or redirect flood flows.

- The project does not propose any development within the 100-year floodplain or other Special Flood Hazard Area (SFHA) designated by FEMA

5.2.3 Flood Hazard

Does the project expose people or structures to a significant risk of loss, injury or death involving flooding, including flooding as a result of a levee or dam?

The project does not place any people or structures at significant risk of loss, injury or death due to flooding.

- The project does not propose any development within the 100-year floodplain or other Special Flood Hazard Area (SFHA) designated by FEMA
- The project will ensure emergency access during significant flood events. The project is not located behind a levee or below a dam that would present a flood hazard upon its failure.

5.2.4 Other Hazards

Is the project at significant risk of inundation by seiche, tsunami, or mudflow?

The project is not located in an area at risk of inundation by seiche (lake slosh), tsunami, or mudflow.

5.3 Waiver and Release Agreements

The project does not alter downstream flow characteristics significantly, either due to increase in flow or flood condition, diversion of flow, or flow concentration. Therefore, it should not be necessary to obtain waiver and release agreements from any affected property owners.

Section 6 – References

San Diego County Hydrology Manual (June 2003), County of San Diego Department of Public Works Flood Control.

San Diego County Hydraulic Design Manual (September 2014), County of San Diego Department of Public Works Flood Control Section.

San Diego County Drainage Design Manual (December 1973), County of San Diego Department of Public Works Flood Control Section.

Soil Conservation Service (December 1973). Soil Survey, San Diego Area, California

EXHIBIT 1
VICINITY MAP

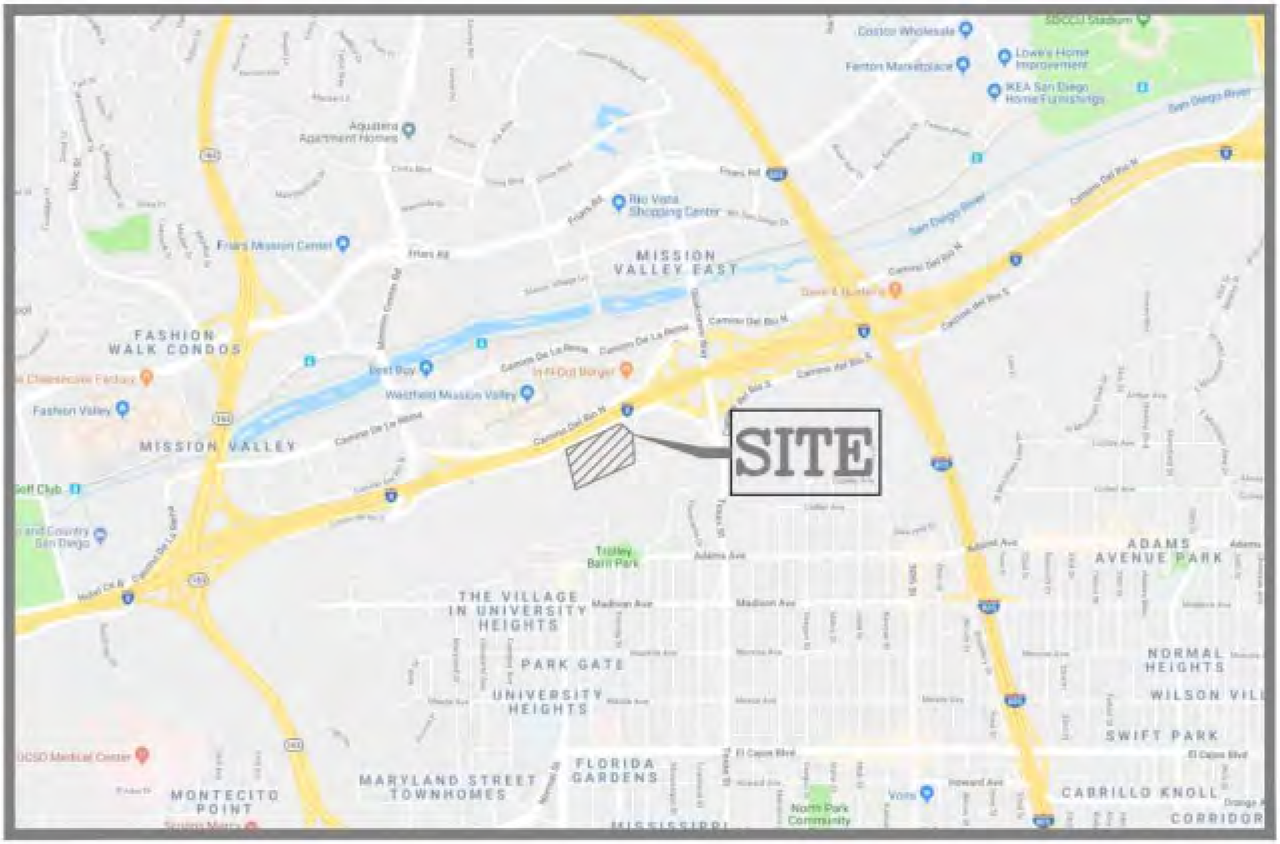


EXHIBIT 2
WATERSHED VICINITY MAP

Caltrans Water Quality Planning Tool

The Water Quality Planning Tool was created to help planners and designers comply with environmental permits. It uses a map interface to find information based on a project's location. **This application is being updated for digital accessibility and will continue to function while updates are in progress.**

Layers

- ☐ 303(d) List and TMDLs 2014-2016 (Legend)
- ☐ Areas of Special Biological Significance
- ☐ Arid and Semi-Arid Regions
- ☐ Caltrans Districts
- ☐ Caltrans Facilities
- ☐ Caltrans Tier 1 Monitoring Sites
- ☐ Calwater Watersheds
- ☐ Coastal Zone
- ☐ Counties
- ○ Geologic Map (Legend)
- ☐ High Risk Receiving Watersheds
- ☐ Monthly Precipitation
- ☐ MS4 Areas
- ☐ Post Miles
- ☐ Rainfall Distribution
- ☐ RWQCB Boundaries
- ○ USGS Topo Maps
- ☐ Watershed Boundary Dataset

Risk Level Determination

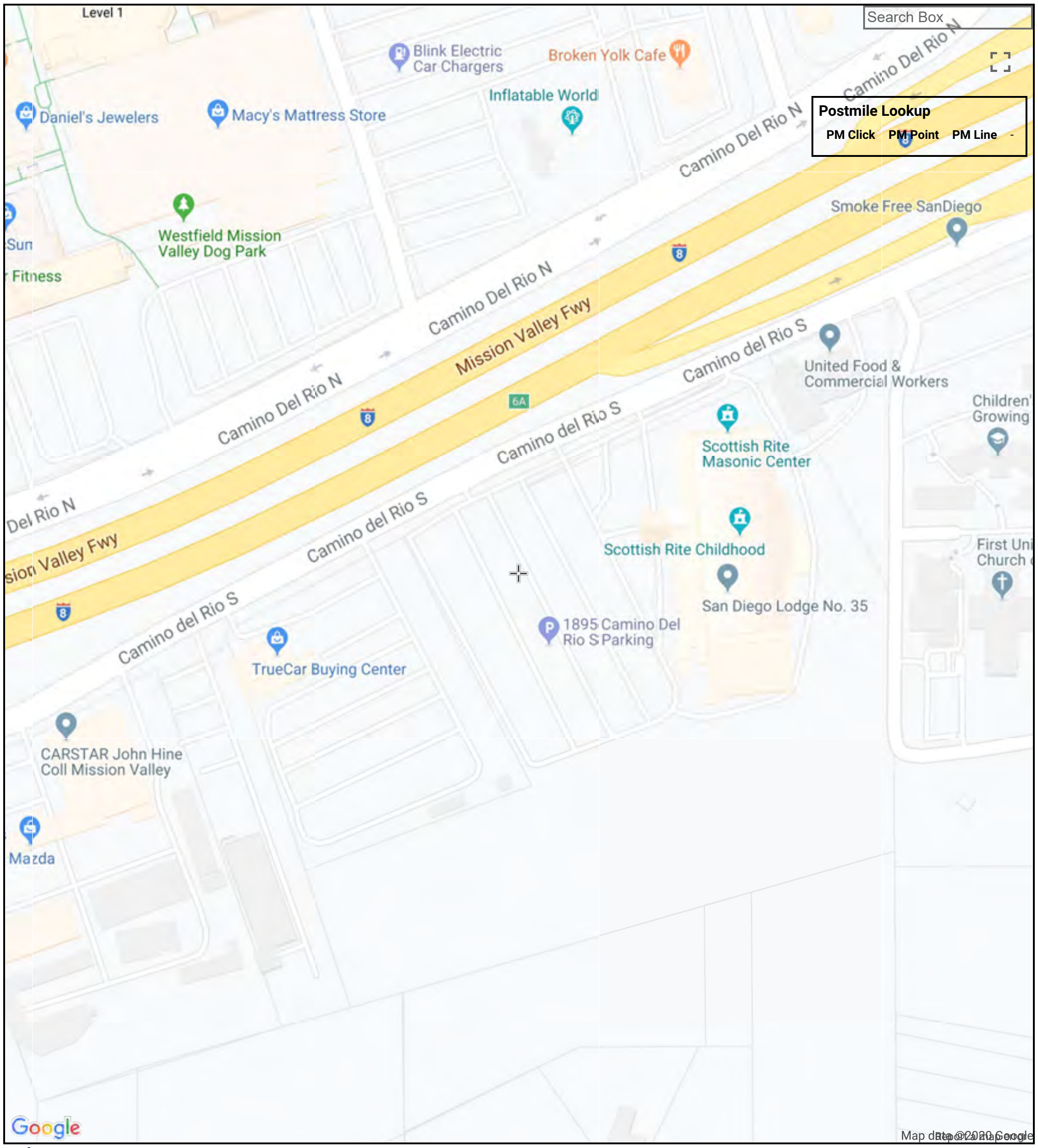
- ☐ Erosivity Index
- ☐ K Factor
- ☐ R Factor (calculations)
- ○ LS Factor
- ☐ Soil Details

Compliance Storm Events

- ○ 5-yr 24-hr North
- ○ 10-yr 24-hr North
- ○ 5-yr 24-hr South
- ○ 10-yr 24-hr South

Distance and Area

Information
Hover over a layer name for a description. Additional information, tables, coordinates, and links are below the map.
Help



Watershed Information

CALWATER WATERSHED

Hydrologic Unit	SAN DIEGO	Hydrologic Area	Lower San Diego	Hydrologic Sub-Area #	907.11
Hydrologic Sub-Area Name	Mission San Diego	Planning Watershed	4907110000	HSA Area (acres)	37059
Latitude, Longitude	32.7672, -117.1449				

WATERSHED BOUNDARY DATASET

Watershed	Lower San Diego River	Subwatershed	Mission Valley-San Diego River	Hydrologic Unit Code	180703040705
Average Annual Precipitation (inches)	12.74				

TMDLs & 303(d) Listed Water Bodies (2014 - 2016 List)

Key: Water body on 303(d) list Water body with a TMDL

Name	Pollutant	Size	Status

907.00	SAN DIEGO HYDROLOGIC UNIT
907.10	Lower San Diego HA
7.11	Mission San Diego HSA
7.12	Santee HSA
7.13	El Cajon HSA
7.14	Coches HSA
7.15	El Monte HSA
907.20	San Vicente HA
7.21	Fernbrook HSA
7.22	Kimball HSA
7.23	Gower HSA
7.24	Barona HSA
907.30	El Capitan HA
7.31	Conejos Creek HSA
7.32	Glen Oaks HSA
7.33	Alpine HSA
907.40	Boulder Creek HA
7.41	Inaja HSA
7.42	Spencer HSA
7.43	Cuyamaca HSA

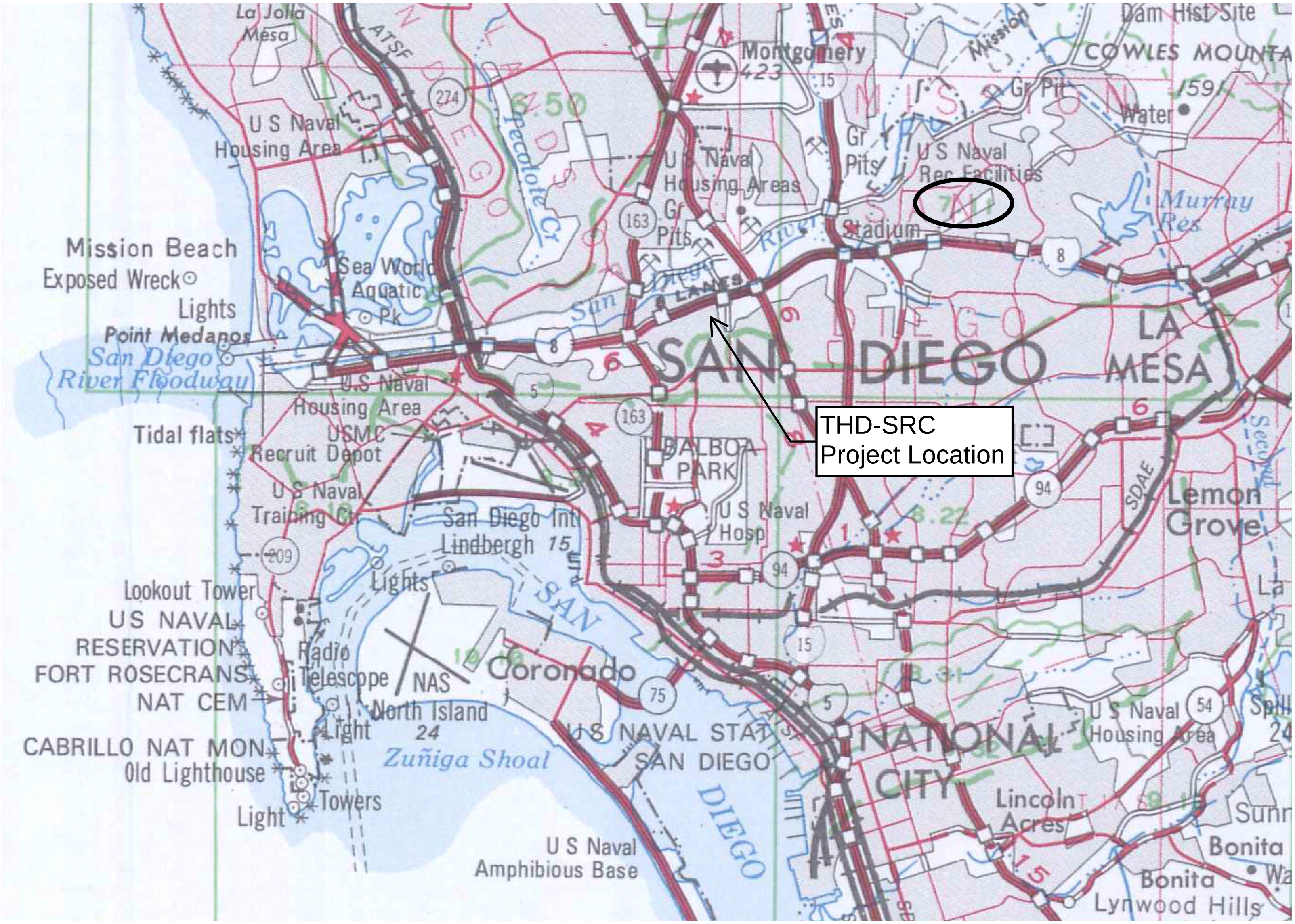

THD-SRC
Project Location
SAN DIEGO
LA MESA
Lemon Grove
NATIONAL CITY
Coronado
Mission Beach
Exposed Wreck
Point Medanos
San Diego River Floodway
Sea World Aquatic Pk
U S Naval Housing Area
Tidal flats
USMC Recruit Depot
U S Naval Training Ctr
San Diego Intl Lindbergh
Lookout Tower
US NAVAL RESERVATION
FORT ROSECRANS NAT CEM
CABRILLO NAT MON
Old Lighthouse
Radio Telescope
NAS North Island
Zuñiga Shoal
U S NAVAL STA SAN DIEGO
U S Naval Amphibious Base
BALBOA PARK
U S Naval Hosp
Montgomery
Stadium
U S Naval Rec Facilities
Murray Res
COWLES MOUNTA
Dam Hist Site
Lincoln Acres
Bonita
Lynwood Hills
La Jolla Mesa
Tecolote Cr
San Diego River

EXHIBIT 3
COUNTY GENERAL PLAN LAND USE/ZONING MAP

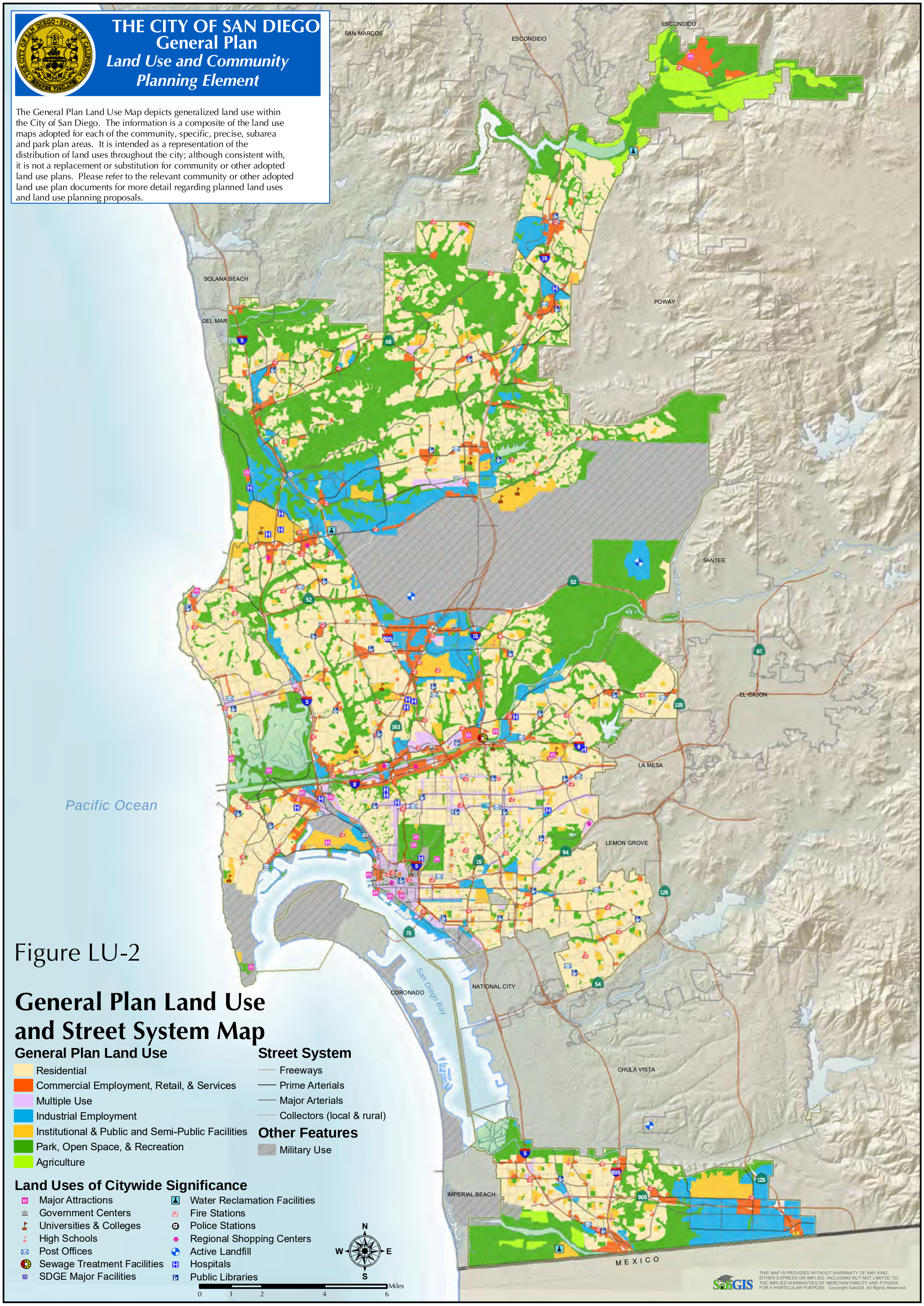

THE CITY OF SAN DIEGO
General Plan
Land Use and Community Planning Element
The General Plan Land Use Map depicts generalized land use within the City of San Diego. The information is a composite of the land use maps adopted for each of the community, specific, precise, subarea and park plan areas. It is intended as a representation of the distribution of land uses throughout the city; although consistent with, it is not a replacement or substitution for community or other adopted land use plans. Please refer to the relevant community or other adopted land use plan documents for more detail regarding planned land uses and land use planning proposals.
SAN MARCOS
ESCONDIDO
SOLANA BEACH
DEL MAR
POWAY
SANTEE
EL CAJON
LA MESA
LEMON GROVE
Pacific Ocean
San Diego Bay
CORONADO
NATIONAL CITY
CHULA VISTA
IMPERIAL BEACH
MEXICO
Figure LU-2
General Plan Land Use and Street System Map
General Plan Land Use
Residential
Commercial Employment, Retail, & Services
Multiple Use
Industrial Employment
Institutional & Public and Semi-Public Facilities
Park, Open Space, & Recreation
Agriculture
Street System
Freeways
Prime Arterials
Major Arterials
Collectors (local & rural)
Other Features
Military Use
Land Uses of Citywide Significance
Major Attractions
Government Centers
Universities & Colleges
High Schools
Post Offices
Sewage Treatment Facilities
SDGE Major Facilities
Water Reclamation Facilities
Fire Stations
Police Stations
Regional Shopping Centers
Active Landfill
Hospitals
Public Libraries
N
W
E
S
0 1 2 4 6 Miles
SanGIS
THIS MAP IS PROVIDED WITHOUT WARRANTY OF ANY KIND, EITHER EXPRESS OR IMPLIED, INCLUDING BUT NOT LIMITED TO, THE IMPLIED WARRANTIES OF MERCHANTABILITY AND FITNESS FOR A PARTICULAR PURPOSE. Copyright SanGIS. All Rights Reserved.

EXHIBIT 4
SOIL TYPES

Hydrologic Soil Group—San Diego County Area, California
(THD- SRC)

Map Scale: 1:2,770 if printed on A landscape (11" x 8.5") sheet.

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 14, Sep 16, 2019

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 7, 2014—Jan 4, 2015

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
RkC	Reiff fine sandy loam, 5 to 9 percent slopes	A	0.0	0.1%
TeF	Terrace escarpments		0.7	4.0%
Ur	Urban land		15.7	95.9%
Totals for Area of Interest			**16.4**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

EXHIBIT 5
FEMA FLOOD INSURANCE RATE MAP

National Flood Hazard Layer FIRMette

FEMA

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 1/2/2020 at 11:11:33 AM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

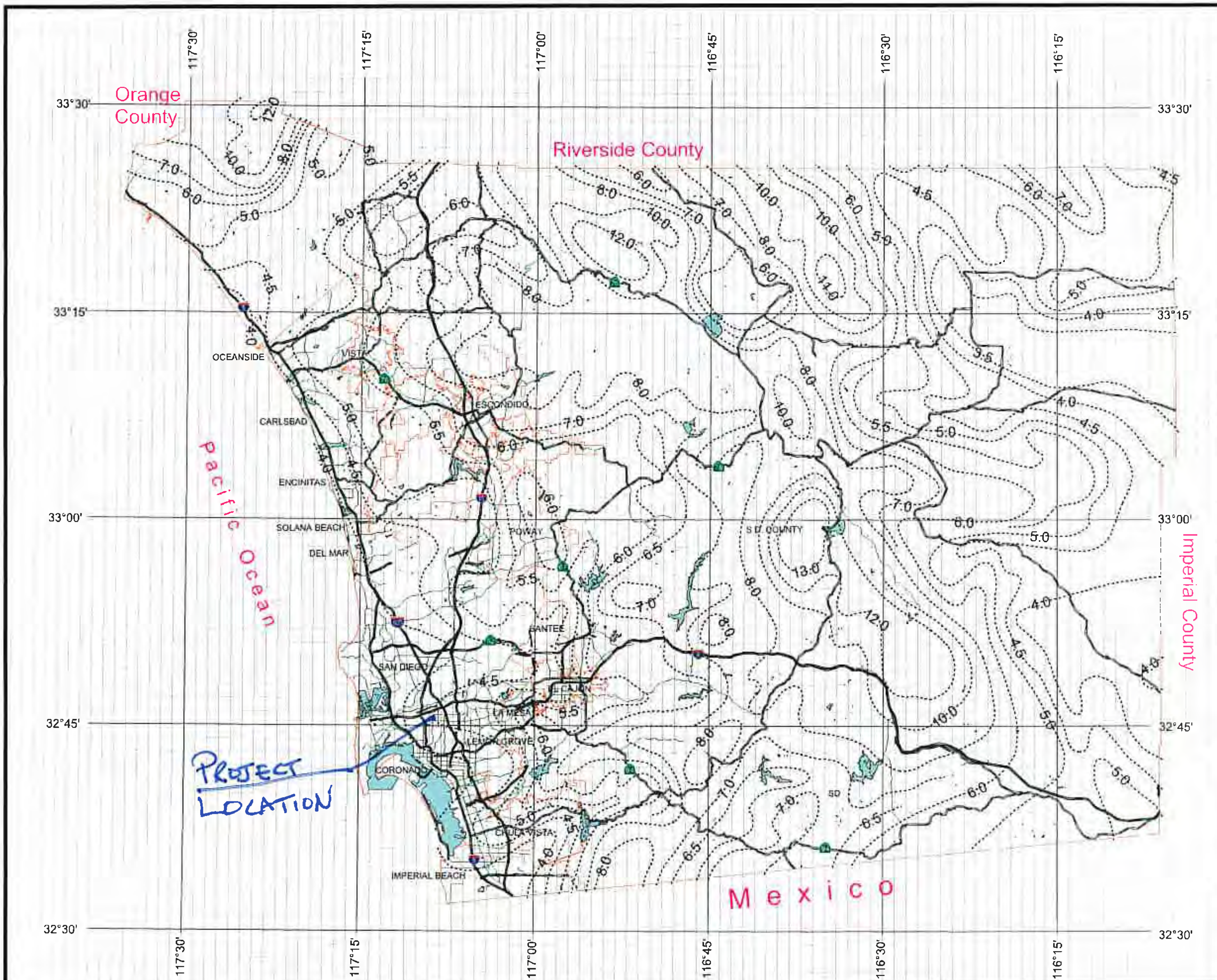
This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Appendix A
SAN DIEGO COUNTY HYDROLOGY ISOPLUVIAL MAPS AND EXCERPTS

County of San Diego Hydrology Manual

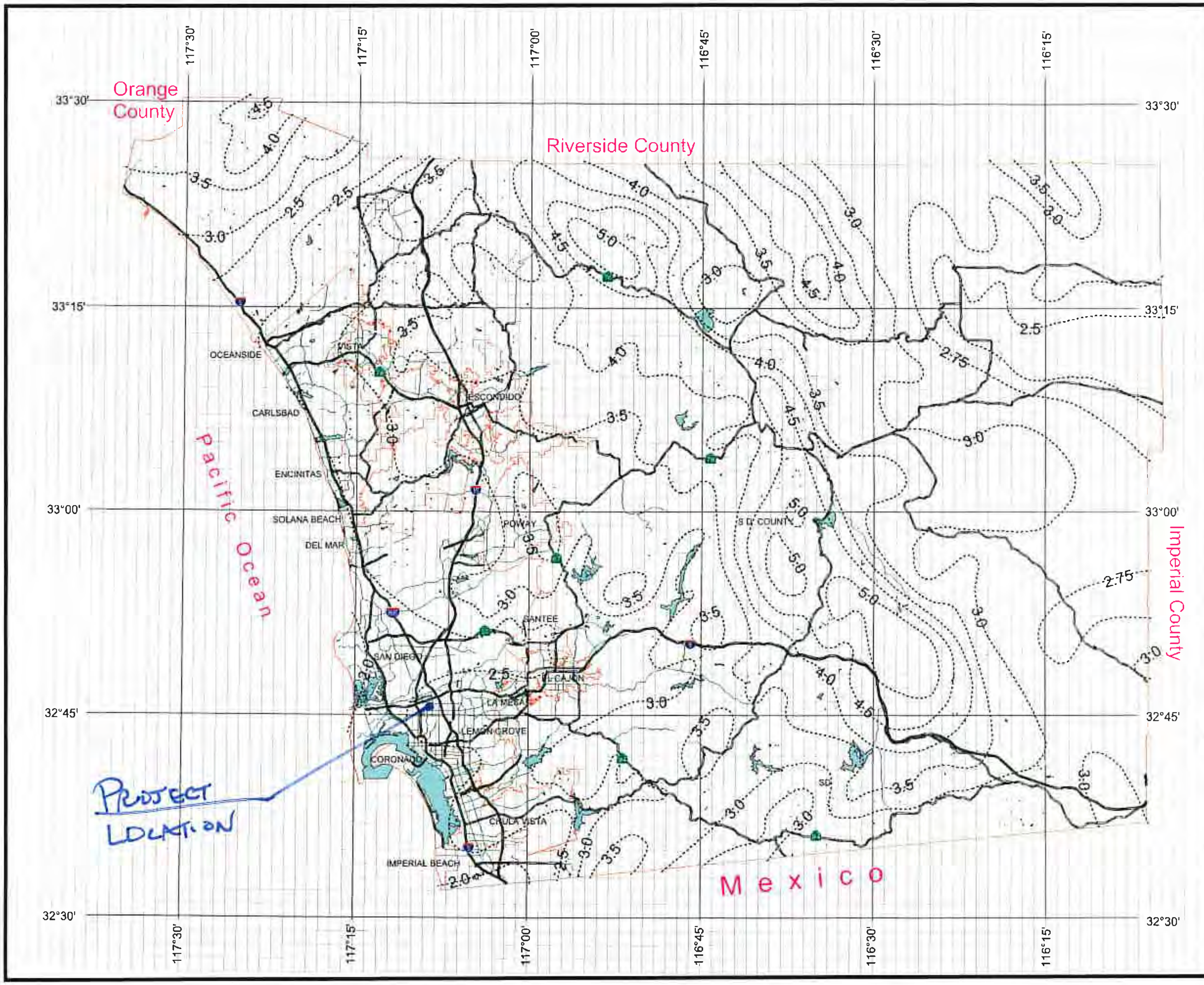
Rainfall Isopluvials

100 Year Rainfall Event - 24 Hours



THD - SRC

$P_{24} = 4.6 IN$



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours

Isopluvial (inches)

THD-SRC

$P_6 = 2.6$ IN

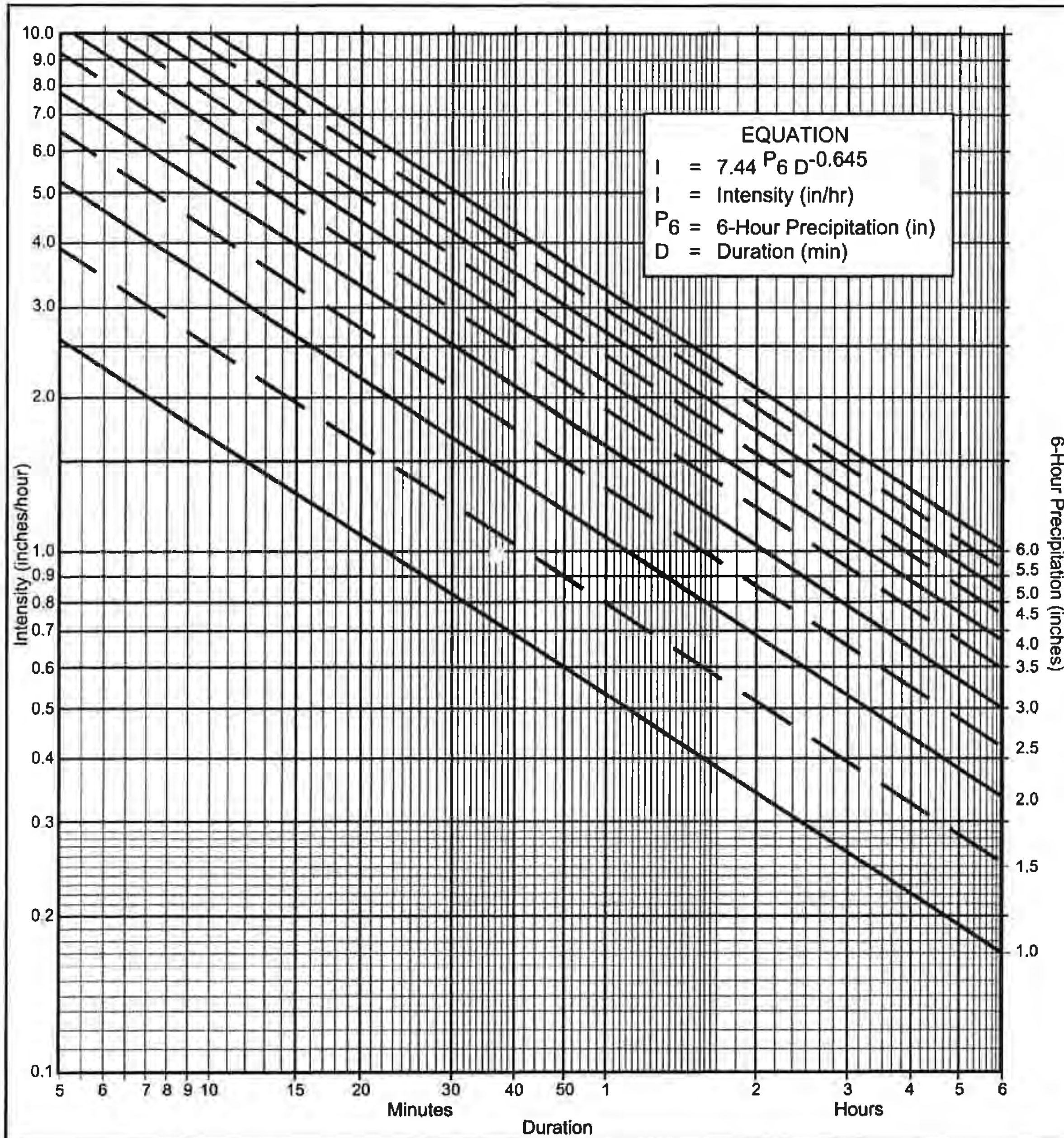
DPW GIS

SanGIS

We Have San Diego Covered!

3 0 3 Miles

THD / SRC



Directions for Application:

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

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

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

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

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

Application Form:

(a) Selected frequency 100 year

(b) P_6 = 2.6 in., P_{24} = 4.6 , $\frac{P_6}{P_{24}}$ = 56.5 %(2)

(c) Adjusted $P_6^{(2)}$ = 2.6 in.

(d) t_x = ______ min.

(e) I = ______ in./hr.

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

P6 / Duration	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10 11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5 13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2 92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

Intensity-Duration Design Chart - Template

FIGURE

3-1

Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS

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

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

Note that the Initial Time of Concentration should be reflective of the general land-use at the upstream end of a drainage basin. A single lot with an area of two or less acres does not have a significant effect where the drainage basin area is 20 to 600 acres.

Table 3-2 provides limits of the length (Maximum Length (L_M)) of sheet flow to be used in hydrology studies. Initial T_i values based on average C values for the Land Use Element are also included. These values can be used in planning and design applications as described below. Exceptions may be approved by the "Regulating Agency" when submitted with a detailed study.

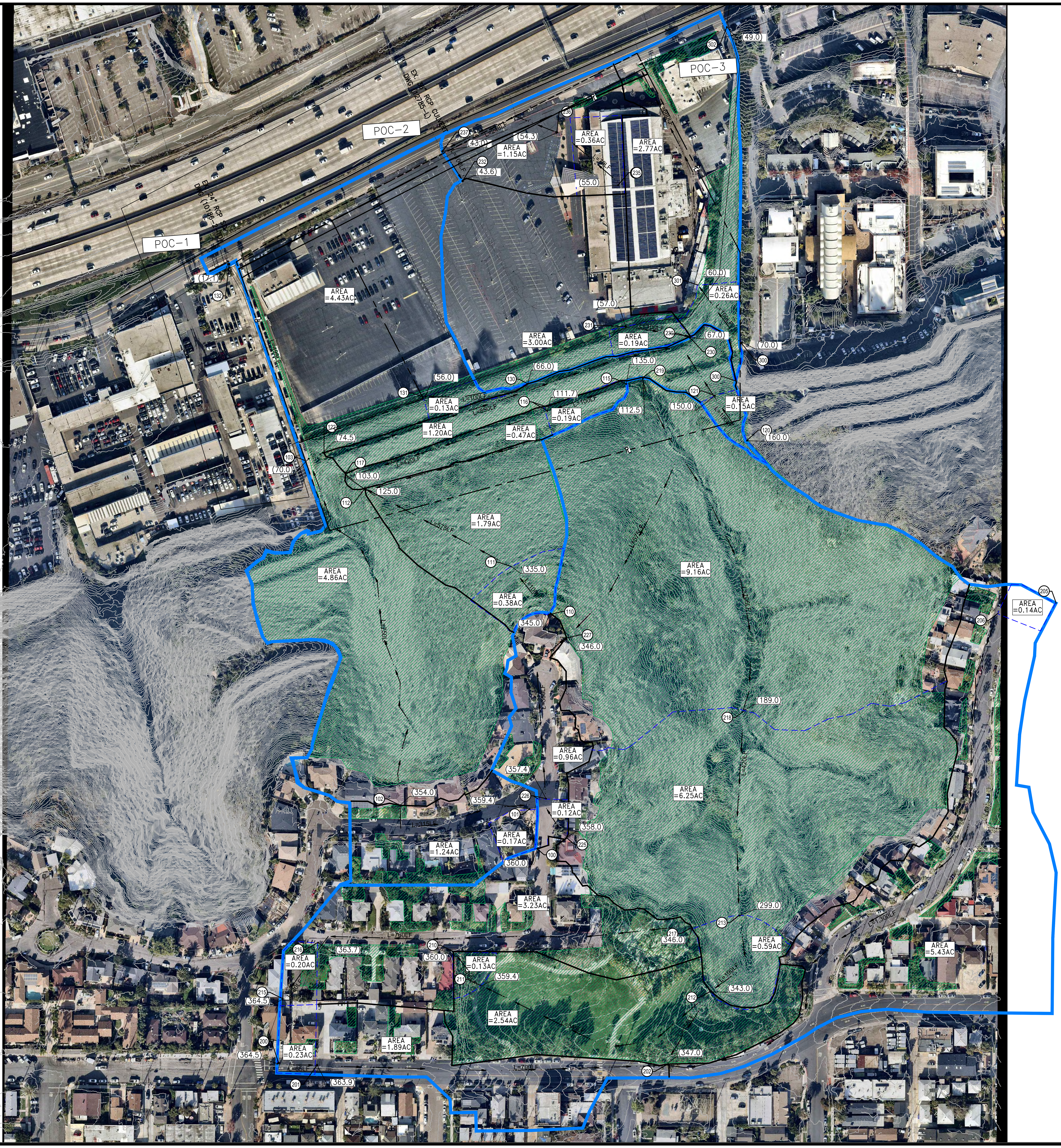
Table 3-2

MAXIMUM OVERLAND FLOW LENGTH (L_M)
& INITIAL TIME OF CONCENTRATION (T_i)

Element*	DU/ Acre	.5%		1%		2%		3%		5%		10%	
		L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i	L_M	T_i
Natural		50	13.2	70	12.5	85	10.9	100	10.3	100	8.7	100	6.9
LDR	1	50	12.2	70	11.5	85	10.0	100	9.5	100	8.0	100	6.4
LDR	2	50	11.3	70	10.5	85	9.2	100	8.8	100	7.4	100	5.8
LDR	2.9	50	10.7	70	10.0	85	8.8	95	8.1	100	7.0	100	5.6
MDR	4.3	50	10.2	70	9.6	80	8.1	95	7.8	100	6.7	100	5.3
MDR	7.3	50	9.2	65	8.4	80	7.4	95	7.0	100	6.0	100	4.8
MDR	10.9	50	8.7	65	7.9	80	6.9	90	6.4	100	5.7	100	4.5
MDR	14.5	50	8.2	65	7.4	80	6.5	90	6.0	100	5.4	100	4.3
HDR	24	50	6.7	65	6.1	75	5.1	90	4.9	95	4.3	100	3.5
HDR	43	50	5.3	65	4.7	75	4.0	85	3.8	95	3.4	100	2.7
N. Com		50	5.3	60	4.5	75	4.0	85	3.8	95	3.4	100	2.7
G. Com		50	4.7	60	4.1	75	3.6	85	3.4	90	2.9	100	2.4
O.P./Com		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
Limited I.		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
General I.		50	3.7	60	3.2	70	2.7	80	2.6	90	2.3	100	1.9

*See Table 3-1 for more detailed description

Appendix A
EXISTING AND PROPOSED HYDROLOGY MAPS



LEGEND

PROPERTY LINE	
EXISTING STORM DRAIN	SD
BASIN LIMITS	
SUB-BASIN LIMITS	
INITIAL AREA LIMITS	
FLOW PATH	
DIRECTION OF FLOW	
POINT OF CONFLUENCE DESIGNATION	POC-#
HYDROLOGY NODE	100
EXISTING CONTOUR	20
EXISTING PERVIOUS AREA	
BASIN AREA	AREA =#####

EXISTING HYDROLOGY SUMMARY

POC	DESCRIPTION	AREA (AC)	Tc (MIN)	Q100 (CFS)
1	EX. 24" RCP SD	15.0	9.94	37.20
2	EX. 36" RCP CULVERT	35.6	15.32	71.31
3	CAMINO DEL RIO S	3.0	10.16	10.15

100' 0' 50' 100'

SCALE: 1" = 100'

THE HOME DEPOT
MISSION VALLEY

EXISTING CONDITIONS
HYDROLOGY MAP

FUSCOE
ENGINEERING

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

JOB NO. 128-020

DRAWN BY: B.C.

SHEET 1 OF 1

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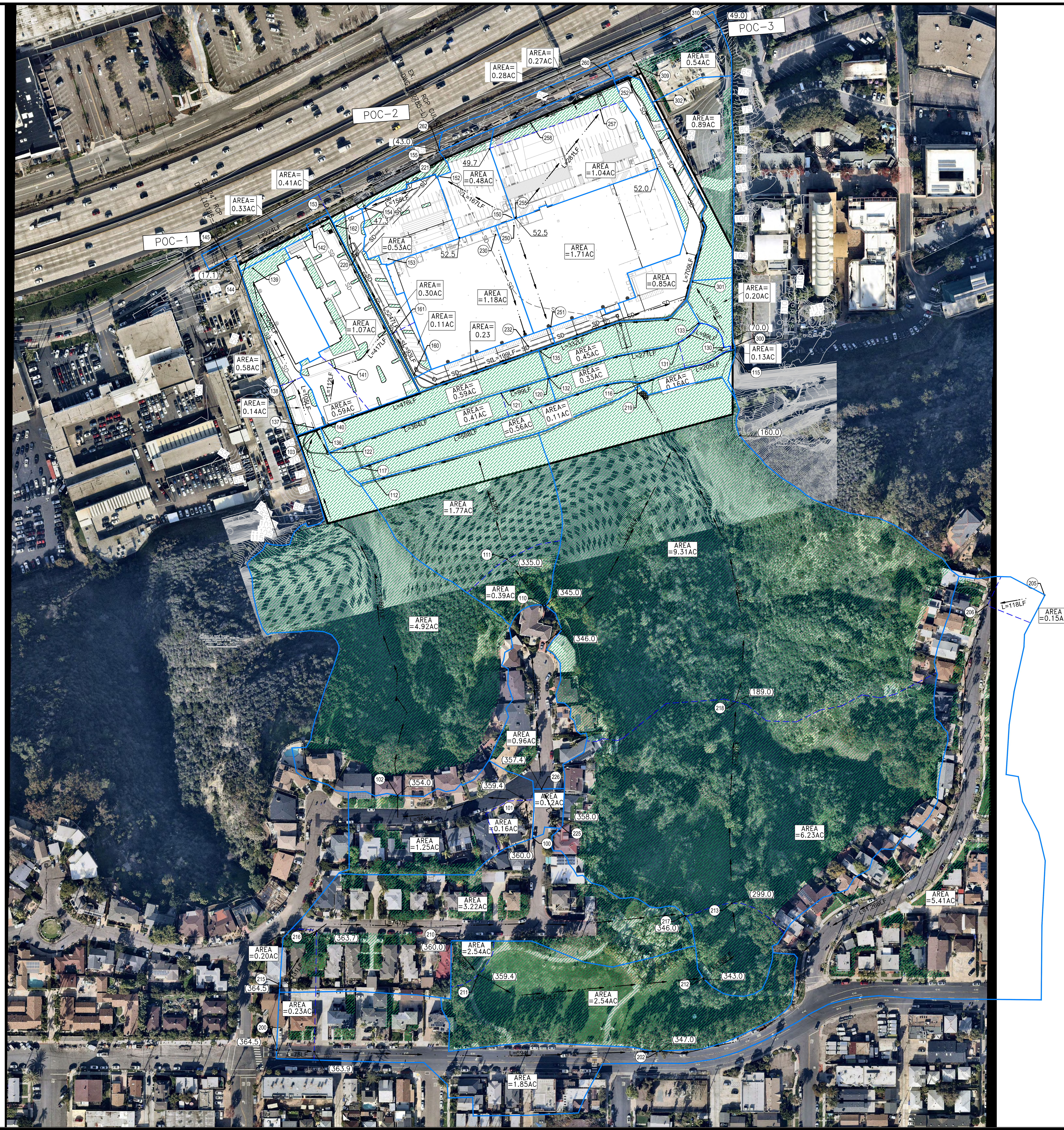

LEGEND

PROPERTY LINE	
EXISTING STORM DRAIN	SD
BASIN LIMITS	
SUB-BASIN LIMITS	
INITIAL AREA LIMITS	
FLOW PATH	
DIRECTION OF FLOW	
POINT OF CONFLUENCE DESIGNATION	POC-#
HYDROLOGY NODE	100
EXISTING CONTOUR	20
EXISTING PERVIOUS AREA	
BASIN AREA	AREA =#####

TOTAL AREA = 2,332,167 SF
IMPERVIOUS AREA = 1,346,227 SF
PERVIOUS AREA = 986,632 SF

PROPOSED HYDROLOGY SUMMARY

POC	DESCRIPTION	AREA (AC)	Tc (MIN)	Q100 (CFS)
1	EX. 24" RCP SD	15.8	9.90	36.85
2	EX. 36" RCP CULVERT	36.6	16.37	70.37
3	CAMINO DEL RIO S	1.6	7.31	4.94

NOTE:

HYDROLOGY CALCULATIONS PERFORMED PER CITY OF SAN DIEGO DRAINAGE DESIGN MANUAL (2017) APPENDIX A, RATIONAL METHOD (Q=CIA).

SCALE: 1" = 100'

THE HOME DEPOT MISSION VALLEY

PROPOSED CONDITIONS HYDROLOGY MAP

FUSCOE ENGINEERING
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SHEET 1 OF 1

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Appendix B
RUNOFF COEFFICIENT CALCULATIONS

FUSCOE ENGINEERING
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Job Name: SCOTTISH RITE & THE HOME DEPOT - MISSION VALLEY
Job #: 128-020
Date: 5/22/2020

Runoff Coefficient Calculations

Runoff Coefficent Variables Per City of San Diego Drainage Design Manual (January 2017)

Assumptions:

TOTAL AREA	2,332,859
TOTAL PERVIOUS	1,346,227
TOTAL IMPERVIOUS	986,632

PROPOSED CONDITIONS:

SCOTTISH RITE AND HOME DEPOT SITE =	**424,345**	**sf**
Area Pervious =	47,898	sf
Area Impervious =	376,447	sf
Actual % Impervious =	89	
Given C Factor per Table 3-1 =	0.84	
EXISTING SLOPE AND PROPOSED TERRACE DRAINS	**1,221,553**	**sf**
Area Pervious =	1,221,553	sf
Area Impervious =	0	sf
Actual % Impervious =	0	
Given C Factor per Table 3-1 =	0.35	
EXISTING COMMERCIAL AREA	**53,077**	**sf**
Area Pervious =	13,223	sf
Area Impervious =	39,854	sf
Actual % Impervious =	75	
Given C Factor per Table 3-1 =	0.76	
EXISTING RESIDENTAIL AREA	**546,250**	**sf**
Area Pervious =	63,553	sf
Area Impervious =	482,697	sf

See existing hydrology for C Factors

*See Note (2) on Table A-1 of the SDDDM included in Appendix 3 for Calculated 'C' equation

Appendix C
RATIONAL METHOD CALCULATIONS (AES)
EXISTING/PROPOSED

EXISTING AES CALCULATIONS

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

             RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
             Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                          2003,1985,1981 HYDROLOGY MANUAL
          (c) Copyright 1982-2016 Advanced Engineering Software (aes)
              Ver. 23.0 Release Date: 07/01/2016  License ID 1355

                            Analysis prepared by:

                              Fuscoe Engineering
                           6390 Greenich Dr Ste 170
                                San Diego, CA
                                    92122

 ************************** DESCRIPTION OF STUDY **************************
* THE HOME DEPOT - MISSION VALLEY - PRE-DEVELOPMENT STUDY                  *
* SERIES 1                                                                 *
* SAN DIEGO, CALIFORNIA                                                    *
 **************************************************************************

  FILE NAME: SR100EX.DAT
  TIME/DATE OF STUDY: 09:52 04/10/2020
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.600
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  12.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
  *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
  2   15.0     10.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0160

  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    100.00 TO NODE    101.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .6800
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    65.00
  UPSTREAM ELEVATION(FEET) =    360.00
```

```
  DOWNSTREAM ELEVATION(FEET) =    359.35
  ELEVATION DIFFERENCE(FEET) =      0.65
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    5.856
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    60.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.187
  SUBAREA RUNOFF(CFS) =      0.71
  TOTAL AREA(ACRES) =      0.17   TOTAL RUNOFF(CFS) =      0.71

****************************************************************************
  FLOW PROCESS FROM NODE    101.00 TO NODE    102.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  359.35  DOWNSTREAM ELEVATION(FEET) =  354.00
  STREET LENGTH(FEET) =   315.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       3.02
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.27
    HALFSTREET FLOOD WIDTH(FEET) =    7.23
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.35
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.64
  STREET FLOW TRAVEL TIME(MIN.) =   2.23   Tc(MIN.) =    8.09
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.024
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7400
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.733
  SUBAREA AREA(ACRES) =    1.24      SUBAREA RUNOFF(CFS) =    4.61
  TOTAL AREA(ACRES) =        1.4        PEAK FLOW RATE(CFS) =       5.18

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.31   HALFSTREET FLOOD WIDTH(FEET) =   9.34
  FLOW VELOCITY(FEET/SEC.) =  2.62   DEPTH*VELOCITY(FT*FT/SEC.) =   0.82
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    102.00 =     380.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    102.00 TO NODE    103.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    354.00  DOWNSTREAM(FEET) =     70.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =   750.00   CHANNEL SLOPE =  0.3787
  CHANNEL BASE(FEET) =    4.00   "Z" FACTOR =   2.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   4.00
```

```
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.587
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .4100
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       9.80
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =  10.21
  AVERAGE FLOW DEPTH(FEET) =   0.22   TRAVEL TIME(MIN.) =   1.22
  Tc(MIN.) =    9.31
  SUBAREA AREA(ACRES) =     4.86       SUBAREA RUNOFF(CFS) =    9.14
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.483
  TOTAL AREA(ACRES) =        6.3         PEAK FLOW RATE(CFS) =      13.87

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.26   FLOW VELOCITY(FEET/SEC.) =  11.60
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    103.00 =    1130.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    103.00 TO NODE    103.00 IS CODE =  10
----------------------------------------------------------------------------
  >>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 1 <<<<<
============================================================================

****************************************************************************
  FLOW PROCESS FROM NODE    110.00 TO NODE    111.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3600
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    345.00
  DOWNSTREAM ELEVATION(FEET) =    335.00
  ELEVATION DIFFERENCE(FEET) =     10.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.183
  WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.973
  SUBAREA RUNOFF(CFS) =      0.82
  TOTAL AREA(ACRES) =      0.38   TOTAL RUNOFF(CFS) =      0.82

****************************************************************************
  FLOW PROCESS FROM NODE    111.00 TO NODE    112.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    335.00  DOWNSTREAM(FEET) =    125.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =   575.00   CHANNEL SLOPE =  0.3652
  CHANNEL BASE(FEET) =    4.00   "Z" FACTOR =   2.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   4.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.156
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3520
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       2.44
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   6.05
  AVERAGE FLOW DEPTH(FEET) =   0.10   TRAVEL TIME(MIN.) =   1.59
  Tc(MIN.) =    7.77
  SUBAREA AREA(ACRES) =     1.78       SUBAREA RUNOFF(CFS) =    3.22
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.353
```

```
  TOTAL AREA(ACRES) =        2.2         PEAK FLOW RATE(CFS) =      3.93

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.13   FLOW VELOCITY(FEET/SEC.) =   7.28
  LONGEST FLOWPATH FROM NODE    110.00 TO NODE    112.00 =     675.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    112.00 TO NODE    117.00 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.) =    7.77
  RAINFALL INTENSITY(INCH/HR) =   5.16
  TOTAL STREAM AREA(ACRES) =     2.16
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      3.93

****************************************************************************
  FLOW PROCESS FROM NODE    115.00 TO NODE    116.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    135.00
  DOWNSTREAM ELEVATION(FEET) =    125.00
  ELEVATION DIFFERENCE(FEET) =     10.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.267
  WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.922
  SUBAREA RUNOFF(CFS) =      0.39
  TOTAL AREA(ACRES) =      0.19   TOTAL RUNOFF(CFS) =      0.39

****************************************************************************
  FLOW PROCESS FROM NODE    116.00 TO NODE    117.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   111.70  DOWNSTREAM(FEET) =   103.00
  FLOW LENGTH(FEET) =   420.00   MANNING'S N =  0.011
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000
  DEPTH OF FLOW IN  12.0 INCH PIPE IS   2.1 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   4.15
  ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       0.39
  PIPE TRAVEL TIME(MIN.) =   1.69    Tc(MIN.) =    7.95
  LONGEST FLOWPATH FROM NODE    115.00 TO NODE    117.00 =     520.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    117.00 TO NODE    117.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.078
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
```

```
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.3500
  SUBAREA AREA(ACRES) =    0.47   SUBAREA RUNOFF(CFS) =    0.84
  TOTAL AREA(ACRES) =        0.7   TOTAL RUNOFF(CFS) =       1.17
  TC(MIN.) =    7.95

****************************************************************************
  FLOW PROCESS FROM NODE    117.00 TO NODE    122.00 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.) =    7.95
  RAINFALL INTENSITY(INCH/HR) =   5.08
  TOTAL STREAM AREA(ACRES) =     0.66
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      1.17

****************************************************************************
  FLOW PROCESS FROM NODE    120.00 TO NODE    121.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =    160.00
  DOWNSTREAM ELEVATION(FEET) =    150.00
  ELEVATION DIFFERENCE(FEET) =     10.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.267
  WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.922
  SUBAREA RUNOFF(CFS) =      0.29
  TOTAL AREA(ACRES) =      0.14   TOTAL RUNOFF(CFS) =      0.29

****************************************************************************
  FLOW PROCESS FROM NODE    121.00 TO NODE    122.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   150.00  DOWNSTREAM(FEET) =    75.50
  FLOW LENGTH(FEET) =   850.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000
  DEPTH OF FLOW IN  12.0 INCH PIPE IS   1.4 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   5.63
  ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       0.29
  PIPE TRAVEL TIME(MIN.) =   2.52    Tc(MIN.) =    8.78
  LONGEST FLOWPATH FROM NODE    120.00 TO NODE    122.00 =     950.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    121.00 TO NODE    122.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.763
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
```

```
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.3500
  SUBAREA AREA(ACRES) =    1.18   SUBAREA RUNOFF(CFS) =    1.97
  TOTAL AREA(ACRES) =        1.3   TOTAL RUNOFF(CFS) =       2.20
  TC(MIN.) =    8.78

****************************************************************************
  FLOW PROCESS FROM NODE    122.00 TO NODE    122.00 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.) =    8.78
  RAINFALL INTENSITY(INCH/HR) =   4.76
  TOTAL STREAM AREA(ACRES) =     1.32
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      2.20

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1        3.93     7.77        5.156          2.16
      2        1.17     7.95        5.078          0.66
      3        2.20     8.78        4.763          1.32

  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        7.02     7.77       5.156
      2        7.03     7.95       5.078
      3        6.93     8.78       4.763

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =       7.03   Tc(MIN.) =    7.95
  TOTAL AREA(ACRES) =        4.1
  LONGEST FLOWPATH FROM NODE    120.00 TO NODE    122.00 =     950.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    122.00 TO NODE    103.00 IS CODE =  11
----------------------------------------------------------------------------
  >>>>>CONFLUENCE MEMORY BANK # 1 WITH THE MAIN-STREAM MEMORY<<<<<
============================================================================

  ** MAIN STREAM CONFLUENCE DATA **
  STREAM     RUNOFF      Tc      INTENSITY     AREA
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)   (ACRE)
      1        7.03     7.95       5.078        4.14
  LONGEST FLOWPATH FROM NODE    120.00 TO NODE    103.00 =     950.00 FEET.

  ** MEMORY BANK #  1 CONFLUENCE DATA **
  STREAM     RUNOFF      Tc      INTENSITY     AREA
  NUMBER      (CFS)    (MIN.)   (INCH/HOUR)   (ACRE)
      1       13.87     9.31       4.587        6.27
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    103.00 =    1130.00 FEET.

  ** PEAK FLOW RATE TABLE **
```

```
  STREAM    RUNOFF       Tc      INTENSITY
  NUMBER     (CFS)     (MIN.)   (INCH/HOUR)
      1      18.89       7.95        5.078
      2      20.23       9.31        4.587

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      20.23   Tc(MIN.) =    9.31
  TOTAL AREA(ACRES) =       10.4

****************************************************************************
  FLOW PROCESS FROM NODE    103.00 TO NODE    103.00 IS CODE =  12
----------------------------------------------------------------------------
  >>>>>CLEAR MEMORY BANK # 1 <<<<<
============================================================================

****************************************************************************
  FLOW PROCESS FROM NODE    103.00 TO NODE    132.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    67.00  DOWNSTREAM(FEET) =    38.50
  FLOW LENGTH(FEET) =   495.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  18.0 INCH PIPE IS  12.7 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  15.18
  ESTIMATED PIPE DIAMETER(INCH) =  18.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      20.23
  PIPE TRAVEL TIME(MIN.) =   0.54    Tc(MIN.) =    9.85
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    132.00 =    1625.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    132.00 TO NODE    132.00 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.) =    9.85
  RAINFALL INTENSITY(INCH/HR) =   4.42
  TOTAL STREAM AREA(ACRES) =    10.40
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     20.23

****************************************************************************
  FLOW PROCESS FROM NODE    130.00 TO NODE    131.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =     66.00
  DOWNSTREAM ELEVATION(FEET) =     56.00
  ELEVATION DIFFERENCE(FEET) =     10.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.267
  WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.922
  SUBAREA RUNOFF(CFS) =      0.29
  TOTAL AREA(ACRES) =      0.14   TOTAL RUNOFF(CFS) =      0.29
```

```
****************************************************************************
  FLOW PROCESS FROM NODE    131.00 TO NODE    132.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =   56.00  DOWNSTREAM ELEVATION(FEET) =   41.50
  STREET LENGTH(FEET) =   705.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       8.46
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.35
    HALFSTREET FLOOD WIDTH(FEET) =   10.98
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.20
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.11
  STREET FLOW TRAVEL TIME(MIN.) =   3.67   Tc(MIN.) =    9.94
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.397
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8700
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.854
  SUBAREA AREA(ACRES) =    4.41      SUBAREA RUNOFF(CFS) =   16.87
  TOTAL AREA(ACRES) =        4.5        PEAK FLOW RATE(CFS) =      17.09

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.42   HALFSTREET FLOOD WIDTH(FEET) =  14.65
  FLOW VELOCITY(FEET/SEC.) =  3.77   DEPTH*VELOCITY(FT*FT/SEC.) =   1.58
  LONGEST FLOWPATH FROM NODE    130.00 TO NODE    132.00 =     805.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    132.00 TO NODE    132.00 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.) =    9.94
  RAINFALL INTENSITY(INCH/HR) =   4.40
  TOTAL STREAM AREA(ACRES) =     4.55
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     17.09

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       20.23     9.85        4.423        10.40
      2       17.09     9.94        4.397         4.55

  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       37.16     9.85       4.423
      2       37.20     9.94       4.397

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      37.20   Tc(MIN.) =    9.94
  TOTAL AREA(ACRES) =       15.0
  LONGEST FLOWPATH FROM NODE    100.00 TO NODE    132.00 =    1625.00 FEET.
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =       15.0  TC(MIN.) =      9.94
  PEAK FLOW RATE(CFS)   =      37.20
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

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

            RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                         2003,1985,1981 HYDROLOGY MANUAL
         (c) Copyright 1982-2016 Advanced Engineering Software (aes)
             Ver. 23.0 Release Date: 07/01/2016  License ID 1355

                           Analysis prepared by:

                             Fuscoe Engineering
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                               San Diego, CA
                                   92122

 ************************** DESCRIPTION OF STUDY **************************
* THE HOME DEPOT - MISSION VALLEY - PRE-DEVELOPMENT STUDY                   *
* SERIES 2                                                                  *
* SAN DIEGO, CALIFORNIA                                                     *
 **************************************************************************

  FILE NAME: SR200EX.DAT
  TIME/DATE OF STUDY: 09:33 04/10/2020
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.600
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  12.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
  *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
  2   15.0     10.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0160

  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    200.00 TO NODE    201.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8500
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    65.00
  UPSTREAM ELEVATION(FEET) =    364.50
```

```
  DOWNSTREAM ELEVATION(FEET) =    363.85
  ELEVATION DIFFERENCE(FEET) =      0.65
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.628
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    65.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.850
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      1.34
  TOTAL AREA(ACRES) =      0.23   TOTAL RUNOFF(CFS) =      1.34

****************************************************************************
  FLOW PROCESS FROM NODE    201.00 TO NODE    202.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  363.85  DOWNSTREAM ELEVATION(FEET) =  347.00
  STREET LENGTH(FEET) =   700.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       5.74
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.31
    HALFSTREET FLOOD WIDTH(FEET) =    9.02
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.08
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.94
  STREET FLOW TRAVEL TIME(MIN.) =   3.79   Tc(MIN.) =    7.42
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.311
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8700
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.868
  SUBAREA AREA(ACRES) =    1.88      SUBAREA RUNOFF(CFS) =    8.69
  TOTAL AREA(ACRES) =        2.1        PEAK FLOW RATE(CFS) =       9.72

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.35   HALFSTREET FLOOD WIDTH(FEET) =  11.29
  FLOW VELOCITY(FEET/SEC.) =  3.49   DEPTH*VELOCITY(FT*FT/SEC.) =   1.23
  LONGEST FLOWPATH FROM NODE    200.00 TO NODE    202.00 =     765.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    202.00 TO NODE    202.00 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.) =    7.42
  RAINFALL INTENSITY(INCH/HR) =   5.31
```

```
  TOTAL STREAM AREA(ACRES) =     2.11
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      9.72

****************************************************************************
  FLOW PROCESS FROM NODE    205.00 TO NODE    206.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7100
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    65.00
  UPSTREAM ELEVATION(FEET) =    370.00
  DOWNSTREAM ELEVATION(FEET) =    369.35
  ELEVATION DIFFERENCE(FEET) =      0.65
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    5.660
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    65.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.324
  SUBAREA RUNOFF(CFS) =      0.67
  TOTAL AREA(ACRES) =      0.15   TOTAL RUNOFF(CFS) =      0.67

****************************************************************************
  FLOW PROCESS FROM NODE    206.00 TO NODE    202.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  369.35  DOWNSTREAM ELEVATION(FEET) =  347.00
  STREET LENGTH(FEET) =  1300.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       8.94
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.36
    HALFSTREET FLOOD WIDTH(FEET) =   11.68
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.02
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.08
  STREET FLOW TRAVEL TIME(MIN.) =   7.19   Tc(MIN.) =   12.85
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.727
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7900
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.788
  SUBAREA AREA(ACRES) =    5.44      SUBAREA RUNOFF(CFS) =   16.02
  TOTAL AREA(ACRES) =        5.6        PEAK FLOW RATE(CFS) =      16.42

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.43   HALFSTREET FLOOD WIDTH(FEET) =  14.96
```

```
  FLOW VELOCITY(FEET/SEC.) =  3.48   DEPTH*VELOCITY(FT*FT/SEC.) =   1.48
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    202.00 =    1365.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    202.00 TO NODE    202.00 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.) =   12.85
  RAINFALL INTENSITY(INCH/HR) =   3.73
  TOTAL STREAM AREA(ACRES) =     5.59
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     16.42

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1        9.72     7.42        5.311          2.11
      2       16.42    12.85        3.727          5.59

  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       19.21     7.42       5.311
      2       23.24    12.85       3.727

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      23.24   Tc(MIN.) =   12.85
  TOTAL AREA(ACRES) =        7.7
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    202.00 =    1365.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    202.00 TO NODE    212.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   344.00  DOWNSTREAM(FEET) =   340.00
  FLOW LENGTH(FEET) =   155.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  21.0 INCH PIPE IS  16.5 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  11.44
  ESTIMATED PIPE DIAMETER(INCH) =  21.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      23.24
  PIPE TRAVEL TIME(MIN.) =   0.23    Tc(MIN.) =   13.07
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    212.00 =    1520.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    212.00 TO NODE    212.00 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.) =   13.07
  RAINFALL INTENSITY(INCH/HR) =   3.69
```

```
  TOTAL STREAM AREA(ACRES) =     7.70
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     23.24

****************************************************************************
  FLOW PROCESS FROM NODE    210.00 TO NODE    211.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .4300
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    65.00
  UPSTREAM ELEVATION(FEET) =    360.00
  DOWNSTREAM ELEVATION(FEET) =    359.35
  ELEVATION DIFFERENCE(FEET) =      0.65
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    9.723
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    65.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.461
  SUBAREA RUNOFF(CFS) =      0.25
  TOTAL AREA(ACRES) =      0.13   TOTAL RUNOFF(CFS) =      0.25

****************************************************************************
  FLOW PROCESS FROM NODE    211.00 TO NODE    212.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  359.35  DOWNSTREAM ELEVATION(FEET) =  343.00
  STREET LENGTH(FEET) =   545.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       2.28
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.23
    HALFSTREET FLOOD WIDTH(FEET) =    5.43
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.76
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.65
  STREET FLOW TRAVEL TIME(MIN.) =   3.29   Tc(MIN.) =   13.01
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.697
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .4300
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.430
  SUBAREA AREA(ACRES) =    2.54      SUBAREA RUNOFF(CFS) =    4.04
  TOTAL AREA(ACRES) =        2.7        PEAK FLOW RATE(CFS) =       4.24

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.28   HALFSTREET FLOOD WIDTH(FEET) =   7.46
```

```
  FLOW VELOCITY(FEET/SEC.) =  3.14   DEPTH*VELOCITY(FT*FT/SEC.) =   0.87
  LONGEST FLOWPATH FROM NODE    210.00 TO NODE    212.00 =     610.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    212.00 TO NODE    212.00 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.) =   13.01
  RAINFALL INTENSITY(INCH/HR) =   3.70
  TOTAL STREAM AREA(ACRES) =     2.67
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      4.24

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       23.24    13.07        3.686          7.70
      2        4.24    13.01        3.697          2.67

  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       27.41    13.01       3.697
      2       27.47    13.07       3.686

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      27.47   Tc(MIN.) =   13.07
  TOTAL AREA(ACRES) =       10.4
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    212.00 =    1520.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    212.00 TO NODE    213.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   340.00  DOWNSTREAM(FEET) =   299.00
  FLOW LENGTH(FEET) =   190.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  15.0 INCH PIPE IS  11.8 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  26.42
  ESTIMATED PIPE DIAMETER(INCH) =  15.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      27.47
  PIPE TRAVEL TIME(MIN.) =   0.12    Tc(MIN.) =   13.19
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    213.00 =    1710.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    213.00 TO NODE    213.00 IS CODE =  81
----------------------------------------------------------------------------
  >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.664
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3500
  S.C.S. CURVE NUMBER (AMC II) =   0
```

```
  AREA-AVERAGE RUNOFF COEFFICIENT = 0.6925
  SUBAREA AREA(ACRES) =    0.59   SUBAREA RUNOFF(CFS) =    0.76
  TOTAL AREA(ACRES) =       11.0   TOTAL RUNOFF(CFS) =      27.81
  TC(MIN.) =   13.19

****************************************************************************
  FLOW PROCESS FROM NODE    213.00 TO NODE    213.00 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.) =   13.19
  RAINFALL INTENSITY(INCH/HR) =   3.66
  TOTAL STREAM AREA(ACRES) =    10.96
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     27.81

****************************************************************************
  FLOW PROCESS FROM NODE    215.00 TO NODE    216.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7100
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    80.00
  UPSTREAM ELEVATION(FEET) =    364.50
  DOWNSTREAM ELEVATION(FEET) =    363.70
  ELEVATION DIFFERENCE(FEET) =      0.80
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    5.660
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    65.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.324
  SUBAREA RUNOFF(CFS) =      0.85
  TOTAL AREA(ACRES) =      0.19   TOTAL RUNOFF(CFS) =      0.85

****************************************************************************
  FLOW PROCESS FROM NODE    216.00 TO NODE    217.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  363.70  DOWNSTREAM ELEVATION(FEET) =  346.00
  STREET LENGTH(FEET) =   760.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       6.40
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.32
```

```
    HALFSTREET FLOOD WIDTH(FEET) =    9.49
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.14
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.99
  STREET FLOW TRAVEL TIME(MIN.) =   4.04   Tc(MIN.) =    9.70
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.469
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7600
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.757
  SUBAREA AREA(ACRES) =    3.22      SUBAREA RUNOFF(CFS) =   10.94
  TOTAL AREA(ACRES) =        3.4        PEAK FLOW RATE(CFS) =      11.54

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.37   HALFSTREET FLOOD WIDTH(FEET) =  12.23
  FLOW VELOCITY(FEET/SEC.) =  3.58   DEPTH*VELOCITY(FT*FT/SEC.) =   1.33
  LONGEST FLOWPATH FROM NODE    215.00 TO NODE    217.00 =     840.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    217.00 TO NODE    213.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   346.00  DOWNSTREAM(FEET) =   299.00
  FLOW LENGTH(FEET) =   115.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  12.0 INCH PIPE IS   6.2 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  27.96
  ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      11.54
  PIPE TRAVEL TIME(MIN.) =   0.07    Tc(MIN.) =    9.76
  LONGEST FLOWPATH FROM NODE    215.00 TO NODE    213.00 =     955.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    213.00 TO NODE    213.00 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.) =    9.76
  RAINFALL INTENSITY(INCH/HR) =   4.45
  TOTAL STREAM AREA(ACRES) =     3.41
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     11.54

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       27.81    13.19        3.664         10.96
      2       11.54     9.76        4.449          3.41

  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       34.44     9.76       4.449
      2       37.31    13.19       3.664
```

```
  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      37.31   Tc(MIN.) =   13.19
  TOTAL AREA(ACRES) =       14.4
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    213.00 =    1710.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    213.00 TO NODE    218.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    299.00  DOWNSTREAM(FEET) =    189.00
  CHANNEL LENGTH THRU SUBAREA(FEET) =   420.00   CHANNEL SLOPE =  0.2619
  CHANNEL BASE(FEET) =    4.00   "Z" FACTOR =   2.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   4.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.580
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .4000
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =      41.79
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =  14.58
  AVERAGE FLOW DEPTH(FEET) =   0.56   TRAVEL TIME(MIN.) =   0.48
  Tc(MIN.) =   13.67
  SUBAREA AREA(ACRES) =     6.25       SUBAREA RUNOFF(CFS) =    8.95
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.615
  TOTAL AREA(ACRES) =       20.6         PEAK FLOW RATE(CFS) =      45.37

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.58   FLOW VELOCITY(FEET/SEC.) =  15.01
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    218.00 =    2130.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    218.00 TO NODE    219.00 IS CODE =  51
----------------------------------------------------------------------------
  >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
  >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    189.00  DOWNSTREAM(FEET) =    112.50
  CHANNEL LENGTH THRU SUBAREA(FEET) =   705.00   CHANNEL SLOPE =  0.1085
  CHANNEL BASE(FEET) =    4.00   "Z" FACTOR =   2.000
  MANNING'S FACTOR = 0.030   MAXIMUM DEPTH(FEET) =   4.00
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  3.418
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3800
  S.C.S. CURVE NUMBER (AMC II) =   0
  TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =      51.33
  TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =  11.49
  AVERAGE FLOW DEPTH(FEET) =   0.80   TRAVEL TIME(MIN.) =   1.02
  Tc(MIN.) =   14.69
  SUBAREA AREA(ACRES) =     9.16       SUBAREA RUNOFF(CFS) =   11.90
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.542
  TOTAL AREA(ACRES) =       29.8         PEAK FLOW RATE(CFS) =      55.21

  END OF SUBAREA CHANNEL FLOW HYDRAULICS:
  DEPTH(FEET) =  0.83   FLOW VELOCITY(FEET/SEC.) =  11.77
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    219.00 =    2835.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    219.00 TO NODE    219.00 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.) =   14.69
  RAINFALL INTENSITY(INCH/HR) =   3.42
  TOTAL STREAM AREA(ACRES) =    29.78
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     55.21

****************************************************************************
  FLOW PROCESS FROM NODE    225.00 TO NODE    226.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7100
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    65.00
  UPSTREAM ELEVATION(FEET) =    358.00
  DOWNSTREAM ELEVATION(FEET) =    357.40
  ELEVATION DIFFERENCE(FEET) =      0.60
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    5.709
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    62.69
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.289
  SUBAREA RUNOFF(CFS) =      0.49
  TOTAL AREA(ACRES) =      0.11   TOTAL RUNOFF(CFS) =      0.49

****************************************************************************
  FLOW PROCESS FROM NODE    226.00 TO NODE    227.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =  357.35  DOWNSTREAM ELEVATION(FEET) =  346.00
  STREET LENGTH(FEET) =   350.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       2.33
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.23
    HALFSTREET FLOOD WIDTH(FEET) =    5.35
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.89
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.67
  STREET FLOW TRAVEL TIME(MIN.) =   2.02   Tc(MIN.) =    7.73
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.172
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .7400
  S.C.S. CURVE NUMBER (AMC II) =   0
```

```
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.737
  SUBAREA AREA(ACRES) =    0.96      SUBAREA RUNOFF(CFS) =    3.67
  TOTAL AREA(ACRES) =        1.1        PEAK FLOW RATE(CFS) =       4.08

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.27   HALFSTREET FLOOD WIDTH(FEET) =   7.15
  FLOW VELOCITY(FEET/SEC.) =  3.24   DEPTH*VELOCITY(FT*FT/SEC.) =   0.87
  LONGEST FLOWPATH FROM NODE    225.00 TO NODE    227.00 =     415.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    227.00 TO NODE    219.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   346.00  DOWNSTREAM(FEET) =   112.50
  FLOW LENGTH(FEET) =   575.00   MANNING'S N =  0.013
  ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000
  DEPTH OF FLOW IN  12.0 INCH PIPE IS   3.5 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  21.10
  ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =       4.08
  PIPE TRAVEL TIME(MIN.) =   0.45    Tc(MIN.) =    8.18
  LONGEST FLOWPATH FROM NODE    225.00 TO NODE    219.00 =     990.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    219.00 TO NODE    219.00 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.) =    8.18
  RAINFALL INTENSITY(INCH/HR) =   4.98
  TOTAL STREAM AREA(ACRES) =     1.07
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      4.08

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       55.21    14.69        3.418         29.78
      2        4.08     8.18        4.985          1.07

  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       41.93     8.18       4.985
      2       58.00    14.69       3.418

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      58.00   Tc(MIN.) =   14.69
  TOTAL AREA(ACRES) =       30.8
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    219.00 =    2835.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    219.00 TO NODE    232.00 IS CODE =  31
```

```
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =   112.50  DOWNSTREAM(FEET) =    43.60
  FLOW LENGTH(FEET) =   650.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  24.0 INCH PIPE IS  16.7 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =  24.87
  ESTIMATED PIPE DIAMETER(INCH) =  24.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      58.00
  PIPE TRAVEL TIME(MIN.) =   0.44    Tc(MIN.) =   15.13
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    232.00 =    3485.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    232.00 TO NODE    232.00 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.) =   15.13
  RAINFALL INTENSITY(INCH/HR) =   3.35
  TOTAL STREAM AREA(ACRES) =    30.85
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     58.00

****************************************************************************
  FLOW PROCESS FROM NODE    230.00 TO NODE    231.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .4300
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =     67.00
  DOWNSTREAM ELEVATION(FEET) =     57.00
  ELEVATION DIFFERENCE(FEET) =     10.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    5.598
  WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.369
  SUBAREA RUNOFF(CFS) =      0.49
  TOTAL AREA(ACRES) =      0.18   TOTAL RUNOFF(CFS) =      0.49

****************************************************************************
  FLOW PROCESS FROM NODE    231.00 TO NODE    232.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =   60.00  DOWNSTREAM ELEVATION(FEET) =   46.00
  STREET LENGTH(FEET) =   430.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
```

```
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       7.19
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.31
    HALFSTREET FLOOD WIDTH(FEET) =    9.34
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.63
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.14
  STREET FLOW TRAVEL TIME(MIN.) =   1.97   Tc(MIN.) =    7.57
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  5.242
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8600
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.836
  SUBAREA AREA(ACRES) =    3.02      SUBAREA RUNOFF(CFS) =   13.61
  TOTAL AREA(ACRES) =        3.2        PEAK FLOW RATE(CFS) =      14.02

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.37   HALFSTREET FLOOD WIDTH(FEET) =  12.30
  FLOW VELOCITY(FEET/SEC.) =  4.30   DEPTH*VELOCITY(FT*FT/SEC.) =   1.60
  LONGEST FLOWPATH FROM NODE    230.00 TO NODE    232.00 =     530.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    232.00 TO NODE    232.00 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.) =    7.57
  RAINFALL INTENSITY(INCH/HR) =   5.24
  TOTAL STREAM AREA(ACRES) =     3.20
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     14.02

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       58.00    15.13        3.354         30.85
      2       14.02     7.57        5.242          3.20

  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       51.13     7.57       5.242
      2       66.97    15.13       3.354

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      66.97   Tc(MIN.) =   15.13
  TOTAL AREA(ACRES) =       34.0
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    232.00 =    3485.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    232.00 TO NODE    237.00 IS CODE =  31
----------------------------------------------------------------------------
  >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
```

```
============================================================================
  ELEVATION DATA: UPSTREAM(FEET) =    43.60  DOWNSTREAM(FEET) =    43.00
  FLOW LENGTH(FEET) =   100.00   MANNING'S N =  0.013
  DEPTH OF FLOW IN  42.0 INCH PIPE IS  31.3 INCHES
  PIPE-FLOW VELOCITY(FEET/SEC.) =   8.70
  ESTIMATED PIPE DIAMETER(INCH) =  42.00    NUMBER OF PIPES =   1
  PIPE-FLOW(CFS) =      66.97
  PIPE TRAVEL TIME(MIN.) =   0.19    Tc(MIN.) =   15.32
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    237.00 =    3585.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    237.00 TO NODE    237.00 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.) =   15.32
  RAINFALL INTENSITY(INCH/HR) =   3.33
  TOTAL STREAM AREA(ACRES) =    34.05
  PEAK FLOW RATE(CFS) AT CONFLUENCE =     66.97

****************************************************************************
  FLOW PROCESS FROM NODE    235.00 TO NODE    236.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8200
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =    75.00
  UPSTREAM ELEVATION(FEET) =     55.00
  DOWNSTREAM ELEVATION(FEET) =     54.25
  ELEVATION DIFFERENCE(FEET) =      0.75
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.904
  WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
           THE MAXIMUM OVERLAND FLOW LENGTH =    60.00
           (Reference: Table 3-1B of Hydrology Manual)
           THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.850
  NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
  SUBAREA RUNOFF(CFS) =      2.02
  TOTAL AREA(ACRES) =      0.36   TOTAL RUNOFF(CFS) =      2.02

****************************************************************************
  FLOW PROCESS FROM NODE    236.00 TO NODE    237.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =   54.25  DOWNSTREAM ELEVATION(FEET) =   45.80
  STREET LENGTH(FEET) =   248.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
```

```
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       5.44
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.29
    HALFSTREET FLOOD WIDTH(FEET) =    8.09
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    3.52
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    1.01
  STREET FLOW TRAVEL TIME(MIN.) =   1.17   Tc(MIN.) =    5.08
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.783
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8700
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.858
  SUBAREA AREA(ACRES) =    1.16      SUBAREA RUNOFF(CFS) =    6.85
  TOTAL AREA(ACRES) =        1.5        PEAK FLOW RATE(CFS) =       8.85

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.33   HALFSTREET FLOOD WIDTH(FEET) =  10.12
  FLOW VELOCITY(FEET/SEC.) =  3.87   DEPTH*VELOCITY(FT*FT/SEC.) =   1.27
  LONGEST FLOWPATH FROM NODE    235.00 TO NODE    237.00 =     323.00 FEET.

****************************************************************************
  FLOW PROCESS FROM NODE    237.00 TO NODE    237.00 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.) =    5.08
  RAINFALL INTENSITY(INCH/HR) =   6.78
  TOTAL STREAM AREA(ACRES) =     1.52
  PEAK FLOW RATE(CFS) AT CONFLUENCE =      8.85

  ** CONFLUENCE DATA **
  STREAM     RUNOFF       Tc      INTENSITY      AREA
  NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
      1       66.97    15.32        3.327         34.05
      2        8.85     5.08        6.783          1.52

  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       41.70     5.08       6.783
      2       71.31    15.32       3.327

  COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
  PEAK FLOW RATE(CFS) =      71.31   Tc(MIN.) =   15.32
  TOTAL AREA(ACRES) =       35.6
  LONGEST FLOWPATH FROM NODE    205.00 TO NODE    237.00 =    3585.00 FEET.
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =       35.6  TC(MIN.) =     15.32
  PEAK FLOW RATE(CFS)   =      71.31
============================================================================
```

==

END OF RATIONAL METHOD ANALYSIS

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

            RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE
            Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT
                         2003,1985,1981 HYDROLOGY MANUAL
         (c) Copyright 1982-2016 Advanced Engineering Software (aes)
             Ver. 23.0 Release Date: 07/01/2016  License ID 1355

                           Analysis prepared by:

                             Fuscoe Engineering
                          6390 Greenich Dr Ste 170
                               San Diego, CA
                                   92122

 ************************** DESCRIPTION OF STUDY **************************
 * THE HOME DEPOT - MISSION VALLEY - PRE-DEVELOPMENT STUDY                  *
 * SERIES 3                                                                 *
 * SAN DIEGO, CALIFORNIA                                                    *
 **************************************************************************

  FILE NAME: SR300EX.DAT
  TIME/DATE OF STUDY: 09:41 04/10/2020
----------------------------------------------------------------------------
  USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:
----------------------------------------------------------------------------
  2003 SAN DIEGO MANUAL CRITERIA

  USER SPECIFIED STORM EVENT(YEAR) = 100.00
  6-HOUR DURATION PRECIPITATION (INCHES) =   2.600
  SPECIFIED MINIMUM PIPE SIZE(INCH) =  12.00
  SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
  SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
  NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
  *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
  2   15.0     10.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0160

  GLOBAL STREET FLOW-DEPTH CONSTRAINTS:
    1. Relative Flow-Depth =  1.00 FEET
       as (Maximum Allowable Street Flow Depth) - (Top-of-Curb)
    2. (Depth)*(Velocity) Constraint =  1.0 (FT*FT/S)
  *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
   OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*

****************************************************************************
  FLOW PROCESS FROM NODE    300.00 TO NODE    301.00 IS CODE =  21
----------------------------------------------------------------------------
  >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .3800
  S.C.S. CURVE NUMBER (AMC II) =   0
  INITIAL SUBAREA FLOW-LENGTH(FEET) =   100.00
  UPSTREAM ELEVATION(FEET) =     70.00
```

```
  DOWNSTREAM ELEVATION(FEET) =     60.00
  ELEVATION DIFFERENCE(FEET) =     10.00
  SUBAREA OVERLAND TIME OF FLOW(MIN.) =    6.016
  WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.080
  SUBAREA RUNOFF(CFS) =      0.58
  TOTAL AREA(ACRES) =      0.25   TOTAL RUNOFF(CFS) =      0.58

****************************************************************************
  FLOW PROCESS FROM NODE    301.00 TO NODE    302.00 IS CODE =  62
----------------------------------------------------------------------------
  >>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<
  >>>>>(STREET TABLE SECTION #  2 USED)<<<<<
============================================================================
  UPSTREAM ELEVATION(FEET) =   60.00  DOWNSTREAM ELEVATION(FEET) =   49.00
  STREET LENGTH(FEET) =   660.00   CURB HEIGHT(INCHES) =  6.0
  STREET HALFWIDTH(FEET) = 15.00

  DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) =  10.00
  INSIDE STREET CROSSFALL(DECIMAL) =  0.020
  OUTSIDE STREET CROSSFALL(DECIMAL)  =  0.020

  SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF =  2
  STREET PARKWAY CROSSFALL(DECIMAL)  =  0.020
  Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) =   0.0160
  Manning's FRICTION FACTOR for Back-of-Walk Flow Section =   0.0150

    **TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       5.33
    STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:
    STREET FLOW DEPTH(FEET) =  0.31
    HALFSTREET FLOOD WIDTH(FEET) =    9.41
    AVERAGE FLOW VELOCITY(FEET/SEC.) =    2.65
    PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) =    0.83
  STREET FLOW TRAVEL TIME(MIN.) =   4.14   Tc(MIN.) =   10.16
   100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.336
  *USER SPECIFIED(SUBAREA):
  USER-SPECIFIED RUNOFF COEFFICIENT = .8200
  S.C.S. CURVE NUMBER (AMC II) =   0
  AREA-AVERAGE RUNOFF COEFFICIENT =  0.783
  SUBAREA AREA(ACRES) =    2.74      SUBAREA RUNOFF(CFS) =    9.74
  TOTAL AREA(ACRES) =        3.0        PEAK FLOW RATE(CFS) =      10.15

  END OF SUBAREA STREET FLOW HYDRAULICS:
  DEPTH(FEET) = 0.37   HALFSTREET FLOOD WIDTH(FEET) =  12.38
  FLOW VELOCITY(FEET/SEC.) =  3.07   DEPTH*VELOCITY(FT*FT/SEC.) =   1.15
  LONGEST FLOWPATH FROM NODE    300.00 TO NODE    302.00 =     760.00 FEET.
============================================================================
  END OF STUDY SUMMARY:
  TOTAL AREA(ACRES)     =        3.0  TC(MIN.) =     10.16
  PEAK FLOW RATE(CFS)   =      10.15
============================================================================
============================================================================
  END OF RATIONAL METHOD ANALYSIS
```

PROPOSED AES CALCULATIONS
50 YEAR STORM EVENT

****

RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE

Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT

2003,1985,1981 HYDROLOGY MANUAL

Ver. 21.0 Release Date: 06/01/2014 License ID 1355

Analysis prepared by:

Fuscoe Engineering

16795 Von Karman

Suite 100

Irvine, California 92606

************************** DESCRIPTION OF STUDY **************************

* THD SR *

* PROPOSED 50 YEAR *

* *

****

FILE NAME: THDPR50.DAT

TIME/DATE OF STUDY: 15:38 05/25/2020

USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:

2003 SAN DIEGO MANUAL CRITERIA

```
 USER SPECIFIED STORM EVENT(YEAR) =  50.00
 6-HOUR DURATION PRECIPITATION (INCHES) =   2.100
 SPECIFIED MINIMUM PIPE SIZE(INCH) =  12.00
 SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
 SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
 NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
 *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
 2   25.0     20.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0160
 3   15.0     10.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0160

 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 =  0.1 (FT*FT/S)
 *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
  OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*

****************************************************************************
 FLOW PROCESS FROM NODE    100.00 TO NODE    101.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6800
```

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 77.00

UPSTREAM ELEVATION(FEET) = 360.00

DOWNSTREAM ELEVATION(FEET) = 359.35

ELEVATION DIFFERENCE(FEET) = 0.65

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 6.213

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.32

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.810

SUBAREA RUNOFF(CFS) = 0.52

TOTAL AREA(ACRES) = 0.16 TOTAL RUNOFF(CFS) = 0.52

****

FLOW PROCESS FROM NODE 101.00 TO NODE 102.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 3 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 359.35 DOWNSTREAM ELEVATION(FEET) = 354.00

STREET LENGTH(FEET) = 308.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 15.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 10.00

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) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 2.39

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.31

HALFSTREET FLOOD WIDTH(FEET) = 8.95

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.61

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.80

STREET FLOW TRAVEL TIME(MIN.) = 1.97 Tc(MIN.) = 8.18

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.027

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .7400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.733

SUBAREA AREA(ACRES) = 1.25 SUBAREA RUNOFF(CFS) = 3.72

TOTAL AREA(ACRES) = 1.4 PEAK FLOW RATE(CFS) = 4.16

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.35 HALFSTREET FLOOD WIDTH(FEET) = 11.37

FLOW VELOCITY(FEET/SEC.) = 2.95 DEPTH*VELOCITY(FT*FT/SEC.) = 1.04

LONGEST FLOWPATH FROM NODE 100.00 TO NODE 102.00 = 385.00 FEET.

****

FLOW PROCESS FROM NODE 102.00 TO NODE 103.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 354.00 DOWNSTREAM(FEET) = 70.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 801.00 CHANNEL SLOPE = 0.3546

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.030 MAXIMUM DEPTH(FEET) = 4.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.627

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .4100

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 7.87

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 9.26

AVERAGE FLOW DEPTH(FEET) = 0.19 TRAVEL TIME(MIN.) = 1.44

Tc(MIN.) = 9.62

SUBAREA AREA(ACRES) = 4.92 SUBAREA RUNOFF(CFS) = 7.32

AREA-AVERAGE RUNOFF COEFFICIENT = 0.482

TOTAL AREA(ACRES) = 6.3 PEAK FLOW RATE(CFS) = 11.07

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.24 FLOW VELOCITY(FEET/SEC.) = 10.32

LONGEST FLOWPATH FROM NODE 100.00 TO NODE 103.00 = 1186.00 FEET.

****

FLOW PROCESS FROM NODE 103.00 TO NODE 103.00 IS CODE = 10

>>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 1 <<<<<

==

****

FLOW PROCESS FROM NODE 110.00 TO NODE 111.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 121.00

UPSTREAM ELEVATION(FEET) = 345.00

DOWNSTREAM ELEVATION(FEET) = 335.00

ELEVATION DIFFERENCE(FEET) = 10.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 6.678

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 100.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.591

SUBAREA RUNOFF(CFS) = 0.63

TOTAL AREA(ACRES) = 0.39 TOTAL RUNOFF(CFS) = 0.63

****

FLOW PROCESS FROM NODE 111.00 TO NODE 112.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 335.00 DOWNSTREAM(FEET) = 125.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 436.00 CHANNEL SLOPE = 0.4817

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.030 MAXIMUM DEPTH(FEET) = 4.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.109

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 1.91

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 5.80

AVERAGE FLOW DEPTH(FEET) = 0.08 TRAVEL TIME(MIN.) = 1.25

Tc(MIN.) = 7.93

SUBAREA AREA(ACRES) = 1.77 SUBAREA RUNOFF(CFS) = 2.55

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 2.2 PEAK FLOW RATE(CFS) = 3.11

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.10 FLOW VELOCITY(FEET/SEC.) = 7.24

LONGEST FLOWPATH FROM NODE 110.00 TO NODE 112.00 = 557.00 FEET.

****

FLOW PROCESS FROM NODE 112.00 TO NODE 117.00 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.) = 7.93

RAINFALL INTENSITY(INCH/HR) = 4.11

TOTAL STREAM AREA(ACRES) = 2.16

PEAK FLOW RATE(CFS) AT CONFLUENCE = 3.11

****

FLOW PROCESS FROM NODE 115.00 TO NODE 116.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 205.00

UPSTREAM ELEVATION(FEET) = 117.00

DOWNSTREAM ELEVATION(FEET) = 113.50

ELEVATION DIFFERENCE(FEET) = 3.50

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 10.141

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 80.61

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.506

SUBAREA RUNOFF(CFS) = 0.20

TOTAL AREA(ACRES) = 0.16 TOTAL RUNOFF(CFS) = 0.20

****

FLOW PROCESS FROM NODE 116.00 TO NODE 117.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 113.50 DOWNSTREAM(FEET) = 105.50

CHANNEL LENGTH THRU SUBAREA(FEET) = 588.00 CHANNEL SLOPE = 0.0136

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   4.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 2.635

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) =   0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =   0.46

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.73

AVERAGE FLOW DEPTH(FEET) =   0.06   TRAVEL TIME(MIN.) =   5.65

Tc(MIN.) =   15.79

SUBAREA AREA(ACRES) =   0.56   SUBAREA RUNOFF(CFS) =   0.52

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) =   0.7   PEAK FLOW RATE(CFS) =   0.66

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.08   FLOW VELOCITY(FEET/SEC.) =   2.02

LONGEST FLOWPATH FROM NODE   115.00 TO NODE   117.00 =   793.00 FEET.

****

FLOW PROCESS FROM NODE   112.00 TO NODE   117.00 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.) =   15.79

RAINFALL INTENSITY(INCH/HR) =   2.64

TOTAL STREAM AREA(ACRES) =   0.72

PEAK FLOW RATE(CFS) AT CONFLUENCE =   0.66

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	3.11	7.93	4.109	2.16
2	0.66	15.79	2.635	0.72

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	3.44	7.93	4.109
2	2.66	15.79	2.635

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 3.44 Tc(MIN.) = 7.93

TOTAL AREA(ACRES) = 2.9

LONGEST FLOWPATH FROM NODE 115.00 TO NODE 117.00 = 793.00 FEET.

****

FLOW PROCESS FROM NODE 117.00 TO NODE 122.00 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.) = 7.93

RAINFALL INTENSITY(INCH/HR) = 4.11

TOTAL STREAM AREA(ACRES) = 2.88

PEAK FLOW RATE(CFS) AT CONFLUENCE = 3.44

****

FLOW PROCESS FROM NODE 120.00 TO NODE 121.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 99.00

UPSTREAM ELEVATION(FEET) = 80.50

DOWNSTREAM ELEVATION(FEET) = 80.10

ELEVATION DIFFERENCE(FEET) = 0.40

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 12.027

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!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.141

SUBAREA RUNOFF(CFS) = 0.12

TOTAL AREA(ACRES) = 0.11 TOTAL RUNOFF(CFS) = 0.12

****

FLOW PROCESS FROM NODE 121.00 TO NODE 122.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 80.10 DOWNSTREAM(FEET) = 73.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 364.00 CHANNEL SLOPE = 0.0195

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.015 MAXIMUM DEPTH(FEET) = 4.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 2.674

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 0.31

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 1.78

AVERAGE FLOW DEPTH(FEET) = 0.04 TRAVEL TIME(MIN.) = 3.41

Tc(MIN.) = 15.43

SUBAREA AREA(ACRES) = 0.41 SUBAREA RUNOFF(CFS) = 0.38

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 0.5 PEAK FLOW RATE(CFS) = 0.49

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.06 FLOW VELOCITY(FEET/SEC.) = 2.03

LONGEST FLOWPATH FROM NODE 120.00 TO NODE 122.00 = 463.00 FEET.

****

FLOW PROCESS FROM NODE 122.00 TO NODE 122.00 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.) = 15.43

RAINFALL INTENSITY(INCH/HR) = 2.67

TOTAL STREAM AREA(ACRES) = 0.52

PEAK FLOW RATE(CFS) AT CONFLUENCE = 0.49

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	3.44	7.93	4.109	2.88
2	0.49	15.43	2.674	0.52

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	3.69	7.93	4.109
2	2.73	15.43	2.674

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 3.69 Tc(MIN.) = 7.93

TOTAL AREA(ACRES) = 3.4

LONGEST FLOWPATH FROM NODE 115.00 TO NODE 122.00 = 793.00 FEET.

****

FLOW PROCESS FROM NODE 122.00 TO NODE 103.00 IS CODE = 11

>>>>>CONFLUENCE MEMORY BANK # 1 WITH THE MAIN-STREAM MEMORY<<<<<

==

** MAIN STREAM CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	3.69	7.93	4.109	3.40

LONGEST FLOWPATH FROM NODE 115.00 TO NODE 103.00 = 793.00 FEET.

** MEMORY BANK # 1 CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	11.07	9.62	3.627	6.33

LONGEST FLOWPATH FROM NODE 100.00 TO NODE 103.00 = 1186.00 FEET.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	12.81	7.93	4.109
2	14.32	9.62	3.627

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 14.32 Tc(MIN.) = 9.62

TOTAL AREA(ACRES) = 9.7

****

FLOW PROCESS FROM NODE 122.00 TO NODE 103.00 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.) = 9.62

RAINFALL INTENSITY(INCH/HR) = 3.63

TOTAL STREAM AREA(ACRES) = 9.73

PEAK FLOW RATE(CFS) AT CONFLUENCE = 14.32

****

FLOW PROCESS FROM NODE 130.00 TO NODE 131.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 99.00

UPSTREAM ELEVATION(FEET) = 115.00

DOWNSTREAM ELEVATION(FEET) = 85.00

ELEVATION DIFFERENCE(FEET) = 30.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 6.235

WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.799

SUBAREA RUNOFF(CFS) = 0.22

TOTAL AREA(ACRES) = 0.13 TOTAL RUNOFF(CFS) = 0.22

****

FLOW PROCESS FROM NODE 131.00 TO NODE 132.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 85.00 DOWNSTREAM(FEET) = 80.50

CHANNEL LENGTH THRU SUBAREA(FEET) = 271.00 CHANNEL SLOPE = 0.0166

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.015 MAXIMUM DEPTH(FEET) = 4.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.877

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 0.44

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 1.85

AVERAGE FLOW DEPTH(FEET) = 0.06 TRAVEL TIME(MIN.) = 2.44

Tc(MIN.) = 8.68

SUBAREA AREA(ACRES) = 0.33 SUBAREA RUNOFF(CFS) = 0.45

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 0.5 PEAK FLOW RATE(CFS) = 0.62

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.07 FLOW VELOCITY(FEET/SEC.) = 2.11

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 132.00 = 370.00 FEET.

****

FLOW PROCESS FROM NODE 132.00 TO NODE 135.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 80.50 DOWNSTREAM(FEET) = 60.00

FLOW LENGTH(FEET) = 60.00 MANNING'S N = 0.013

ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000

DEPTH OF FLOW IN 12.0 INCH PIPE IS 1.5 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 11.49

ESTIMATED PIPE DIAMETER(INCH) = 12.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 0.62

PIPE TRAVEL TIME(MIN.) = 0.09 Tc(MIN.) = 8.76

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 135.00 = 430.00 FEET.

****

FLOW PROCESS FROM NODE 133.00 TO NODE 135.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 90.00 DOWNSTREAM(FEET) = 60.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 332.00 CHANNEL SLOPE = 0.0904

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.015 MAXIMUM DEPTH(FEET) = 4.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.530

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

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.) = 4.34

AVERAGE FLOW DEPTH(FEET) = 0.05 TRAVEL TIME(MIN.) = 1.27

Tc(MIN.) = 10.04

SUBAREA AREA(ACRES) = 0.45 SUBAREA RUNOFF(CFS) = 0.56

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 0.9 PEAK FLOW RATE(CFS) = 1.12

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.06 FLOW VELOCITY(FEET/SEC.) = 4.69

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 135.00 = 762.00 FEET.

****

FLOW PROCESS FROM NODE 135.00 TO NODE 136.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 60.00 DOWNSTREAM(FEET) = 55.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 476.00 CHANNEL SLOPE = 0.0105

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.015 MAXIMUM DEPTH(FEET) = 4.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 2.962

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 1.43

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 2.53

AVERAGE FLOW DEPTH(FEET) = 0.13 TRAVEL TIME(MIN.) = 3.13

Tc(MIN.) = 13.17

SUBAREA AREA(ACRES) = 0.59 SUBAREA RUNOFF(CFS) = 0.61

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 1.5 PEAK FLOW RATE(CFS) = 1.56

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.14 FLOW VELOCITY(FEET/SEC.) = 2.59

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 136.00 = 1238.00 FEET.

****

FLOW PROCESS FROM NODE 136.00 TO NODE 103.00 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.) = 13.17

RAINFALL INTENSITY(INCH/HR) = 2.96

TOTAL STREAM AREA(ACRES) = 1.50

PEAK FLOW RATE(CFS) AT CONFLUENCE = 1.56

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	14.32	9.62	3.627	9.73
2	1.56	13.17	2.962	1.50

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)

1	15.46	9.62	3.627
2	13.25	13.17	2.962

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 15.46 Tc(MIN.) = 9.62

TOTAL AREA(ACRES) = 11.2

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 103.00 = 1238.00 FEET.

****

FLOW PROCESS FROM NODE 103.00 TO NODE 144.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 70.50 DOWNSTREAM(FEET) = 41.50

FLOW LENGTH(FEET) = 450.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 15.0 INCH PIPE IS 12.2 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 14.46

ESTIMATED PIPE DIAMETER(INCH) = 15.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 15.46

PIPE TRAVEL TIME(MIN.) = 0.52 Tc(MIN.) = 10.14

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 144.00 = 1688.00 FEET.

****

FLOW PROCESS FROM NODE 103.00 TO NODE 144.00 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.) = 10.14

RAINFALL INTENSITY(INCH/HR) = 3.51

TOTAL STREAM AREA(ACRES) = 11.23

PEAK FLOW RATE(CFS) AT CONFLUENCE = 15.46

****

FLOW PROCESS FROM NODE 137.00 TO NODE 138.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 105.00

UPSTREAM ELEVATION(FEET) = 56.00

DOWNSTREAM ELEVATION(FEET) = 50.50

ELEVATION DIFFERENCE(FEET) = 5.50

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 2.563

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 90.48

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 0.65

TOTAL AREA(ACRES) = 0.14 TOTAL RUNOFF(CFS) = 0.65

****

FLOW PROCESS FROM NODE 138.00 TO NODE 139.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 50.50 DOWNSTREAM(FEET) = 49.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 243.00 CHANNEL SLOPE = 0.0062

CHANNEL BASE(FEET) = 1.50 "Z" FACTOR = 0.500

MANNING'S FACTOR = 0.015 MAXIMUM DEPTH(FEET) = 0.50

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 2.00

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 3.21

AVERAGE FLOW DEPTH(FEET) = 0.37 TRAVEL TIME(MIN.) = 1.26

Tc(MIN.) = 3.82

SUBAREA AREA(ACRES) = 0.58 SUBAREA RUNOFF(CFS) = 2.70

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

TOTAL AREA(ACRES) = 0.7 PEAK FLOW RATE(CFS) = 3.35

==>>WARNING: FLOW IN CHANNEL EXCEEDS CHANNEL

CAPACITY(NORMAL DEPTH EQUAL TO SPECIFIED MAXIMUM

ALLOWABLE DEPTH).

AS AN APPROXIMATION, FLOWDEPTH IS SET AT MAXIMUM

ALLOWABLE DEPTH AND IS USED FOR TRAVELTIME CALCULATIONS.

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.50 FLOW VELOCITY(FEET/SEC.) = 3.82

==>FLOWDEPTH EXCEEDS MAXIMUM ALLOWABLE DEPTH

LONGEST FLOWPATH FROM NODE 137.00 TO NODE 139.00 = 348.00 FEET.

****

FLOW PROCESS FROM NODE 139.00 TO NODE 144.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 49.00 DOWNSTREAM(FEET) = 39.00

FLOW LENGTH(FEET) = 20.00 MANNING'S N = 0.013

ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000

DEPTH OF FLOW IN 12.0 INCH PIPE IS 3.0 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 21.45

ESTIMATED PIPE DIAMETER(INCH) = 12.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 3.35

PIPE TRAVEL TIME(MIN.) = 0.02 Tc(MIN.) = 3.84

LONGEST FLOWPATH FROM NODE 137.00 TO NODE 144.00 = 368.00 FEET.

****

FLOW PROCESS FROM NODE 139.00 TO NODE 144.00 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.) = 3.84

RAINFALL INTENSITY(INCH/HR) = 5.53

TOTAL STREAM AREA(ACRES) = 0.72

PEAK FLOW RATE(CFS) AT CONFLUENCE = 3.35

****

FLOW PROCESS FROM NODE 140.00 TO NODE 141.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 112.00

UPSTREAM ELEVATION(FEET) = 54.00

DOWNSTREAM ELEVATION(FEET) = 50.00

ELEVATION DIFFERENCE(FEET) = 4.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 2.847

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 86.43

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 2.74

TOTAL AREA(ACRES) = 0.59 TOTAL RUNOFF(CFS) = 2.74

****

FLOW PROCESS FROM NODE 141.00 TO NODE 142.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 50.00 DOWNSTREAM(FEET) = 45.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 417.00 CHANNEL SLOPE = 0.0120

CHANNEL BASE(FEET) = 3.00 "Z" FACTOR = 1.000

MANNING'S FACTOR = 0.030 MAXIMUM DEPTH(FEET) = 1.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.391

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 5.16

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 2.95

AVERAGE FLOW DEPTH(FEET) = 0.50 TRAVEL TIME(MIN.) = 2.36

Tc(MIN.) = 5.21

SUBAREA AREA(ACRES) = 1.07 SUBAREA RUNOFF(CFS) = 4.85

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

TOTAL AREA(ACRES) = 1.7 PEAK FLOW RATE(CFS) = 7.52

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.63 FLOW VELOCITY(FEET/SEC.) = 3.30

LONGEST FLOWPATH FROM NODE 140.00 TO NODE 142.00 = 529.00 FEET.

****

FLOW PROCESS FROM NODE 142.00 TO NODE 144.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

```
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
==============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    45.00  DOWNSTREAM(FEET) =    39.00
 FLOW LENGTH(FEET) =   170.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  15.0 INCH PIPE IS   8.8 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   9.99
 ESTIMATED PIPE DIAMETER(INCH) =  15.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =       7.52
 PIPE TRAVEL TIME(MIN.) =   0.28    Tc(MIN.) =    5.49
 LONGEST FLOWPATH FROM NODE    140.00 TO NODE    144.00 =     699.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    142.00 TO NODE    144.00 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.) =    5.49
 RAINFALL INTENSITY(INCH/HR) =   5.21
 TOTAL STREAM AREA(ACRES) =     1.66
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      7.52

 ** CONFLUENCE DATA **
 STREAM     RUNOFF      Tc     INTENSITY     AREA
 NUMBER      (CFS)     (MIN.)   (INCH/HOUR)    (ACRE)
    1      15.46    10.14       3.506        11.23
    2       3.35     3.84       5.533         0.72
```

3 7.52 5.49 5.210 1.66

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 3 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	18.40	3.84	5.533
2	21.07	5.49	5.210
3	22.64	10.14	3.506

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 22.64 Tc(MIN.) = 10.14

TOTAL AREA(ACRES) = 13.6

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 144.00 = 1688.00 FEET.

****

FLOW PROCESS FROM NODE 144.00 TO NODE 145.00 IS CODE = 10

>>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 2 <<<<<

==

****

FLOW PROCESS FROM NODE 150.00 TO NODE 151.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 167.00

UPSTREAM ELEVATION(FEET) = 52.50

DOWNSTREAM ELEVATION(FEET) = 50.00

ELEVATION DIFFERENCE(FEET) = 2.50

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.360

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 67.46

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 2.23

TOTAL AREA(ACRES) = 0.48 TOTAL RUNOFF(CFS) = 2.23

****

FLOW PROCESS FROM NODE 151.00 TO NODE 152.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 2 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 50.00 DOWNSTREAM ELEVATION(FEET) = 47.30

STREET LENGTH(FEET) = 160.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 25.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 20.00

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) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 3.46

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.34

HALFSTREET FLOOD WIDTH(FEET) = 10.59

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.80

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.94

STREET FLOW TRAVEL TIME(MIN.) = 0.95 Tc(MIN.) = 4.31

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

SUBAREA AREA(ACRES) = 0.53 SUBAREA RUNOFF(CFS) = 2.46

TOTAL AREA(ACRES) = 1.0 PEAK FLOW RATE(CFS) = 4.69

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.37 HALFSTREET FLOOD WIDTH(FEET) = 11.99

FLOW VELOCITY(FEET/SEC.) = 3.02 DEPTH*VELOCITY(FT*FT/SEC.) = 1.10

LONGEST FLOWPATH FROM NODE 150.00 TO NODE 152.00 = 327.00 FEET.

****

FLOW PROCESS FROM NODE 152.00 TO NODE 153.00 IS CODE = 31

```
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
==============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    47.30  DOWNSTREAM(FEET) =    46.00
 FLOW LENGTH(FEET) =    75.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  15.0 INCH PIPE IS   8.2 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =   6.82
 ESTIMATED PIPE DIAMETER(INCH) =  15.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =       4.69
 PIPE TRAVEL TIME(MIN.) =   0.18    Tc(MIN.) =    4.50
 LONGEST FLOWPATH FROM NODE    150.00 TO NODE    153.00 =     402.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    152.00 TO NODE    153.00 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.) =    4.50
 RAINFALL INTENSITY(INCH/HR) =   5.53
 TOTAL STREAM AREA(ACRES) =     1.01
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      4.69

****************************************************************************
 FLOW PROCESS FROM NODE     16.00 TO NODE    161.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
```

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 93.00

UPSTREAM ELEVATION(FEET) = 55.30

DOWNSTREAM ELEVATION(FEET) = 54.70

ELEVATION DIFFERENCE(FEET) = 0.60

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.939

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 52.90

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 0.51

TOTAL AREA(ACRES) = 0.11 TOTAL RUNOFF(CFS) = 0.51

****

FLOW PROCESS FROM NODE 161.00 TO NODE 162.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 3 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 54.70 DOWNSTREAM ELEVATION(FEET) = 44.00

STREET LENGTH(FEET) = 340.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 15.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 10.00

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) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 1.14

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.23

HALFSTREET FLOOD WIDTH(FEET) = 5.27

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.87

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.66

STREET FLOW TRAVEL TIME(MIN.) = 1.98 Tc(MIN.) = 5.91

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.965

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

SUBAREA AREA(ACRES) = 0.30 SUBAREA RUNOFF(CFS) = 1.25

TOTAL AREA(ACRES) = 0.4 PEAK FLOW RATE(CFS) = 1.71

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.26 HALFSTREET FLOOD WIDTH(FEET) = 6.60

FLOW VELOCITY(FEET/SEC.) = 3.09 DEPTH*VELOCITY(FT*FT/SEC.) = 0.80

LONGEST FLOWPATH FROM NODE 16.00 TO NODE 162.00 = 433.00 FEET.

****

FLOW PROCESS FROM NODE 162.00 TO NODE 153.00 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.) = 5.91

RAINFALL INTENSITY(INCH/HR) = 4.96

TOTAL STREAM AREA(ACRES) = 0.41

PEAK FLOW RATE(CFS) AT CONFLUENCE = 1.71

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	4.69	4.50	5.533	1.01
2	1.71	5.91	4.965	0.41

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	5.99	4.50	5.533
2	5.92	5.91	4.965

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 5.99 Tc(MIN.) = 4.50

TOTAL AREA(ACRES) = 1.4

LONGEST FLOWPATH FROM NODE 16.00 TO NODE 153.00 = 433.00 FEET.

****

FLOW PROCESS FROM NODE 155.00 TO NODE 153.00 IS CODE = 81

>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<

==

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.8400

SUBAREA AREA(ACRES) = 0.41 SUBAREA RUNOFF(CFS) = 1.91

TOTAL AREA(ACRES) = 1.8 TOTAL RUNOFF(CFS) = 8.51

TC(MIN.) = 4.50

****

FLOW PROCESS FROM NODE 153.00 TO NODE 145.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 2 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 43.60 DOWNSTREAM ELEVATION(FEET) = 42.50

STREET LENGTH(FEET) = 275.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 25.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 20.00

INSIDE STREET CROSSFALL(DECIMAL) = 0.020

OUTSIDE STREET CROSSFALL(DECIMAL) = 0.020

SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF = 2

STREET PARKWAY CROSSFALL(DECIMAL) = 0.020

Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 9.11

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.44

HALFSTREET FLOOD WIDTH(FEET) = 15.82

AVERAGE FLOW VELOCITY(FEET/SEC.) = 1.74

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.77

STREET FLOW TRAVEL TIME(MIN.) = 2.64 Tc(MIN.) = 7.13

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.400

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

SUBAREA AREA(ACRES) = 0.33 SUBAREA RUNOFF(CFS) = 1.22

TOTAL AREA(ACRES) = 2.2 PEAK FLOW RATE(CFS) = 8.51

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.43 HALFSTREET FLOOD WIDTH(FEET) = 15.43

FLOW VELOCITY(FEET/SEC.) = 1.70 DEPTH*VELOCITY(FT*FT/SEC.) = 0.74

LONGEST FLOWPATH FROM NODE 16.00 TO NODE 145.00 = 708.00 FEET.

****

FLOW PROCESS FROM NODE 153.00 TO NODE 145.00 IS CODE = 11

>>>>>CONFLUENCE MEMORY BANK # 2 WITH THE MAIN-STREAM MEMORY<<<<<

==

** MAIN STREAM CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	8.51	7.13	4.400	2.16

LONGEST FLOWPATH FROM NODE 16.00 TO NODE 145.00 = 708.00 FEET.

** MEMORY BANK # 2 CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	22.64	10.14	3.506	13.61

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 145.00 = 1688.00 FEET.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	24.42	7.13	4.400
2	29.42	10.14	3.506

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 29.42 Tc(MIN.) = 10.14

TOTAL AREA(ACRES) = 15.8

****

FLOW PROCESS FROM NODE 145.00 TO NODE 145.00 IS CODE = 12

>>>>>CLEAR MEMORY BANK # 1 <<<<<

==

****

FLOW PROCESS FROM NODE 145.00 TO NODE 145.00 IS CODE = 12

>>>>>CLEAR MEMORY BANK # 2 <<<<<

==

****

FLOW PROCESS FROM NODE 218.00 TO NODE 218.00 IS CODE = 7

>>>>>USER SPECIFIED HYDROLOGY INFORMATION AT NODE<<<<<

==

USER-SPECIFIED VALUES ARE AS FOLLOWS:

TC(MIN) = 14.08 RAIN INTENSITY(INCH/HOUR) = 2.84

TOTAL AREA(ACRES) = 20.62 TOTAL RUNOFF(CFS) = 42.37

****

FLOW PROCESS FROM NODE 218.00 TO NODE 219.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 189.00 DOWNSTREAM(FEET) = 112.50

CHANNEL LENGTH THRU SUBAREA(FEET) = 705.00 CHANNEL SLOPE = 0.1085

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.030 MAXIMUM DEPTH(FEET) = 4.00

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 2.709

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3800

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 47.17

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 11.24

AVERAGE FLOW DEPTH(FEET) = 0.76 TRAVEL TIME(MIN.) = 1.05

Tc(MIN.) = 15.13

SUBAREA AREA(ACRES) = 9.31 SUBAREA RUNOFF(CFS) = 9.59

AREA-AVERAGE RUNOFF COEFFICIENT = 0.617

TOTAL AREA(ACRES) = 29.9 PEAK FLOW RATE(CFS) = 50.04

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.79 FLOW VELOCITY(FEET/SEC.) = 11.38

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 219.00 = 2393.00 FEET.

****

FLOW PROCESS FROM NODE 219.00 TO NODE 219.00 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.) = 15.13

RAINFALL INTENSITY(INCH/HR) = 2.71

TOTAL STREAM AREA(ACRES) = 29.93

PEAK FLOW RATE(CFS) AT CONFLUENCE = 50.04

****

FLOW PROCESS FROM NODE 225.00 TO NODE 226.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .7100

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 65.00

UPSTREAM ELEVATION(FEET) = 358.00

DOWNSTREAM ELEVATION(FEET) = 357.35

ELEVATION DIFFERENCE(FEET) = 0.65

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 5.438

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.241

SUBAREA RUNOFF(CFS) = 0.41

TOTAL AREA(ACRES) = 0.11 TOTAL RUNOFF(CFS) = 0.41

****

FLOW PROCESS FROM NODE 226.00 TO NODE 227.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 3 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 357.35 DOWNSTREAM ELEVATION(FEET) = 346.00

STREET LENGTH(FEET) = 350.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 15.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 10.00

INSIDE STREET CROSSFALL(DECIMAL) = 0.020

OUTSIDE STREET CROSSFALL(DECIMAL) = 0.020

SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF = 2

STREET PARKWAY CROSSFALL(DECIMAL) = 0.020

Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 1.93

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.22

HALFSTREET FLOOD WIDTH(FEET) = 4.75

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.80

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.62

STREET FLOW TRAVEL TIME(MIN.) = 2.08 Tc(MIN.) = 7.52

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.252

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .7400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.737

SUBAREA AREA(ACRES) = 0.96 SUBAREA RUNOFF(CFS) = 3.02

TOTAL AREA(ACRES) = 1.1 PEAK FLOW RATE(CFS) = 3.35

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.26 HALFSTREET FLOOD WIDTH(FEET) = 6.52

FLOW VELOCITY(FEET/SEC.) = 3.08 DEPTH*VELOCITY(FT*FT/SEC.) = 0.79

LONGEST FLOWPATH FROM NODE 225.00 TO NODE 227.00 = 415.00 FEET.

```
****************************************************************************
 FLOW PROCESS FROM NODE    227.00 TO NODE    219.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =   346.00  DOWNSTREAM(FEET) =   112.50
 FLOW LENGTH(FEET) =   575.00   MANNING'S N =  0.013
 ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000
 DEPTH OF FLOW IN  12.0 INCH PIPE IS   3.2 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  19.92
 ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =       3.35
 PIPE TRAVEL TIME(MIN.) =   0.48    Tc(MIN.) =    8.00
 LONGEST FLOWPATH FROM NODE    225.00 TO NODE    219.00 =     990.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    219.00 TO NODE    219.00 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.) =    8.00
 RAINFALL INTENSITY(INCH/HR) =   4.09
 TOTAL STREAM AREA(ACRES) =     1.07
 PEAK FLOW RATE(CFS) AT CONFLUENCE =      3.35
```

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	50.04	15.13	2.709	29.93
2	3.35	8.00	4.086	1.07

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	29.83	8.00	4.086
2	52.27	15.13	2.709

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 52.27 Tc(MIN.) = 15.13

TOTAL AREA(ACRES) = 31.0

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 219.00 = 2393.00 FEET.

****

FLOW PROCESS FROM NODE 219.00 TO NODE 220.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 112.50 DOWNSTREAM(FEET) = 46.00

FLOW LENGTH(FEET) = 950.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 24.0 INCH PIPE IS 18.1 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 20.51

ESTIMATED PIPE DIAMETER(INCH) = 24.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 52.27

PIPE TRAVEL TIME(MIN.) = 0.77 Tc(MIN.) = 15.90

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 220.00 = 3343.00 FEET.

****

FLOW PROCESS FROM NODE 219.00 TO NODE 220.00 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.) = 15.90

RAINFALL INTENSITY(INCH/HR) = 2.62

TOTAL STREAM AREA(ACRES) = 31.00

PEAK FLOW RATE(CFS) AT CONFLUENCE = 52.27

****

FLOW PROCESS FROM NODE 230.00 TO NODE 232.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 300.00

UPSTREAM ELEVATION(FEET) = 100.00

DOWNSTREAM ELEVATION(FEET) = 97.00

ELEVATION DIFFERENCE(FEET) = 3.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.625

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 5.48

TOTAL AREA(ACRES) = 1.18 TOTAL RUNOFF(CFS) = 5.48

****

FLOW PROCESS FROM NODE 232.00 TO NODE 232.00 IS CODE = 81

>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<

==

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.8400

SUBAREA AREA(ACRES) = 0.23 SUBAREA RUNOFF(CFS) = 1.07

TOTAL AREA(ACRES) = 1.4 TOTAL RUNOFF(CFS) = 6.55

TC(MIN.) = 3.63

****

FLOW PROCESS FROM NODE 232.00 TO NODE 220.00 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.) = 3.63

RAINFALL INTENSITY(INCH/HR) = 5.53

TOTAL STREAM AREA(ACRES) = 1.41

PEAK FLOW RATE(CFS) AT CONFLUENCE = 6.55

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	52.27	15.90	2.624	31.00
2	6.55	3.63	5.533	1.41

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	31.34	3.63	5.533
2	55.37	15.90	2.624

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 55.37 Tc(MIN.) = 15.90

TOTAL AREA(ACRES) = 32.4

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 220.00 = 3343.00 FEET.

****

FLOW PROCESS FROM NODE 220.00 TO NODE 262.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 46.00 DOWNSTREAM(FEET) = 38.00

FLOW LENGTH(FEET) = 385.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 30.0 INCH PIPE IS 24.2 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 13.03

ESTIMATED PIPE DIAMETER(INCH) = 30.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 55.37

PIPE TRAVEL TIME(MIN.) = 0.49 Tc(MIN.) = 16.39

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 262.00 = 3728.00 FEET.

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 IS CODE = 10

>>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 1 <<<<<

==

****

FLOW PROCESS FROM NODE 250.00 TO NODE 252.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 200.00

UPSTREAM ELEVATION(FEET) = 100.00

DOWNSTREAM ELEVATION(FEET) = 98.00

ELEVATION DIFFERENCE(FEET) = 2.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.625

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 7.95

TOTAL AREA(ACRES) = 1.71 TOTAL RUNOFF(CFS) = 7.95

****

FLOW PROCESS FROM NODE 251.00 TO NODE 252.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 40.00 DOWNSTREAM(FEET) = 33.00

FLOW LENGTH(FEET) = 710.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 18.0 INCH PIPE IS 12.2 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 6.23

ESTIMATED PIPE DIAMETER(INCH) = 18.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 7.95

PIPE TRAVEL TIME(MIN.) = 1.90 Tc(MIN.) = 5.53

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 252.00 = 910.00 FEET.

****

FLOW PROCESS FROM NODE 251.00 TO NODE 252.00 IS CODE = 81

>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<

==

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.187

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.8400

SUBAREA AREA(ACRES) = 0.85 SUBAREA RUNOFF(CFS) = 3.70

TOTAL AREA(ACRES) = 2.6 TOTAL RUNOFF(CFS) = 11.15

TC(MIN.) = 5.53

****

FLOW PROCESS FROM NODE 252.00 TO NODE 258.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 52.00 DOWNSTREAM(FEET) = 42.00

FLOW LENGTH(FEET) = 275.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 15.0 INCH PIPE IS 11.7 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 10.84

ESTIMATED PIPE DIAMETER(INCH) = 15.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 11.15

PIPE TRAVEL TIME(MIN.) = 0.42 Tc(MIN.) = 5.95

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 258.00 = 1185.00 FEET.

****

FLOW PROCESS FROM NODE 258.00 TO NODE 258.00 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.) = 5.95

RAINFALL INTENSITY(INCH/HR) = 4.95

TOTAL STREAM AREA(ACRES) = 2.56

PEAK FLOW RATE(CFS) AT CONFLUENCE = 11.15

****

FLOW PROCESS FROM NODE 255.00 TO NODE 257.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 281.00

UPSTREAM ELEVATION(FEET) = 100.00

DOWNSTREAM ELEVATION(FEET) = 97.00

ELEVATION DIFFERENCE(FEET) = 3.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.577

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 61.01

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 4.83

TOTAL AREA(ACRES) = 1.04 TOTAL RUNOFF(CFS) = 4.83

****

FLOW PROCESS FROM NODE 257.00 TO NODE 258.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 45.00 DOWNSTREAM(FEET) = 42.00

FLOW LENGTH(FEET) = 225.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 15.0 INCH PIPE IS 9.1 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 6.21

ESTIMATED PIPE DIAMETER(INCH) = 15.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 4.83

PIPE TRAVEL TIME(MIN.) = 0.60 Tc(MIN.) = 4.18

LONGEST FLOWPATH FROM NODE 255.00 TO NODE 258.00 = 506.00 FEET.

****

FLOW PROCESS FROM NODE 257.00 TO NODE 258.00 IS CODE = 81

>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<

==

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .4000

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.7493

SUBAREA AREA(ACRES) = 0.27 SUBAREA RUNOFF(CFS) = 0.60

TOTAL AREA(ACRES) = 1.3 TOTAL RUNOFF(CFS) = 5.43

TC(MIN.) = 4.18

****

FLOW PROCESS FROM NODE 258.00 TO NODE 258.00 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.) = 4.18

RAINFALL INTENSITY(INCH/HR) = 5.53

TOTAL STREAM AREA(ACRES) = 1.31

PEAK FLOW RATE(CFS) AT CONFLUENCE = 5.43

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	11.15	5.95	4.946	2.56
2	5.43	4.18	5.533	1.31

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	13.27	4.18	5.533
2	16.01	5.95	4.946

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 16.01 Tc(MIN.) = 5.95

TOTAL AREA(ACRES) = 3.9

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 258.00 = 1185.00 FEET.

****

FLOW PROCESS FROM NODE 258.00 TO NODE 262.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 42.00 DOWNSTREAM(FEET) = 40.00

FLOW LENGTH(FEET) = 175.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 21.0 INCH PIPE IS 17.1 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 7.62

ESTIMATED PIPE DIAMETER(INCH) = 21.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 16.01

PIPE TRAVEL TIME(MIN.) = 0.38 Tc(MIN.) = 6.33

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 262.00 = 1360.00 FEET.

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 IS CODE = 11

>>>>>CONFLUENCE MEMORY BANK # 1 WITH THE MAIN-STREAM MEMORY<<<<<

==

** MAIN STREAM CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	16.01	6.33	4.751	3.87

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 262.00 = 1360.00 FEET.

** MEMORY BANK # 1 CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	55.37	16.39	2.573	32.41

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 262.00 = 3728.00 FEET.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	37.40	6.33	4.751
2	64.04	16.39	2.573

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 64.04 Tc(MIN.) = 16.39

TOTAL AREA(ACRES) = 36.3

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 IS CODE = 12

>>>>>CLEAR MEMORY BANK # 1 <<<<<

==

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 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.) = 16.39

RAINFALL INTENSITY(INCH/HR) = 2.57

TOTAL STREAM AREA(ACRES) = 36.28

PEAK FLOW RATE(CFS) AT CONFLUENCE = 64.04

****

FLOW PROCESS FROM NODE 260.00 TO NODE 262.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 300.00

UPSTREAM ELEVATION(FEET) = 51.50

DOWNSTREAM ELEVATION(FEET) = 48.50

ELEVATION DIFFERENCE(FEET) = 3.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.625

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 1.30

TOTAL AREA(ACRES) = 0.28 TOTAL RUNOFF(CFS) = 1.30

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 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.) = 3.63

RAINFALL INTENSITY(INCH/HR) = 5.53

TOTAL STREAM AREA(ACRES) = 0.28

PEAK FLOW RATE(CFS) AT CONFLUENCE = 1.30

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	64.04	16.39	2.573	36.28
2	1.30	3.63	5.533	0.28

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	15.47	3.63	5.533
2	64.65	16.39	2.573

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 64.65 Tc(MIN.) = 16.39

TOTAL AREA(ACRES) = 36.6

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 262.00 = 3728.00 FEET.

****

FLOW PROCESS FROM NODE 300.00 TO NODE 301.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 10.00

UPSTREAM ELEVATION(FEET) = 70.00

DOWNSTREAM ELEVATION(FEET) = 60.00

ELEVATION DIFFERENCE(FEET) = 10.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 1.982

WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 0.39

TOTAL AREA(ACRES) = 0.20 TOTAL RUNOFF(CFS) = 0.39

****

FLOW PROCESS FROM NODE 301.00 TO NODE 302.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 3 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 60.00 DOWNSTREAM ELEVATION(FEET) = 49.00

STREET LENGTH(FEET) = 395.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 15.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 10.00

INSIDE STREET CROSSFALL(DECIMAL) = 0.020

OUTSIDE STREET CROSSFALL(DECIMAL) = 0.020

SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF = 2

STREET PARKWAY CROSSFALL(DECIMAL) = 0.020

Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 2.01

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.23

HALFSTREET FLOOD WIDTH(FEET) = 5.12

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.65

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.61

STREET FLOW TRAVEL TIME(MIN.) = 2.49 Tc(MIN.) = 4.47

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.533

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .6600

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.603

SUBAREA AREA(ACRES) = 0.89 SUBAREA RUNOFF(CFS) = 3.25

TOTAL AREA(ACRES) = 1.1 PEAK FLOW RATE(CFS) = 3.64

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.27 HALFSTREET FLOOD WIDTH(FEET) = 7.07

FLOW VELOCITY(FEET/SEC.) = 2.94 DEPTH*VELOCITY(FT*FT/SEC.) = 0.79

LONGEST FLOWPATH FROM NODE 300.00 TO NODE 302.00 = 405.00 FEET.

****

FLOW PROCESS FROM NODE 302.00 TO NODE 309.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 48.00 DOWNSTREAM(FEET) = 45.00

FLOW LENGTH(FEET) = 205.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 12.0 INCH PIPE IS 8.8 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 5.88

ESTIMATED PIPE DIAMETER(INCH) = 12.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 3.64

PIPE TRAVEL TIME(MIN.) = 0.58 Tc(MIN.) = 5.05

LONGEST FLOWPATH FROM NODE 300.00 TO NODE 309.00 = 610.00 FEET.

****

FLOW PROCESS FROM NODE 309.00 TO NODE 310.00 IS CODE = 81

>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<

==

50 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.498

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .7600

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.6551

SUBAREA AREA(ACRES) = 0.54 SUBAREA RUNOFF(CFS) = 2.26

TOTAL AREA(ACRES) = 1.6 TOTAL RUNOFF(CFS) = 5.87

TC(MIN.) = 5.05

==

END OF STUDY SUMMARY:

TOTAL AREA(ACRES) = 1.6 TC(MIN.) = 5.05

PEAK FLOW RATE(CFS) = 5.87

==

==

END OF RATIONAL METHOD ANALYSIS

PROPOSED AES CALCULATIONS
100 YEAR STORM EVENT

****

RATIONAL METHOD HYDROLOGY COMPUTER PROGRAM PACKAGE

Reference: SAN DIEGO COUNTY FLOOD CONTROL DISTRICT

2003,1985,1981 HYDROLOGY MANUAL

Ver. 21.0 Release Date: 06/01/2014 License ID 1355

Analysis prepared by:

Fuscoe Engineering

16795 Von Karman

Suite 100

Irvine, California 92606

************************** DESCRIPTION OF STUDY **************************

* THD SR *

* PROPOSED 100 YEAR *

* *

****

FILE NAME: THDPR100.DAT

TIME/DATE OF STUDY: 00:03 05/24/2020

USER SPECIFIED HYDROLOGY AND HYDRAULIC MODEL INFORMATION:

2003 SAN DIEGO MANUAL CRITERIA

```
 USER SPECIFIED STORM EVENT(YEAR) = 100.00
 6-HOUR DURATION PRECIPITATION (INCHES) =   2.600
 SPECIFIED MINIMUM PIPE SIZE(INCH) =  12.00
 SPECIFIED PERCENT OF GRADIENTS(DECIMAL) TO USE FOR FRICTION SLOPE = 0.90
 SAN DIEGO HYDROLOGY MANUAL "C"-VALUES USED FOR RATIONAL METHOD
 NOTE: USE MODIFIED RATIONAL METHOD PROCEDURES FOR CONFLUENCE ANALYSIS
 *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
 2   25.0     20.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0160
 3   15.0     10.0    0.020/0.020/0.020   0.50    1.50 0.0313 0.125 0.0160

 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 =  0.1 (FT*FT/S)
 *SIZE PIPE WITH A FLOW CAPACITY GREATER THAN
  OR EQUAL TO THE UPSTREAM TRIBUTARY PIPE.*

****************************************************************************
 FLOW PROCESS FROM NODE    100.00 TO NODE    101.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .6800
```

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 77.00

UPSTREAM ELEVATION(FEET) = 360.00

DOWNSTREAM ELEVATION(FEET) = 359.35

ELEVATION DIFFERENCE(FEET) = 0.65

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 6.213

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.32

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.955

SUBAREA RUNOFF(CFS) = 0.65

TOTAL AREA(ACRES) = 0.16 TOTAL RUNOFF(CFS) = 0.65

****

FLOW PROCESS FROM NODE 101.00 TO NODE 102.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 3 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 359.35 DOWNSTREAM ELEVATION(FEET) = 354.00

STREET LENGTH(FEET) = 308.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 15.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 10.00

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) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 2.98

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.32

HALFSTREET FLOOD WIDTH(FEET) = 9.80

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.76

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.89

STREET FLOW TRAVEL TIME(MIN.) = 1.86 Tc(MIN.) = 8.07

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.029

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .7400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.733

SUBAREA AREA(ACRES) = 1.25 SUBAREA RUNOFF(CFS) = 4.65

TOTAL AREA(ACRES) = 1.4 PEAK FLOW RATE(CFS) = 5.20

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.37 HALFSTREET FLOOD WIDTH(FEET) = 12.38

FLOW VELOCITY(FEET/SEC.) = 3.15 DEPTH*VELOCITY(FT*FT/SEC.) = 1.18

LONGEST FLOWPATH FROM NODE 100.00 TO NODE 102.00 = 385.00 FEET.

****

FLOW PROCESS FROM NODE 102.00 TO NODE 103.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 354.00 DOWNSTREAM(FEET) = 70.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 801.00 CHANNEL SLOPE = 0.3546

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.030 MAXIMUM DEPTH(FEET) = 4.00

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.551

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .4100

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 9.84

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 9.87

AVERAGE FLOW DEPTH(FEET) = 0.22 TRAVEL TIME(MIN.) = 1.35

Tc(MIN.) = 9.43

SUBAREA AREA(ACRES) = 4.92 SUBAREA RUNOFF(CFS) = 9.18

AREA-AVERAGE RUNOFF COEFFICIENT = 0.482

TOTAL AREA(ACRES) = 6.3 PEAK FLOW RATE(CFS) = 13.88

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.27 FLOW VELOCITY(FEET/SEC.) = 11.24

LONGEST FLOWPATH FROM NODE 100.00 TO NODE 103.00 = 1186.00 FEET.

****

FLOW PROCESS FROM NODE 103.00 TO NODE 103.00 IS CODE = 10

>>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 1 <<<<<

==

****

FLOW PROCESS FROM NODE 110.00 TO NODE 111.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 121.00

UPSTREAM ELEVATION(FEET) = 345.00

DOWNSTREAM ELEVATION(FEET) = 335.00

ELEVATION DIFFERENCE(FEET) = 10.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 6.678

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 100.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.684

SUBAREA RUNOFF(CFS) = 0.78

TOTAL AREA(ACRES) = 0.39 TOTAL RUNOFF(CFS) = 0.78

****

FLOW PROCESS FROM NODE 111.00 TO NODE 112.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 335.00 DOWNSTREAM(FEET) = 125.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 436.00 CHANNEL SLOPE = 0.4817

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.030 MAXIMUM DEPTH(FEET) = 4.00

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.149

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 2.38

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 6.56

AVERAGE FLOW DEPTH(FEET) = 0.09 TRAVEL TIME(MIN.) = 1.11

Tc(MIN.) = 7.79

SUBAREA AREA(ACRES) = 1.77 SUBAREA RUNOFF(CFS) = 3.19

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 2.2 PEAK FLOW RATE(CFS) = 3.89

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.12 FLOW VELOCITY(FEET/SEC.) = 7.84

LONGEST FLOWPATH FROM NODE 110.00 TO NODE 112.00 = 557.00 FEET.

****

FLOW PROCESS FROM NODE 112.00 TO NODE 117.00 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.) = 7.79

RAINFALL INTENSITY(INCH/HR) = 5.15

TOTAL STREAM AREA(ACRES) = 2.16

PEAK FLOW RATE(CFS) AT CONFLUENCE = 3.89

****

FLOW PROCESS FROM NODE 115.00 TO NODE 116.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 205.00

UPSTREAM ELEVATION(FEET) = 117.00

DOWNSTREAM ELEVATION(FEET) = 113.50

ELEVATION DIFFERENCE(FEET) = 3.50

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 10.141

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 80.61

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 4.341

SUBAREA RUNOFF(CFS) = 0.24

TOTAL AREA(ACRES) = 0.16 TOTAL RUNOFF(CFS) = 0.24

****

FLOW PROCESS FROM NODE 116.00 TO NODE 117.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 113.50 DOWNSTREAM(FEET) = 105.50

CHANNEL LENGTH THRU SUBAREA(FEET) = 588.00 CHANNEL SLOPE = 0.0136

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   4.00

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.344

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) =   0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =   0.57

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   1.94

AVERAGE FLOW DEPTH(FEET) =   0.07   TRAVEL TIME(MIN.) =   5.06

Tc(MIN.) =   15.20

SUBAREA AREA(ACRES) =   0.56   SUBAREA RUNOFF(CFS) =   0.66

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) =   0.7   PEAK FLOW RATE(CFS) =   0.84

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.09   FLOW VELOCITY(FEET/SEC.) =   2.27

LONGEST FLOWPATH FROM NODE   115.00 TO NODE   117.00 =   793.00 FEET.

****

FLOW PROCESS FROM NODE   112.00 TO NODE   117.00 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.) =   15.20

RAINFALL INTENSITY(INCH/HR) =   3.34

TOTAL STREAM AREA(ACRES) =   0.72

PEAK FLOW RATE(CFS) AT CONFLUENCE =   0.84

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	3.89	7.79	5.149	2.16
2	0.84	15.20	3.344	0.72

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	4.32	7.79	5.149
2	3.37	15.20	3.344

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 4.32 Tc(MIN.) = 7.79

TOTAL AREA(ACRES) = 2.9

LONGEST FLOWPATH FROM NODE 115.00 TO NODE 117.00 = 793.00 FEET.

****

FLOW PROCESS FROM NODE 117.00 TO NODE 122.00 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.) = 7.79

RAINFALL INTENSITY(INCH/HR) = 5.15

TOTAL STREAM AREA(ACRES) = 2.88

PEAK FLOW RATE(CFS) AT CONFLUENCE = 4.32

****

FLOW PROCESS FROM NODE 120.00 TO NODE 121.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 99.00

UPSTREAM ELEVATION(FEET) = 80.50

DOWNSTREAM ELEVATION(FEET) = 80.10

ELEVATION DIFFERENCE(FEET) = 0.40

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 12.027

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) = 3.889

SUBAREA RUNOFF(CFS) = 0.15

TOTAL AREA(ACRES) = 0.11 TOTAL RUNOFF(CFS) = 0.15

****

FLOW PROCESS FROM NODE 121.00 TO NODE 122.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 80.10 DOWNSTREAM(FEET) = 73.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 364.00 CHANNEL SLOPE = 0.0195

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.015 MAXIMUM DEPTH(FEET) = 4.00

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.335

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 0.39

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 1.87

AVERAGE FLOW DEPTH(FEET) = 0.05 TRAVEL TIME(MIN.) = 3.24

Tc(MIN.) = 15.26

SUBAREA AREA(ACRES) = 0.41 SUBAREA RUNOFF(CFS) = 0.48

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 0.5 PEAK FLOW RATE(CFS) = 0.61

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.07 FLOW VELOCITY(FEET/SEC.) = 2.23

LONGEST FLOWPATH FROM NODE 120.00 TO NODE 122.00 = 463.00 FEET.

****

FLOW PROCESS FROM NODE 122.00 TO NODE 122.00 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.) = 15.26

RAINFALL INTENSITY(INCH/HR) = 3.34

TOTAL STREAM AREA(ACRES) = 0.52

PEAK FLOW RATE(CFS) AT CONFLUENCE = 0.61

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	4.32	7.79	5.149	2.88
2	0.61	15.26	3.335	0.52

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	4.63	7.79	5.149
2	3.41	15.26	3.335

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 4.63 Tc(MIN.) = 7.79

TOTAL AREA(ACRES) = 3.4

LONGEST FLOWPATH FROM NODE 115.00 TO NODE 122.00 = 793.00 FEET.

****

FLOW PROCESS FROM NODE 122.00 TO NODE 103.00 IS CODE = 11

>>>>>CONFLUENCE MEMORY BANK # 1 WITH THE MAIN-STREAM MEMORY<<<<<

==

** MAIN STREAM CONFLUENCE DATA **

STREAM RUNOFF Tc INTENSITY AREA

NUMBER (CFS) (MIN.) (INCH/HOUR) (ACRE)

1 4.63 7.79 5.149 3.40

LONGEST FLOWPATH FROM NODE 115.00 TO NODE 103.00 = 793.00 FEET.

** MEMORY BANK # 1 CONFLUENCE DATA **

STREAM RUNOFF Tc INTENSITY AREA

NUMBER (CFS) (MIN.) (INCH/HOUR) (ACRE)

1 13.88 9.43 4.551 6.33

LONGEST FLOWPATH FROM NODE 100.00 TO NODE 103.00 = 1186.00 FEET.

** PEAK FLOW RATE TABLE **

STREAM RUNOFF Tc INTENSITY

NUMBER (CFS) (MIN.) (INCH/HOUR)

1 16.10 7.79 5.149

2 17.98 9.43 4.551

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 17.98 Tc(MIN.) = 9.43

TOTAL AREA(ACRES) = 9.7

****

FLOW PROCESS FROM NODE 122.00 TO NODE 103.00 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.) = 9.43

RAINFALL INTENSITY(INCH/HR) = 4.55

TOTAL STREAM AREA(ACRES) = 9.73

PEAK FLOW RATE(CFS) AT CONFLUENCE = 17.98

****

FLOW PROCESS FROM NODE 130.00 TO NODE 131.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 99.00

UPSTREAM ELEVATION(FEET) = 115.00

DOWNSTREAM ELEVATION(FEET) = 85.00

ELEVATION DIFFERENCE(FEET) = 30.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 6.235

WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.941

SUBAREA RUNOFF(CFS) = 0.27

TOTAL AREA(ACRES) = 0.13 TOTAL RUNOFF(CFS) = 0.27

****

FLOW PROCESS FROM NODE 131.00 TO NODE 132.00 IS CODE = 51

```
 >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
==========================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    85.00  DOWNSTREAM(FEET) =    80.50
 CHANNEL LENGTH THRU SUBAREA(FEET) =   271.00   CHANNEL SLOPE =  0.0166
 CHANNEL BASE(FEET) =    4.00   "Z" FACTOR =   2.000
 MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   4.00
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.881
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .3500
 S.C.S. CURVE NUMBER (AMC II) =   0
 TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =      0.55
 TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   2.03
 AVERAGE FLOW DEPTH(FEET) =   0.07   TRAVEL TIME(MIN.) =   2.22
 Tc(MIN.) =    8.46
 SUBAREA AREA(ACRES) =     0.33      SUBAREA RUNOFF(CFS) =    0.56
 AREA-AVERAGE RUNOFF COEFFICIENT =  0.350
 TOTAL AREA(ACRES) =       0.5        PEAK FLOW RATE(CFS) =      0.79

 END OF SUBAREA CHANNEL FLOW HYDRAULICS:
 DEPTH(FEET) =  0.08   FLOW VELOCITY(FEET/SEC.) =   2.33
 LONGEST FLOWPATH FROM NODE    130.00 TO NODE    132.00 =     370.00 FEET.

**************************************************************************
 FLOW PROCESS FROM NODE    132.00 TO NODE    135.00 IS CODE =  31
--------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
==========================================================================
```

```
 ELEVATION DATA: UPSTREAM(FEET) =    80.50  DOWNSTREAM(FEET) =    60.00
 FLOW LENGTH(FEET) =    60.00   MANNING'S N =  0.013
 ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000
 DEPTH OF FLOW IN  12.0 INCH PIPE IS   1.6 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  12.23
 ESTIMATED PIPE DIAMETER(INCH) =  12.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =       0.79
 PIPE TRAVEL TIME(MIN.) =   0.08    Tc(MIN.) =    8.54
 LONGEST FLOWPATH FROM NODE    130.00 TO NODE    135.00 =     430.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    133.00 TO NODE    135.00 IS CODE =  51
----------------------------------------------------------------------------
 >>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<
 >>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =     90.00  DOWNSTREAM(FEET) =     60.00
 CHANNEL LENGTH THRU SUBAREA(FEET) =   332.00   CHANNEL SLOPE =  0.0904
 CHANNEL BASE(FEET) =    4.00   "Z" FACTOR =   2.000
 MANNING'S FACTOR = 0.015   MAXIMUM DEPTH(FEET) =   4.00
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  4.432
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .3500
 S.C.S. CURVE NUMBER (AMC II) =   0
 TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) =       1.14
 TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) =   4.31
 AVERAGE FLOW DEPTH(FEET) =   0.06   TRAVEL TIME(MIN.) =   1.28
 Tc(MIN.) =    9.82
 SUBAREA AREA(ACRES) =     0.45      SUBAREA RUNOFF(CFS) =    0.70
```

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 0.9 PEAK FLOW RATE(CFS) = 1.41

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.07 FLOW VELOCITY(FEET/SEC.) = 4.76

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 135.00 = 762.00 FEET.

****

FLOW PROCESS FROM NODE 135.00 TO NODE 136.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 60.00 DOWNSTREAM(FEET) = 55.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 476.00 CHANNEL SLOPE = 0.0105

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.015 MAXIMUM DEPTH(FEET) = 4.00

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.763

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 1.80

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 2.80

AVERAGE FLOW DEPTH(FEET) = 0.15 TRAVEL TIME(MIN.) = 2.84

Tc(MIN.) = 12.66

SUBAREA AREA(ACRES) = 0.59 SUBAREA RUNOFF(CFS) = 0.78

AREA-AVERAGE RUNOFF COEFFICIENT = 0.350

TOTAL AREA(ACRES) = 1.5 PEAK FLOW RATE(CFS) = 1.98

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.16 FLOW VELOCITY(FEET/SEC.) = 2.80

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 136.00 = 1238.00 FEET.

****

FLOW PROCESS FROM NODE 136.00 TO NODE 103.00 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.) = 12.66

RAINFALL INTENSITY(INCH/HR) = 3.76

TOTAL STREAM AREA(ACRES) = 1.50

PEAK FLOW RATE(CFS) AT CONFLUENCE = 1.98

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	17.98	9.43	4.551	9.73
2	1.98	12.66	3.763	1.50

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 19.45 9.43 4.551

2 16.84 12.66 3.763

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 19.45 Tc(MIN.) = 9.43

TOTAL AREA(ACRES) = 11.2

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 103.00 = 1238.00 FEET.

****

FLOW PROCESS FROM NODE 103.00 TO NODE 144.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 70.50 DOWNSTREAM(FEET) = 41.50

FLOW LENGTH(FEET) = 450.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 18.0 INCH PIPE IS 11.8 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 15.79

ESTIMATED PIPE DIAMETER(INCH) = 18.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 19.45

PIPE TRAVEL TIME(MIN.) = 0.48 Tc(MIN.) = 9.90

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 144.00 = 1688.00 FEET.

****

FLOW PROCESS FROM NODE 103.00 TO NODE 144.00 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.) = 9.90

RAINFALL INTENSITY(INCH/HR) = 4.41

TOTAL STREAM AREA(ACRES) = 11.23

PEAK FLOW RATE(CFS) AT CONFLUENCE = 19.45

****

FLOW PROCESS FROM NODE 137.00 TO NODE 138.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 105.00

UPSTREAM ELEVATION(FEET) = 56.00

DOWNSTREAM ELEVATION(FEET) = 50.50

ELEVATION DIFFERENCE(FEET) = 5.50

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 2.563

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 90.48

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 0.81

TOTAL AREA(ACRES) = 0.14 TOTAL RUNOFF(CFS) = 0.81

****

FLOW PROCESS FROM NODE 138.00 TO NODE 139.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 50.50 DOWNSTREAM(FEET) = 49.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 243.00 CHANNEL SLOPE = 0.0062

CHANNEL BASE(FEET) = 1.50 "Z" FACTOR = 0.500

MANNING'S FACTOR = 0.015 MAXIMUM DEPTH(FEET) = 0.50

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 2.47

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 3.42

AVERAGE FLOW DEPTH(FEET) = 0.42 TRAVEL TIME(MIN.) = 1.18

Tc(MIN.) = 3.75

SUBAREA AREA(ACRES) = 0.58 SUBAREA RUNOFF(CFS) = 3.34

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

TOTAL AREA(ACRES) = 0.7 PEAK FLOW RATE(CFS) = 4.14

==>>WARNING: FLOW IN CHANNEL EXCEEDS CHANNEL

CAPACITY(NORMAL DEPTH EQUAL TO SPECIFIED MAXIMUM

ALLOWABLE DEPTH).

AS AN APPROXIMATION, FLOWDEPTH IS SET AT MAXIMUM

ALLOWABLE DEPTH AND IS USED FOR TRAVELTIME CALCULATIONS.

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.50 FLOW VELOCITY(FEET/SEC.) = 4.73

==>FLOWDEPTH EXCEEDS MAXIMUM ALLOWABLE DEPTH

LONGEST FLOWPATH FROM NODE 137.00 TO NODE 139.00 = 348.00 FEET.

****

FLOW PROCESS FROM NODE 139.00 TO NODE 144.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 49.00 DOWNSTREAM(FEET) = 39.00

FLOW LENGTH(FEET) = 20.00 MANNING'S N = 0.013

ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000

DEPTH OF FLOW IN 12.0 INCH PIPE IS 3.4 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 22.82

ESTIMATED PIPE DIAMETER(INCH) = 12.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 4.14

PIPE TRAVEL TIME(MIN.) = 0.01 Tc(MIN.) = 3.76

LONGEST FLOWPATH FROM NODE 137.00 TO NODE 144.00 = 368.00 FEET.

****

FLOW PROCESS FROM NODE 139.00 TO NODE 144.00 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.) = 3.76

RAINFALL INTENSITY(INCH/HR) = 6.85

TOTAL STREAM AREA(ACRES) = 0.72

PEAK FLOW RATE(CFS) AT CONFLUENCE = 4.14

****

FLOW PROCESS FROM NODE 140.00 TO NODE 141.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 112.00

UPSTREAM ELEVATION(FEET) = 54.00

DOWNSTREAM ELEVATION(FEET) = 50.00

ELEVATION DIFFERENCE(FEET) = 4.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 2.847

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 86.43

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 3.40

TOTAL AREA(ACRES) = 0.59 TOTAL RUNOFF(CFS) = 3.40

****

FLOW PROCESS FROM NODE 141.00 TO NODE 142.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 50.00 DOWNSTREAM(FEET) = 45.00

CHANNEL LENGTH THRU SUBAREA(FEET) = 417.00 CHANNEL SLOPE = 0.0120

CHANNEL BASE(FEET) = 3.00 "Z" FACTOR = 1.000

MANNING'S FACTOR = 0.030 MAXIMUM DEPTH(FEET) = 1.00

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.797

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 6.45

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 3.14

AVERAGE FLOW DEPTH(FEET) = 0.58 TRAVEL TIME(MIN.) = 2.21

Tc(MIN.) = 5.06

SUBAREA AREA(ACRES) = 1.07 SUBAREA RUNOFF(CFS) = 6.11

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

TOTAL AREA(ACRES) = 1.7 PEAK FLOW RATE(CFS) = 9.48

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.72 FLOW VELOCITY(FEET/SEC.) = 3.54

LONGEST FLOWPATH FROM NODE 140.00 TO NODE 142.00 = 529.00 FEET.

****

FLOW PROCESS FROM NODE 142.00 TO NODE 144.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 45.00 DOWNSTREAM(FEET) = 39.00

FLOW LENGTH(FEET) = 170.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 15.0 INCH PIPE IS 10.4 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 10.48

ESTIMATED PIPE DIAMETER(INCH) = 15.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 9.48

PIPE TRAVEL TIME(MIN.) = 0.27 Tc(MIN.) = 5.33

LONGEST FLOWPATH FROM NODE 140.00 TO NODE 144.00 = 699.00 FEET.

****

FLOW PROCESS FROM NODE 142.00 TO NODE 144.00 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.) = 5.33

RAINFALL INTENSITY(INCH/HR) = 6.57

TOTAL STREAM AREA(ACRES) = 1.66

PEAK FLOW RATE(CFS) AT CONFLUENCE = 9.48

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	19.45	9.90	4.409	11.23
2	4.14	3.76	6.850	0.72

3 9.48 5.33 6.573 1.66

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 3 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	23.35	3.76	6.850
2	26.50	5.33	6.573
3	28.48	9.90	4.409

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 28.48 Tc(MIN.) = 9.90

TOTAL AREA(ACRES) = 13.6

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 144.00 = 1688.00 FEET.

****

FLOW PROCESS FROM NODE 144.00 TO NODE 145.00 IS CODE = 10

>>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 2 <<<<<

==

****

FLOW PROCESS FROM NODE 150.00 TO NODE 151.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 167.00

UPSTREAM ELEVATION(FEET) = 52.50

DOWNSTREAM ELEVATION(FEET) = 50.00

ELEVATION DIFFERENCE(FEET) = 2.50

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.360

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 67.46

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 2.76

TOTAL AREA(ACRES) = 0.48 TOTAL RUNOFF(CFS) = 2.76

****

FLOW PROCESS FROM NODE 151.00 TO NODE 152.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 2 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 50.00 DOWNSTREAM ELEVATION(FEET) = 47.30

STREET LENGTH(FEET) = 160.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 25.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 20.00

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) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 4.29

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.36

HALFSTREET FLOOD WIDTH(FEET) = 11.52

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.96

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 1.06

STREET FLOW TRAVEL TIME(MIN.) = 0.90 Tc(MIN.) = 4.26

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

SUBAREA AREA(ACRES) = 0.53 SUBAREA RUNOFF(CFS) = 3.05

TOTAL AREA(ACRES) = 1.0 PEAK FLOW RATE(CFS) = 5.81

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.39 HALFSTREET FLOOD WIDTH(FEET) = 13.09

FLOW VELOCITY(FEET/SEC.) = 3.17 DEPTH*VELOCITY(FT*FT/SEC.) = 1.23

LONGEST FLOWPATH FROM NODE 150.00 TO NODE 152.00 = 327.00 FEET.

****

FLOW PROCESS FROM NODE 152.00 TO NODE 153.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 47.30 DOWNSTREAM(FEET) = 46.00

FLOW LENGTH(FEET) = 75.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 15.0 INCH PIPE IS 9.4 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 7.15

ESTIMATED PIPE DIAMETER(INCH) = 15.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 5.81

PIPE TRAVEL TIME(MIN.) = 0.17 Tc(MIN.) = 4.43

LONGEST FLOWPATH FROM NODE 150.00 TO NODE 153.00 = 402.00 FEET.

****

FLOW PROCESS FROM NODE 152.00 TO NODE 153.00 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.) = 4.43

RAINFALL INTENSITY(INCH/HR) = 6.85

TOTAL STREAM AREA(ACRES) = 1.01

PEAK FLOW RATE(CFS) AT CONFLUENCE = 5.81

****

FLOW PROCESS FROM NODE 16.00 TO NODE 161.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

===

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 93.00

UPSTREAM ELEVATION(FEET) = 55.30

DOWNSTREAM ELEVATION(FEET) = 54.70

ELEVATION DIFFERENCE(FEET) = 0.60

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.939

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 52.90

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 0.63

TOTAL AREA(ACRES) = 0.11 TOTAL RUNOFF(CFS) = 0.63

****

FLOW PROCESS FROM NODE 161.00 TO NODE 162.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 3 USED)<<<<<

===

UPSTREAM ELEVATION(FEET) = 54.70 DOWNSTREAM ELEVATION(FEET) = 44.00

STREET LENGTH(FEET) = 340.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 15.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 10.00

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) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 1.41

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.25

HALFSTREET FLOOD WIDTH(FEET) = 5.98

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.97

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.73

STREET FLOW TRAVEL TIME(MIN.) = 1.91 Tc(MIN.) = 5.85

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.194

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

SUBAREA AREA(ACRES) = 0.30 SUBAREA RUNOFF(CFS) = 1.56

TOTAL AREA(ACRES) = 0.4 PEAK FLOW RATE(CFS) = 2.13

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.27 HALFSTREET FLOOD WIDTH(FEET) = 7.38

FLOW VELOCITY(FEET/SEC.) = 3.22 DEPTH*VELOCITY(FT*FT/SEC.) = 0.88

LONGEST FLOWPATH FROM NODE 16.00 TO NODE 162.00 = 433.00 FEET.

****

FLOW PROCESS FROM NODE 162.00 TO NODE 153.00 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.) = 5.85

RAINFALL INTENSITY(INCH/HR) = 6.19

TOTAL STREAM AREA(ACRES) = 0.41

PEAK FLOW RATE(CFS) AT CONFLUENCE = 2.13

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	5.81	4.43	6.850	1.01
2	2.13	5.85	6.194	0.41

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	7.43	4.43	6.850
2	7.39	5.85	6.194

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 7.43 Tc(MIN.) = 4.43

TOTAL AREA(ACRES) = 1.4

LONGEST FLOWPATH FROM NODE 16.00 TO NODE 153.00 = 433.00 FEET.

****

FLOW PROCESS FROM NODE 155.00 TO NODE 153.00 IS CODE = 81

>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<

==

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.8400

SUBAREA AREA(ACRES) = 0.41 SUBAREA RUNOFF(CFS) = 2.36

TOTAL AREA(ACRES) = 1.8 TOTAL RUNOFF(CFS) = 10.53

TC(MIN.) = 4.43

****

FLOW PROCESS FROM NODE 153.00 TO NODE 145.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 2 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 43.60 DOWNSTREAM ELEVATION(FEET) = 42.50

STREET LENGTH(FEET) = 275.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 25.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 20.00

INSIDE STREET CROSSFALL(DECIMAL) = 0.020

OUTSIDE STREET CROSSFALL(DECIMAL) = 0.020

SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF = 2

STREET PARKWAY CROSSFALL(DECIMAL) = 0.020

Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 11.30

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.47

HALFSTREET FLOOD WIDTH(FEET) = 17.23

AVERAGE FLOW VELOCITY(FEET/SEC.) = 1.83

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.86

STREET FLOW TRAVEL TIME(MIN.) = 2.50 Tc(MIN.) = 6.94

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.546

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.840

SUBAREA AREA(ACRES) = 0.33 SUBAREA RUNOFF(CFS) = 1.54

TOTAL AREA(ACRES) = 2.2 PEAK FLOW RATE(CFS) = 10.53

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.46 HALFSTREET FLOOD WIDTH(FEET) = 16.76

FLOW VELOCITY(FEET/SEC.) = 1.80 DEPTH*VELOCITY(FT*FT/SEC.) = 0.83

LONGEST FLOWPATH FROM NODE 16.00 TO NODE 145.00 = 708.00 FEET.

****

```
 FLOW PROCESS FROM NODE    153.00 TO NODE    145.00 IS CODE =  11
----------------------------------------------------------------------------
 >>>>>CONFLUENCE MEMORY BANK # 2 WITH THE MAIN-STREAM MEMORY<<<<<
============================================================================

 ** MAIN STREAM CONFLUENCE DATA **
 STREAM    RUNOFF     Tc     INTENSITY    AREA
 NUMBER     (CFS)    (MIN.)   (INCH/HOUR)   (ACRE)
    1      10.53    6.94      5.546       2.16
 LONGEST FLOWPATH FROM NODE     16.00 TO NODE    145.00 =     708.00 FEET.

 ** MEMORY BANK #  2 CONFLUENCE DATA **
 STREAM    RUNOFF     Tc     INTENSITY    AREA
 NUMBER     (CFS)    (MIN.)   (INCH/HOUR)   (ACRE)
    1      28.48    9.90      4.409      13.61
 LONGEST FLOWPATH FROM NODE    130.00 TO NODE    145.00 =    1688.00 FEET.

 ** PEAK FLOW RATE TABLE **
 STREAM    RUNOFF      Tc     INTENSITY
 NUMBER     (CFS)    (MIN.)   (INCH/HOUR)
    1     30.48      6.94       5.546
    2     36.85      9.90       4.409

 COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:
 PEAK FLOW RATE(CFS) =      36.85   Tc(MIN.) =    9.90
 TOTAL AREA(ACRES) =       15.8

****************************************************************************
 FLOW PROCESS FROM NODE    145.00 TO NODE    145.00 IS CODE =  12
```

>>>>>CLEAR MEMORY BANK # 1 <<<<<

==

****

FLOW PROCESS FROM NODE 145.00 TO NODE 145.00 IS CODE = 12

>>>>>CLEAR MEMORY BANK # 2 <<<<<

==

****

FLOW PROCESS FROM NODE 218.00 TO NODE 218.00 IS CODE = 7

>>>>>USER SPECIFIED HYDROLOGY INFORMATION AT NODE<<<<<

==

USER-SPECIFIED VALUES ARE AS FOLLOWS:

TC(MIN) = 14.08 RAIN INTENSITY(INCH/HOUR) = 3.51

TOTAL AREA(ACRES) = 20.62 TOTAL RUNOFF(CFS) = 42.37

****

FLOW PROCESS FROM NODE 218.00 TO NODE 219.00 IS CODE = 51

>>>>>COMPUTE TRAPEZOIDAL CHANNEL FLOW<<<<<

>>>>>TRAVELTIME THRU SUBAREA (EXISTING ELEMENT)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 189.00 DOWNSTREAM(FEET) = 112.50

CHANNEL LENGTH THRU SUBAREA(FEET) = 705.00 CHANNEL SLOPE = 0.1085

CHANNEL BASE(FEET) = 4.00 "Z" FACTOR = 2.000

MANNING'S FACTOR = 0.030 MAXIMUM DEPTH(FEET) = 4.00

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 3.355

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3800

S.C.S. CURVE NUMBER (AMC II) = 0

TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 48.31

TRAVEL TIME THRU SUBAREA BASED ON VELOCITY(FEET/SEC.) = 11.26

AVERAGE FLOW DEPTH(FEET) = 0.77 TRAVEL TIME(MIN.) = 1.04

Tc(MIN.) = 15.12

SUBAREA AREA(ACRES) = 9.31 SUBAREA RUNOFF(CFS) = 11.87

AREA-AVERAGE RUNOFF COEFFICIENT = 0.521

TOTAL AREA(ACRES) = 29.9 PEAK FLOW RATE(CFS) = 52.33

END OF SUBAREA CHANNEL FLOW HYDRAULICS:

DEPTH(FEET) = 0.81 FLOW VELOCITY(FEET/SEC.) = 11.57

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 219.00 = 2393.00 FEET.

****

FLOW PROCESS FROM NODE 219.00 TO NODE 219.00 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.) = 15.12

RAINFALL INTENSITY(INCH/HR) = 3.35

TOTAL STREAM AREA(ACRES) = 29.93

PEAK FLOW RATE(CFS) AT CONFLUENCE = 52.33

****

FLOW PROCESS FROM NODE 225.00 TO NODE 226.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .7100

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 65.00

UPSTREAM ELEVATION(FEET) = 358.00

DOWNSTREAM ELEVATION(FEET) = 357.35

ELEVATION DIFFERENCE(FEET) = 0.65

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 5.438

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.489

SUBAREA RUNOFF(CFS) = 0.51

TOTAL AREA(ACRES) = 0.11 TOTAL RUNOFF(CFS) = 0.51

****

FLOW PROCESS FROM NODE 226.00 TO NODE 227.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 3 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 357.35 DOWNSTREAM ELEVATION(FEET) = 346.00

STREET LENGTH(FEET) = 350.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 15.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 10.00

INSIDE STREET CROSSFALL(DECIMAL) = 0.020

OUTSIDE STREET CROSSFALL(DECIMAL) = 0.020

SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF = 2

STREET PARKWAY CROSSFALL(DECIMAL) = 0.020

Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 2.40

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.23

HALFSTREET FLOOD WIDTH(FEET) = 5.43

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.90

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.68

STREET FLOW TRAVEL TIME(MIN.) = 2.01 Tc(MIN.) = 7.45

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 5.298

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .7400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.737

SUBAREA AREA(ACRES) = 0.96 SUBAREA RUNOFF(CFS) = 3.76

TOTAL AREA(ACRES) = 1.1 PEAK FLOW RATE(CFS) = 4.18

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.27 HALFSTREET FLOOD WIDTH(FEET) = 7.23

FLOW VELOCITY(FEET/SEC.) = 3.26 DEPTH*VELOCITY(FT*FT/SEC.) = 0.88

LONGEST FLOWPATH FROM NODE 225.00 TO NODE 227.00 = 415.00 FEET.

****

FLOW PROCESS FROM NODE 227.00 TO NODE 219.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 346.00 DOWNSTREAM(FEET) = 112.50

FLOW LENGTH(FEET) = 575.00 MANNING'S N = 0.013

ESTIMATED PIPE DIAMETER(INCH) INCREASED TO 12.000

DEPTH OF FLOW IN 12.0 INCH PIPE IS 3.6 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 21.23

ESTIMATED PIPE DIAMETER(INCH) = 12.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 4.18

PIPE TRAVEL TIME(MIN.) = 0.45 Tc(MIN.) = 7.90

LONGEST FLOWPATH FROM NODE 225.00 TO NODE 219.00 = 990.00 FEET.

****

FLOW PROCESS FROM NODE 219.00 TO NODE 219.00 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.) = 7.90

RAINFALL INTENSITY(INCH/HR) = 5.10

TOTAL STREAM AREA(ACRES) = 1.07

PEAK FLOW RATE(CFS) AT CONFLUENCE = 4.18

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	52.33	15.12	3.355	29.93
2	4.18	7.90	5.100	1.07

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	31.51	7.90	5.100
2	55.08	15.12	3.355

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 55.08 Tc(MIN.) = 15.12

TOTAL AREA(ACRES) = 31.0

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 219.00 = 2393.00 FEET.

****

FLOW PROCESS FROM NODE 219.00 TO NODE 220.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 112.50 DOWNSTREAM(FEET) = 46.00

FLOW LENGTH(FEET) = 950.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 24.0 INCH PIPE IS 19.1 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 20.59

ESTIMATED PIPE DIAMETER(INCH) = 24.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 55.08

PIPE TRAVEL TIME(MIN.) = 0.77 Tc(MIN.) = 15.89

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 220.00 = 3343.00 FEET.

****

FLOW PROCESS FROM NODE 219.00 TO NODE 220.00 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.) = 15.89

RAINFALL INTENSITY(INCH/HR) = 3.25

TOTAL STREAM AREA(ACRES) = 31.00

PEAK FLOW RATE(CFS) AT CONFLUENCE = 55.08

****

FLOW PROCESS FROM NODE 230.00 TO NODE 232.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 300.00

UPSTREAM ELEVATION(FEET) = 100.00

```
 DOWNSTREAM ELEVATION(FEET) =     97.00
 ELEVATION DIFFERENCE(FEET) =      3.00
 SUBAREA OVERLAND TIME OF FLOW(MIN.) =    3.625
 WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN
         THE MAXIMUM OVERLAND FLOW LENGTH =    60.00
         (Reference: Table 3-1B of Hydrology Manual)
         THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.850
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 SUBAREA RUNOFF(CFS) =      6.79
 TOTAL AREA(ACRES) =      1.18   TOTAL RUNOFF(CFS) =      6.79

****************************************************************************
 FLOW PROCESS FROM NODE    232.00 TO NODE    232.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.850
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .8400
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT = 0.8400
 SUBAREA AREA(ACRES) =    0.23   SUBAREA RUNOFF(CFS) =    1.32
 TOTAL AREA(ACRES) =       1.4   TOTAL RUNOFF(CFS) =      8.11
 TC(MIN.) =    3.63

****************************************************************************
 FLOW PROCESS FROM NODE    232.00 TO NODE    220.00 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.) = 3.63

RAINFALL INTENSITY(INCH/HR) = 6.85

TOTAL STREAM AREA(ACRES) = 1.41

PEAK FLOW RATE(CFS) AT CONFLUENCE = 8.11

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	55.08	15.89	3.249	31.00
2	8.11	3.63	6.850	1.41

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	34.24	3.63	6.850
2	58.93	15.89	3.249

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 58.93 Tc(MIN.) = 15.89

TOTAL AREA(ACRES) = 32.4

```
 LONGEST FLOWPATH FROM NODE    130.00 TO NODE    220.00 =    3343.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    220.00 TO NODE    262.00 IS CODE =  31
----------------------------------------------------------------------------
 >>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<
 >>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<
============================================================================
 ELEVATION DATA: UPSTREAM(FEET) =    46.00  DOWNSTREAM(FEET) =    38.00
 FLOW LENGTH(FEET) =   385.00   MANNING'S N =  0.013
 DEPTH OF FLOW IN  33.0 INCH PIPE IS  22.6 INCHES
 PIPE-FLOW VELOCITY(FEET/SEC.) =  13.57
 ESTIMATED PIPE DIAMETER(INCH) =  33.00    NUMBER OF PIPES =   1
 PIPE-FLOW(CFS) =      58.93
 PIPE TRAVEL TIME(MIN.) =   0.47    Tc(MIN.) =   16.37
 LONGEST FLOWPATH FROM NODE    130.00 TO NODE    262.00 =    3728.00 FEET.

****************************************************************************
 FLOW PROCESS FROM NODE    262.00 TO NODE    262.00 IS CODE =  10
----------------------------------------------------------------------------
 >>>>>MAIN-STREAM MEMORY COPIED ONTO MEMORY BANK # 1 <<<<<
============================================================================

****************************************************************************
 FLOW PROCESS FROM NODE    250.00 TO NODE    252.00 IS CODE =  21
----------------------------------------------------------------------------
 >>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<
============================================================================
 *USER SPECIFIED(SUBAREA):
```

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 200.00

UPSTREAM ELEVATION(FEET) = 100.00

DOWNSTREAM ELEVATION(FEET) = 98.00

ELEVATION DIFFERENCE(FEET) = 2.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.625

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 9.84

TOTAL AREA(ACRES) = 1.71 TOTAL RUNOFF(CFS) = 9.84

****

FLOW PROCESS FROM NODE 251.00 TO NODE 252.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 40.00 DOWNSTREAM(FEET) = 33.00

FLOW LENGTH(FEET) = 710.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 18.0 INCH PIPE IS 14.7 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 6.38

ESTIMATED PIPE DIAMETER(INCH) = 18.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 9.84

PIPE TRAVEL TIME(MIN.) = 1.85 Tc(MIN.) = 5.48

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 252.00 = 910.00 FEET.

****

FLOW PROCESS FROM NODE 251.00 TO NODE 252.00 IS CODE = 81

>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<

==

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.458

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.8400

SUBAREA AREA(ACRES) = 0.85 SUBAREA RUNOFF(CFS) = 4.61

TOTAL AREA(ACRES) = 2.6 TOTAL RUNOFF(CFS) = 13.89

TC(MIN.) = 5.48

****

FLOW PROCESS FROM NODE 252.00 TO NODE 258.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 52.00 DOWNSTREAM(FEET) = 42.00

FLOW LENGTH(FEET) = 275.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 18.0 INCH PIPE IS 11.4 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 11.74

ESTIMATED PIPE DIAMETER(INCH) = 18.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 13.89

PIPE TRAVEL TIME(MIN.) = 0.39 Tc(MIN.) = 5.87

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 258.00 = 1185.00 FEET.

****

FLOW PROCESS FROM NODE 258.00 TO NODE 258.00 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.) = 5.87

RAINFALL INTENSITY(INCH/HR) = 6.18

TOTAL STREAM AREA(ACRES) = 2.56

PEAK FLOW RATE(CFS) AT CONFLUENCE = 13.89

****

FLOW PROCESS FROM NODE 255.00 TO NODE 257.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 281.00

UPSTREAM ELEVATION(FEET) = 100.00

DOWNSTREAM ELEVATION(FEET) = 97.00

ELEVATION DIFFERENCE(FEET) = 3.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.577

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 61.01

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 5.98

TOTAL AREA(ACRES) = 1.04 TOTAL RUNOFF(CFS) = 5.98

****

FLOW PROCESS FROM NODE 257.00 TO NODE 258.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 45.00 DOWNSTREAM(FEET) = 42.00

FLOW LENGTH(FEET) = 225.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 15.0 INCH PIPE IS 10.6 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 6.47

ESTIMATED PIPE DIAMETER(INCH) = 15.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 5.98

PIPE TRAVEL TIME(MIN.) = 0.58 Tc(MIN.) = 4.16

LONGEST FLOWPATH FROM NODE 255.00 TO NODE 258.00 = 506.00 FEET.

****

FLOW PROCESS FROM NODE 257.00 TO NODE 258.00 IS CODE = 81

>>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<

==

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .4000

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.7493

SUBAREA AREA(ACRES) = 0.27 SUBAREA RUNOFF(CFS) = 0.74

TOTAL AREA(ACRES) = 1.3 TOTAL RUNOFF(CFS) = 6.72

TC(MIN.) = 4.16

****

FLOW PROCESS FROM NODE 258.00 TO NODE 258.00 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.) = 4.16

RAINFALL INTENSITY(INCH/HR) = 6.85

TOTAL STREAM AREA(ACRES) = 1.31

PEAK FLOW RATE(CFS) AT CONFLUENCE = 6.72

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	13.89	5.87	6.178	2.56
2	6.72	4.16	6.850	1.31

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	16.56	4.16	6.850
2	19.95	5.87	6.178

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 19.95 Tc(MIN.) = 5.87

TOTAL AREA(ACRES) = 3.9

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 258.00 = 1185.00 FEET.

****

FLOW PROCESS FROM NODE 258.00 TO NODE 262.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 42.00 DOWNSTREAM(FEET) = 40.00

FLOW LENGTH(FEET) = 175.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 24.0 INCH PIPE IS 17.3 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 8.23

ESTIMATED PIPE DIAMETER(INCH) = 24.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 19.95

PIPE TRAVEL TIME(MIN.) = 0.35 Tc(MIN.) = 6.22

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 262.00 = 1360.00 FEET.

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 IS CODE = 11

>>>>>CONFLUENCE MEMORY BANK # 1 WITH THE MAIN-STREAM MEMORY<<<<<

==

** MAIN STREAM CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	19.95	6.22	5.948	3.87

LONGEST FLOWPATH FROM NODE 250.00 TO NODE 262.00 = 1360.00 FEET.

** MEMORY BANK # 1 CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	58.93	16.37	3.188	32.41

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 262.00 = 3728.00 FEET.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	42.36	6.22	5.948
2	69.62	16.37	3.188

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 69.62 Tc(MIN.) = 16.37

TOTAL AREA(ACRES) = 36.3

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 IS CODE = 12

>>>>>CLEAR MEMORY BANK # 1 <<<<<

==

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 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.) = 16.37

RAINFALL INTENSITY(INCH/HR) = 3.19

TOTAL STREAM AREA(ACRES) = 36.28

PEAK FLOW RATE(CFS) AT CONFLUENCE = 69.62

****

FLOW PROCESS FROM NODE 260.00 TO NODE 262.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .8400

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 300.00

UPSTREAM ELEVATION(FEET) = 51.50

DOWNSTREAM ELEVATION(FEET) = 48.50

ELEVATION DIFFERENCE(FEET) = 3.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 3.625

WARNING: INITIAL SUBAREA FLOW PATH LENGTH IS GREATER THAN

THE MAXIMUM OVERLAND FLOW LENGTH = 60.00

(Reference: Table 3-1B of Hydrology Manual)

THE MAXIMUM OVERLAND FLOW LENGTH IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 1.61

TOTAL AREA(ACRES) = 0.28 TOTAL RUNOFF(CFS) = 1.61

****

FLOW PROCESS FROM NODE 262.00 TO NODE 262.00 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.) = 3.63

RAINFALL INTENSITY(INCH/HR) = 6.85

TOTAL STREAM AREA(ACRES) = 0.28

PEAK FLOW RATE(CFS) AT CONFLUENCE = 1.61

** CONFLUENCE DATA **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)	AREA (ACRE)
1	69.62	16.37	3.188	36.28
2	1.61	3.63	6.850	0.28

RAINFALL INTENSITY AND TIME OF CONCENTRATION RATIO

CONFLUENCE FORMULA USED FOR 2 STREAMS.

** PEAK FLOW RATE TABLE **

STREAM NUMBER	RUNOFF (CFS)	Tc (MIN.)	INTENSITY (INCH/HOUR)
1	17.03	3.63	6.850
2	70.37	16.37	3.188

COMPUTED CONFLUENCE ESTIMATES ARE AS FOLLOWS:

PEAK FLOW RATE(CFS) = 70.37 Tc(MIN.) = 16.37

TOTAL AREA(ACRES) = 36.6

LONGEST FLOWPATH FROM NODE 130.00 TO NODE 262.00 = 3728.00 FEET.

****

FLOW PROCESS FROM NODE 300.00 TO NODE 301.00 IS CODE = 21

>>>>>RATIONAL METHOD INITIAL SUBAREA ANALYSIS<<<<<

==

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .3500

S.C.S. CURVE NUMBER (AMC II) = 0

INITIAL SUBAREA FLOW-LENGTH(FEET) = 10.00

UPSTREAM ELEVATION(FEET) = 70.00

DOWNSTREAM ELEVATION(FEET) = 60.00

ELEVATION DIFFERENCE(FEET) = 10.00

SUBAREA OVERLAND TIME OF FLOW(MIN.) = 1.982

WARNING: THE MAXIMUM OVERLAND FLOW SLOPE, 10.%, IS USED IN Tc CALCULATION!

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

SUBAREA RUNOFF(CFS) = 0.48

TOTAL AREA(ACRES) = 0.20 TOTAL RUNOFF(CFS) = 0.48

****

FLOW PROCESS FROM NODE 301.00 TO NODE 302.00 IS CODE = 62

>>>>>COMPUTE STREET FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>(STREET TABLE SECTION # 3 USED)<<<<<

==

UPSTREAM ELEVATION(FEET) = 60.00 DOWNSTREAM ELEVATION(FEET) = 49.00

STREET LENGTH(FEET) = 395.00 CURB HEIGHT(INCHES) = 6.0

STREET HALFWIDTH(FEET) = 15.00

DISTANCE FROM CROWN TO CROSSFALL GRADEBREAK(FEET) = 10.00

INSIDE STREET CROSSFALL(DECIMAL) = 0.020

OUTSIDE STREET CROSSFALL(DECIMAL) = 0.020

SPECIFIED NUMBER OF HALFSTREETS CARRYING RUNOFF = 2

STREET PARKWAY CROSSFALL(DECIMAL) = 0.020

Manning's FRICTION FACTOR for Streetflow Section(curb-to-curb) = 0.0160

Manning's FRICTION FACTOR for Back-of-Walk Flow Section = 0.0160

**TRAVEL TIME COMPUTED USING ESTIMATED FLOW(CFS) = 2.49

STREETFLOW MODEL RESULTS USING ESTIMATED FLOW:

STREET FLOW DEPTH(FEET) = 0.24

HALFSTREET FLOOD WIDTH(FEET) = 5.82

AVERAGE FLOW VELOCITY(FEET/SEC.) = 2.73

PRODUCT OF DEPTH&VELOCITY(FT*FT/SEC.) = 0.66

STREET FLOW TRAVEL TIME(MIN.) = 2.41 Tc(MIN.) = 4.40

100 YEAR RAINFALL INTENSITY(INCH/HOUR) = 6.850

NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.

*USER SPECIFIED(SUBAREA):

USER-SPECIFIED RUNOFF COEFFICIENT = .6600

S.C.S. CURVE NUMBER (AMC II) = 0

AREA-AVERAGE RUNOFF COEFFICIENT = 0.603

SUBAREA AREA(ACRES) = 0.89 SUBAREA RUNOFF(CFS) = 4.02

TOTAL AREA(ACRES) = 1.1 PEAK FLOW RATE(CFS) = 4.50

END OF SUBAREA STREET FLOW HYDRAULICS:

DEPTH(FEET) = 0.28 HALFSTREET FLOOD WIDTH(FEET) = 7.77

FLOW VELOCITY(FEET/SEC.) = 3.12 DEPTH*VELOCITY(FT*FT/SEC.) = 0.88

LONGEST FLOWPATH FROM NODE 300.00 TO NODE 302.00 = 405.00 FEET.

****

FLOW PROCESS FROM NODE 302.00 TO NODE 309.00 IS CODE = 31

>>>>>COMPUTE PIPE-FLOW TRAVEL TIME THRU SUBAREA<<<<<

>>>>>USING COMPUTER-ESTIMATED PIPESIZE (NON-PRESSURE FLOW)<<<<<

==

ELEVATION DATA: UPSTREAM(FEET) = 48.00 DOWNSTREAM(FEET) = 45.00

FLOW LENGTH(FEET) = 205.00 MANNING'S N = 0.013

DEPTH OF FLOW IN 15.0 INCH PIPE IS 8.4 INCHES

PIPE-FLOW VELOCITY(FEET/SEC.) = 6.33

ESTIMATED PIPE DIAMETER(INCH) = 15.00 NUMBER OF PIPES = 1

PIPE-FLOW(CFS) = 4.50

PIPE TRAVEL TIME(MIN.) = 0.54 Tc(MIN.) = 4.94

LONGEST FLOWPATH FROM NODE 300.00 TO NODE 309.00 = 610.00 FEET.

****

```
 FLOW PROCESS FROM NODE    309.00 TO NODE    310.00 IS CODE =  81
----------------------------------------------------------------------------
 >>>>>ADDITION OF SUBAREA TO MAINLINE PEAK FLOW<<<<<
==============================================================================
  100 YEAR RAINFALL INTENSITY(INCH/HOUR) =  6.850
 NOTE: RAINFALL INTENSITY IS BASED ON Tc = 5-MINUTE.
 *USER SPECIFIED(SUBAREA):
 USER-SPECIFIED RUNOFF COEFFICIENT = .7600
 S.C.S. CURVE NUMBER (AMC II) =   0
 AREA-AVERAGE RUNOFF COEFFICIENT = 0.6551
 SUBAREA AREA(ACRES) =    0.54   SUBAREA RUNOFF(CFS) =    2.81
 TOTAL AREA(ACRES) =       1.6   TOTAL RUNOFF(CFS) =       7.31
 TC(MIN.) =    4.94
==============================================================================
 END OF STUDY SUMMARY:
 TOTAL AREA(ACRES)     =       1.6  TC(MIN.) =      4.94
 PEAK FLOW RATE(CFS)   =      7.31
==============================================================================
==============================================================================
 END OF RATIONAL METHOD ANALYSIS
```

Appendix D
HYDRAULIC CALCULATIONS

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

WARE MALCOMB
Leading Design for Commercial Real Estate

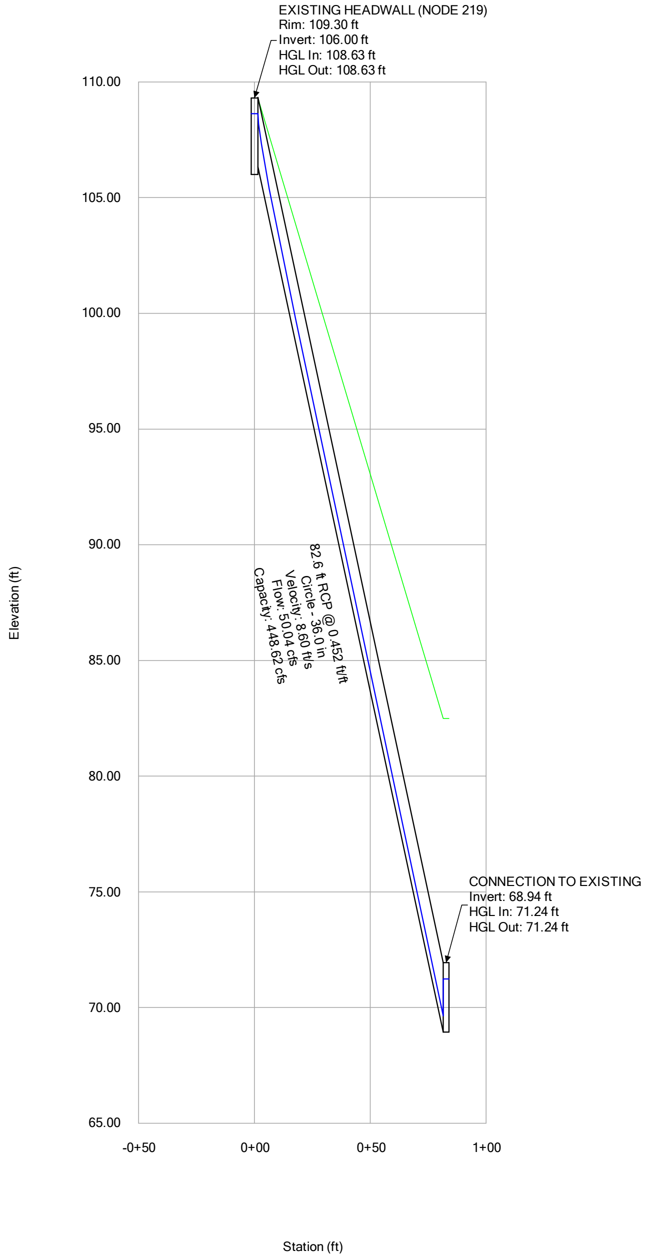
SCOTTISH RITE / HOME DEPOT
1895 & 1561 CAMINO DEL RIO S
SAN DIEGO, CA 92108

STORMCAD LAYOUT PUBLIC STORM SEWER
05.27.2020
18134.00

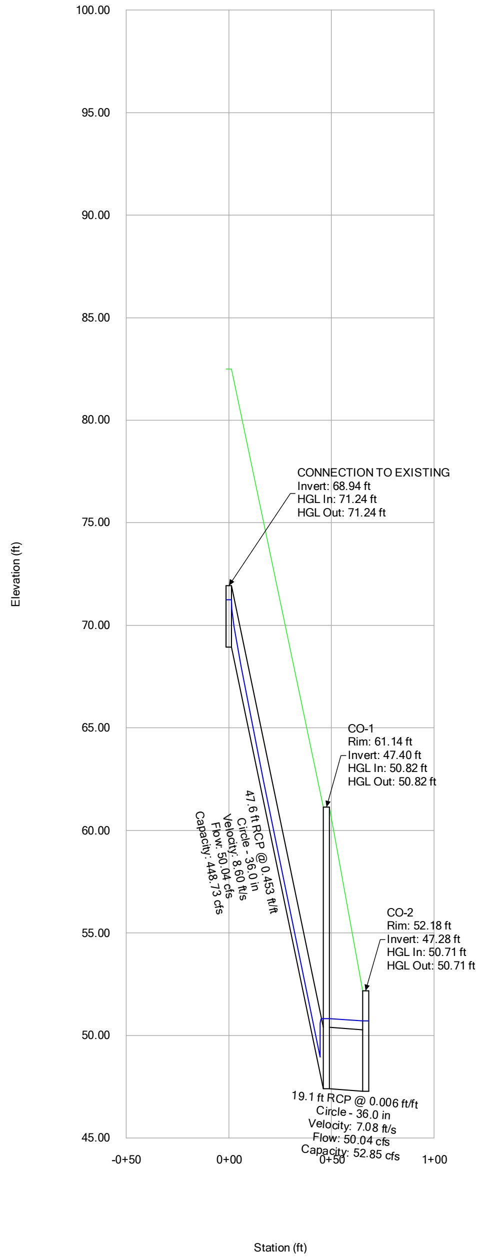
THE HOME DEPOT - MISSION VALLEY
PUBLIC STORM SEWER
Active Scenario: 50-YEAR

Label	Start Node	Stop Node	Invert (Start) (ft)	Invert (Stop) (ft)	Length (Scaled) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Depth (Out) (ft)	Capacity (Full Flow) (cfs)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)
1	EXISTING HEADWALL (NODE 219)	CONNECTION TO EXISTING	106.33	68.94	82.6	0.452	36.0	0.013	50.04	41.90	0.71	448.62	108.63	69.65
2	CONNECTION TO EXISTING	CO-1	68.94	47.40	47.6	0.453	36.0	0.013	50.04	41.91	3.42	448.73	71.24	50.82
3	CO-1	CO-2	47.40	47.28	19.1	0.006	36.0	0.013	50.04	7.08	3.43	52.85	50.82	50.71
4	CO-2	CO-3	47.28	45.01	388.4	0.006	36.0	0.013	50.04	7.08	3.52	50.99	50.71	48.53
5	CO-3	CO-4	45.01	44.33	42.4	0.016	36.0	0.013	50.04	7.08	3.96	84.50	48.53	48.29
6	CO-4	CO-5 (NODE 220)	44.33	42.60	263.7	0.007	36.0	0.013	50.04	7.08	4.20	54.02	48.29	46.80
7	CO-5 (NODE 220)	DEFLECTION 1	42.60	41.12	230.0	0.006	36.0	0.013	55.37	7.83	4.10	53.50	46.80	45.22
8	DEFLECTION 1	PROP CLEANOUT CONNECTION	41.12	40.93	31.5	0.006	36.0	0.013	55.37	7.83	4.07	51.81	45.22	45.00

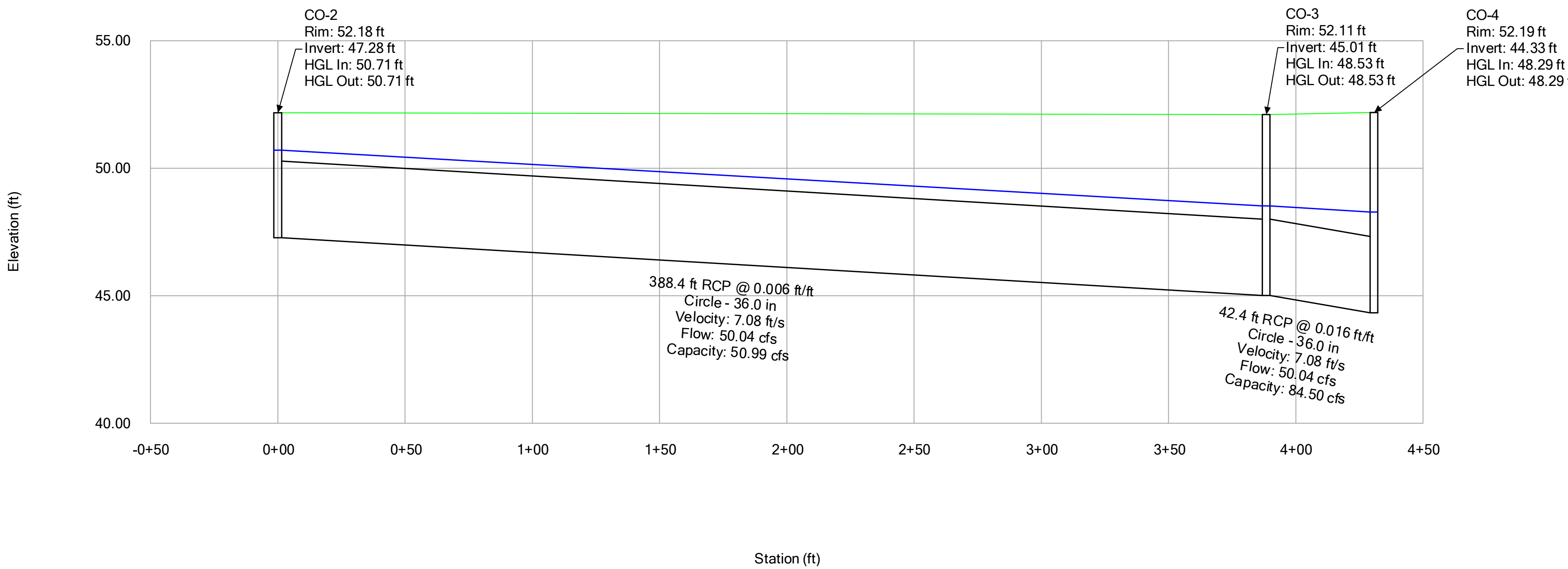
THE HOME DEPOT - MISSION VALLEY
PUBLIC STORM SEWER
Active Scenario: 50-YEAR



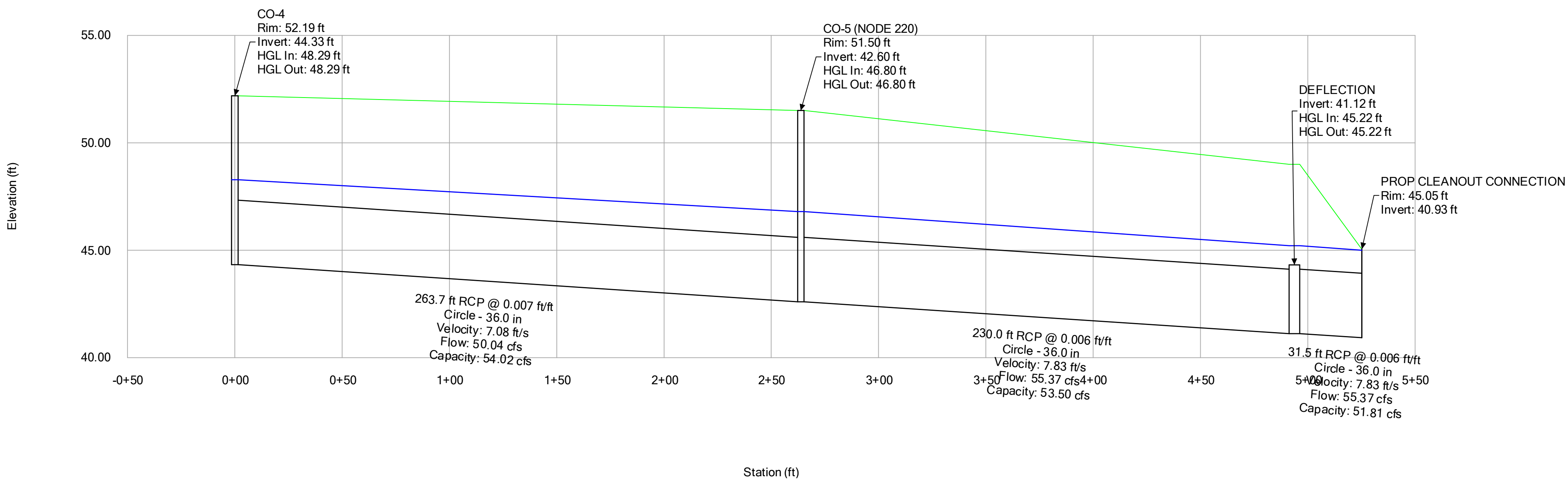
THE HOME DEPOT - MISSION VALLEY
PUBLIC STORM SEWER
Active Scenario: 50-YEAR



THE HOME DEPOT - MISSION VALLEY
PUBLIC STORM SEWER
Active Scenario: 50-YEAR



THE HOME DEPOT - MISSION VALLEY
PUBLIC STORM SEWER
Active Scenario: 50-YEAR



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.

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MOORE TWINING ASSOCIATES, INC.

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED 2-STORY OFFICE BUILDING
SCOTTISH RITE CORPORATE BUSINESS CENTER
1561 CAMINO DEL RIO SOUTH
SAN DIEGO, CALIFORNIA

Project Number: G84101.01

For:

Cushman & Wakefield
4747 Executive Drive, Suite 900
San Diego, California 92121

January 13, 2020

www.mooretwining.com
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

January 13, 2020 G84101.01

Cushman & Wakefield
4747 Executive Drive, Suite 900
San Diego, California 92121

Attention: Mr. Jonathon Perot

Subject: **Preliminary Geotechnical Engineering Investigation**
Proposed 2-Story Office Building
Scottish Rite Corporate Business Center
1561 Camino Del Rio South
San Diego, California

Dear Mr. Perot:

We are pleased to submit this preliminary geotechnical engineering investigation report prepared for a 2-Story office building to be located at the proposed Scottish Rite Corporate Business Center, 1561 Camino Del Rio South (Mission Valley area) in San Diego, California. The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations.

It is recommended that Moore Twining Associates, Inc. (Moore Twining) be provided with updated plans that pertain to the anticipated grading and structure details. Once these details are provided, a design level geotechnical report should be prepared to provide specific recommendations for design and construction.

In addition, it is recommended that Moore Twining be retained to review project plans and specifications, as well as to conduct inspection and testing services for the excavation, earthwork, and foundation phases of construction. These services are necessary to determine if the subsurface conditions are consistent with those used in the analyses and formulation of recommendations for this investigation, and if the construction complies with our recommendations. These services are not, however, part of this current contractual agreement.

PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

We appreciate the opportunity to be of service to Cushman & Wakefield. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

MOORE TWINING ASSOCIATES, INC.

Kenneth J. Clark, CEG 1864
Engineering Geologist
Geotechnical Engineering Division

REGISTERED GEOLOGIST
KENNETH JAMES CLARK
No EG 1864
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA
Exp 5-31-15

EXECUTIVE SUMMARY

Moore Twining Associates, Inc. (Moore Twining) was authorized by Cushman & Wakefield to conduct a preliminary geotechnical engineering investigation for a 2-Story office building to be located at 1561 Camino Del Rio South in San Diego, California.

The subject site comprises a 2.1 acre parcel (Parcel B) is proposed to be established for the Scottish Rite Corporate Business Center (relocation of the Scottish Rite facility currently located east of the site. The site appears to slope down gently, about 11 feet from the south to north ends of the property, and was recently used as an auto dealership. The site includes an existing vacant, two-story, building (approximately 7,000 square feet in plan area) in the northwest portion of the site. Asphaltic concrete and Portland cement concrete parking and drive areas make up the majority of the remainder of the site. An approximate 14 foot wide landscaped strip (lawn) and sidewalk is located between the parking and drive areas and Camino Del Rio South to the north.

An approximate 8-foot high CMU wall is located at the toe of the slope, along the south side of the site. The wall appears to retain about 4 feet of off-site soil. Other CMU site walls/fences occur within the site.

A total of seven (7) test borings were drilled for this investigation to depths of 5 to 50 feet below site grade (BSG). It should be noted that auger refusal was encountered in boring B-2, due to very dense soils, at a depth of about 40 feet BSG. The test borings were drilled by Baja Exploration using a CME-95 drill rig equipped with 8-inch outside diameter (O.D.) hollow-stem augers. The soils encountered in the test borings were logged during drilling.

In addition, two (2) percolation tests were installed at depths of about 3½ feet and 8¼ feet BSG (P-1 and P-2, respectively) in the north portion of the site (proposed bioswale area).

The site is located within the Peninsular Ranges geomorphic province. The project site is located on the southern edge of Mission Valley, which is a narrow valley cut by the west flowing San Diego River drainage. The San Diego River has cut the Mission Valley through older geologic formations which are described in the following sections. The river is also responsible for fluvial sediments deposited within the valley, including a part of the site. The site was graded in the early 50's and 60's. The referenced 1928 aerial image shows the pre-grading condition with the former toe of a north facing slope trending northeast-southwest across the north portion of the site. The image suggests that the majority of the site was cut to achieve the existing grade.

Based on the "Geologic Map of the San Diego 30' x 60' Quadrangle, California," prepared by the California Geological Survey and compiled by Michael P. Kennedy and Siang S. Tan, dated 2005, the south portion of the site is mapped as being underlain by older Mission Valley Formation (Middle Eocene), and the northern portion of the site is shown to be underlain by young colluvial deposits (Holocene and late Pleistocene).

Descriptions of local formations presented in Bulletin 200 - "Geology of the San Diego Metropolitan Area," prepared by Michael P. Kennedy and the California Division of Mines and Geology, dated 1975, indicate the Mission Valley Formation is a marine sandstone unit which is soft and friable with cobble conglomerate tongues comprising up to 30 percent of the section mapped. The formation description also indicates that interbeds and tongues of claystone of brackish water origin locally comprise 20 percent of the section.

The younger colluvial deposits along the north boundary of the site are described as poorly consolidated, poorly sorted, permeable flood-plain deposits of sandy, silty or clay-bearing alluvium.

Based on the scope and results of this investigation, the soils encountered could not be strictly differentiated between alluvial/colluvial soils and the marine/non marine sediments of the older Mission Valley Formation based on lithology alone. However, based on the relatively low N-values obtained during drilling, the upper soils encountered to depth of about 35 feet BSG on the north side of the site (boring B-1) are interpreted to be colluvial/alluvial sediments, with older Mission Valley formation sediments below. Conversely, based on the relatively higher N-values obtained throughout the drilling of borings B-2 and B-3 in the south portion of the site (borings B-2 and B-3), the soils encountered throughout these borings are interpreted to be Mission Valley Formation sediments. Thus, variable soil conditions occur from north to south across the site and proposed building area.

The City of San Diego Seismic Safety Study, "Geologic Hazards and Faults" indicates the site is located on Grid Map 21 of the Hazard Map Series. The map shows the northern portion of the site, in the area where the north portion of the proposed building is planned, is located within a zone of high potential liquefaction (category 31). The south portion of the proposed building is located outside the area of high liquefaction potential.

The ascending slope area in the south portion of the site is located in a zone indicated as "sloping terrain, unfavorable geologic structure, low to moderate risk" (category 53).

The site is not located in a mapped fault rupture hazard zone. The potential for fault rupture on the site is estimated to be low.

The site is considered geotechnically and geologically suitable for the proposed construction with regard to support of the proposed improvements, provided the recommendations contained in this report, and future design level geotechnical investigation reports, are followed. It should be noted that the recommended design consultation and observations during construction by Moore Twining are integral to this conclusion.

Expansion index (swell) testing was performed on a sample of the near surface silty clay soils. The tests indicated a low expansion potential, with expansion index values of 31.

EXECUTIVE SUMMARY (Continued)

Fine to coarse gravel and cobbles were encountered in some of the borings and are common to the geologic nature of the subsurface materials. In order for the onsite soils to be used as engineered fill on the site, removal of over-sized rock should be anticipated. Screening should be anticipated due to the presence of cobbles and coarse gravel. In order to reduce export of materials screened from the soils, it may be possible to crush the oversized material on-site to sizes suitable for use in engineered fill.

Groundwater was encountered in borings B-1 and B-2 at a depth of about 29 feet BSG. Based on our review of California Department of Water Resources Control Board Geotracker data, for sites within about ½ mile of the site, and the range of groundwater depths encountered during the field investigation, an historic high groundwater of about 20 feet was considered for this report.

The results of the liquefaction analyses (based on a groundwater depth of 20 feet BSG) indicate that the lower 5 foot portion of the silty sand layer, encountered at a depth of about 38½ to 43½ feet BSG in test boring B-1, would be susceptible to liquefaction with a total seismic settlement estimated to be about 1.75 inches. Analysis of the other deep boring (B-2) drilled about 200 feet southwest of boring B-1, did not identify any zones of liquefaction or significant seismic settlement. Given the depth and nature of the soils susceptible to liquefaction, design of the building may be based on a differential seismic settlement of 1 inch in 40 feet in addition to the static settlement. In the event that the differential seismic settlement predicted exceeds tolerable limits for design, it is recommended that CPT testing be conducted in the proposed building pad area to better quantify the differential seismic settlement for design of the building. It should be noted that conducting CPTs may be difficult due to potential to encounter gravel and cobbles.

R value tests were conducted on bulk samples of soil collected from depths of ½ to 3 feet BSG. The results of testing a sample of silty sand and clayey sand (mixed) from boring B-6 indicated an R-value of 20. The results of testing a sample of silty clay from boring B-7 indicated an R-value of less than 5. Based on the R-values conducted for this investigation, an R-value of 5 was used for design.

Based on the results of the percolation testing, the near surface soils in the area tested have negligible potential for infiltration of stormwater (less than 0.01 inches per hour). Thus, the predicted infiltration would not meet City of San Diego Storm Water Standards for infiltration type systems.

Chemical testing of soil samples indicated the soils exhibit a “highly corrosive” potential for metallic corrosion and a “negligible” potential for sulfate attack on concrete placed in contact with the near surface soils.

This executive summary should not be used for preliminary design and should be reviewed in conjunction with the details included in the attached report.

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED 2-STORY OFFICE BUILDING
SCOTTISH RITE CORPORATE BUSINESS CENTER
1561 CAMINO DEL RIO SOUTH
SAN DIEGO, CALIFORNIA

Project Number: G84101.01

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical engineering investigation prepared for a 2-Story office building to be located at 1561 Camino Del Rio South in San Diego, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Cushman & Wakefield to perform this investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, existing site features, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The report appendices contain the drawings and site photographs (Appendix A), the logs of borings (Appendix B), the results of laboratory tests (Appendix C), the results of percolation tests (Appendix D), and the results of liquefaction analysis (Appendix E).

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The purpose of the investigation was to conduct a field exploration, a laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following:

2.1.1 A description of general subsurface soil and groundwater conditions encountered;

2.1.2 Soil profile type, site coefficients and adjusted Maximum Considered Earthquake spectral response acceleration parameters in accordance with the 2019 California Building Code, with the exception that this proposal does not include site specific ground motion procedures (see "Purpose of Investigation," above);

2.1.3 Recommendations for earthwork construction, including site and subgrade preparation, and engineered fill. This proposal does not include slope grading/drainage mitigation recommendations for the "off-site" slope located south of the developed portion of the site.

2.1.4 Foundation design parameters including allowable soil bearing capacity, foundation depth, and lateral resistance;

2.1.5 Recommendations for asphalt concrete and Portland cement concrete pavements;

2.1.6 Recommendations for temporary excavations, trench excavation, trench backfill, and excavation stability;

2.1.7 Assessment of liquefaction potential and estimates of static and seismic settlement for foundation design;

2.1.8 Recommendations for slab-on-grade floors and exterior concrete flatwork;

2.1.9 General discussion of the stability of the off-site, (north facing slope) located south of the site.

2.1.10 Discussion of geologic hazards in accordance with the City of San Diego Guidelines for Geotechnical Reports, dated 2018.

2.1.11 The results of two (2) percolation tests;

2.1.12 Evaluation of soil corrosivity potential; and

2.1.13 Final test boring logs and laboratory test results.

This report is provided specifically for the proposed project referenced in the Anticipated Construction section of this report. This investigation did not include a floodplain investigation, environmental investigation, or environmental audit.

2.2 **Scope:** Our proposal, dated December 3, 2019, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 The City of San Diego Guidelines for Geotechnical Reports (2018) and the City of San Diego, Storm Water Standards (2018), were reviewed

2.2.2 The Preliminary Site Plan (Sheet 1), prepared by Ware Malcomb, dated November 22, 2019, was reviewed. This plan is referred to as the “site plan” in this report.

2.2.3 A Conceptual Grading Plan (with site topography), Sheet 2 of 2, prepared by San Dieguito Engineering, Inc., was reviewed.

2.2.4 Historical aerial photographs of the site and surrounding area, produced by EDR, for the years 1928, 1949, 1953, 1964, 1966, 1970, 1979, and 1985, were reviewed. Online historic aerial images were also reviewed.

2.2.5 A report entitled: "Preliminary Geotechnical Engineering Investigation Report, New Home Depot Store - Mission Valley, 1895 Camino Del Rio South, San Diego, California, dated January 10, 2020, prepared by Moore Twining Associates Inc. for the adjacent Home Depot development, was reviewed. This report is referred to herein as the "preliminary geotechnical report for the Home Depot development."

2.2.6 Research regarding the existing site and regional geology was conducted, and the following maps and reports were reviewed and utilized during this investigation:

- Geologic Map of the San Diego 30'x60' Quadrangle, California, Regional Geologic Map Series, prepared by the California Geological Survey and compiled by Michael P. Kennedy and Siang S. Tan, dated 2008;

- City of San Diego's Seismic Safety Study, Geologic Hazards and Faults, Grid Title 21, dated April 3, 2008; and,

2.2.7 Boring permit number LMWP-004295 was obtained from the County of San Diego.

2.2.8 Visual site reconnaissance and subsurface exploration were conducted.

2.2.9 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils encountered.

2.2.10 Mr. Jonathon Perot (Cushman & Wakefield) and Mr. Anthony Khouphongsy (Cushman & Wakefield) were consulted during the investigation.

2.2.11 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and engineering properties of the subsurface soils.

2.2.12 This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, and evaluation, as well as conclusions and recommendations.

3.0 BACKGROUND INFORMATION

The existing site features, site history, previous studies, and the anticipated construction are summarized in the following subsections.

3.1 Site Description: The subject site comprises a 2.1 acre parcel (Parcel B) at 1561 Camino Del Rio South in the City of San Diego, California (see Drawing No. 1 in Appendix A). Parcel B is proposed to be established for the Scottish Rite Corporate Business Center.

For the purpose of this report, project north is considered to be towards Camino Del Rio South, which is about 15 degrees to the west of true north. The site is bordered to the north by Camino Del Rio South, which is a frontage road for Interstate 8 beyond; to the west by an existing auto dealership; to the east by a parking lot for the existing Scottish Rite facility located at 1895 Camino Del Rio South. The south side of the site is bordered by a north facing slope (described below). As part of the adjacent proposed Home Depot development, this slope is proposed to be improved and maintained by Home Depot. The slope is described in Section 5.3 of this report.

The subject property appears to slope gently, about 11 feet from the south to north ends of the property, and was recently used as an auto dealership. The site includes an existing vacant, two-story, building (approximately 7,000 square feet in plan area) in the northwest portion of the site. Asphaltic concrete and Portland cement concrete parking and drive areas make up the majority of the remainder of the site. An approximate 14 foot wide landscaped strip (lawn) and sidewalk is located between the parking and drive areas and Camino Del Rio South to the north. The existing asphalt concrete parking lot appears to be in relatively good condition with only minor block cracking noted in some areas.

Evidence of underground utilities were noted in the landscaped area along the north side of the site. Underground electrical utilities for parking lot lighting were also noted.

An approximate 8-foot high CMU wall is located at the toe of the slope, along the south side of the site. The wall appears to retain about 4 feet of off-site soil. Other CMU site walls/fences occur within the site.

3.2 Site History and Previous Studies: The afore-referenced historical aerial photograph from 1928 shows the site as undeveloped rangeland. The 1928 image shows the pre-grading condition with the former toe of a north facing slope trending northeast-southwest across the north portion of the site. North of the site, a two-lane road is present with undeveloped areas on the banks of the San Diego River which is further north.

The next available aerial image from 1949 shows the site prior to grading with about 3 residential type structures and several large trees in the north portion of the site. The 1953 aerial image shows the slope located south of the site had been cut with terraces, generally consistent with the current slope configuration. The structures are not evident and the site appears to have been graded.

The next available aerial image from 1964 shows cut grading of the south slope had been completed. The image also shows a completed Camino Del Rio South roadway and adjacent multi-lane freeway with a shopping center beyond between the freeway and the San Diego River.

The 1966 aerial image shows the graded site as undeveloped. The 1970 and 1979 images show the site as a parking lot.

Several aerial images from 1994 to 2000 (available on-line) show a building located in the northwest portion of the site and the remainder of the site as a parking lot. Several aerial images from 2001 to 2018 (available on-line) show a larger building located in the northwest portion of the site, and the site appears generally consistent with the conditions noted during our site investigation conducted in December 2019.

No geotechnical or environmental assessment reports had been provided to Moore Twining at the time of this investigation. If available, these reports should be provided to Moore Twining for review.

3.3 **<u>Anticipated Construction</u>**: Based on review of the site plan for the proposed project, the existing former auto dealership building will be demolished and a two-story office building, with a plan area of about 21,000 square feet will be constructed in the north portion of the site. The proposed development is intended to relocate the existing Scottish Rite facility, currently located east of the site. The north side of the building is proposed to be located about 70 feet south of Camino Del Rio South, and the south side of the building is proposed to be located about 300 feet north of the off-site slope. Parking lots and drive aisles (approximately 125 stalls) are proposed south and east of the building.

It is also our understanding that the building design has not been initiated at the time of this report. However, maximum wall and column loads of about 3 kips per lineal foot and 100 kips, respectively, are anticipated for the structure based on our experience with other similar projects.

Appurtenant construction is anticipated to include asphalt and concrete pavements, flatwork and underground utilities.

It is our understanding that the retaining wall located at the toe of the slope, along the south side of the site, is to remain. No new retaining walls are anticipated.

The site appears to slope gently to the north, so cuts and fills on the order of two to three feet are anticipated to provide a flat building pad. It is our understanding that an a stormwater infiltration and/or retention system will be installed in the northeast portion of the site. However, the details were not available at the time this report was prepared.

Off-site improvements to the existing slope located south of the site are planned as part of the Home Depot development to improve drainage, provide erosion protection and to improve shallow slope instability. These improvements may include new lined (concrete or asphalt) brow and terrace ditches, debris fences, drainage structures, etc.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing programs conducted for this investigation are summarized in the following subsections.

4.1 Field Exploration: The field exploration consisted of a site reconnaissance, drilling test borings, conducting standard penetration tests, soil sampling, and percolation testing.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. A site reconnaissance was conducted by Mr. Joe Clark (Staff Geologist with Moore Twining) on December 17th and 18th, 2019. The features noted are described in the Background Information section of this report.

4.1.2 Drilling Test Borings: A total of seven (7) test borings were drilled for this investigation to depths of 5 to 50 feet below site grade (BSG). It should be noted that auger refusal was encountered in boring B-2, due to very dense soils, at a depth of about 40 feet BSG.

The test borings were drilled by Baja Exploration using a CME-95 drill rig equipped with 8-inch outside diameter (O.D.) hollow-stem augers. The soils encountered in the test borings were logged during drilling. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil. Soil samples were collected and returned to our laboratory for classification and testing.

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of borings.

Test boring locations were determined by pacing or steel tape with reference to the existing site features. The boring locations, as shown on Drawing No. 2 in Appendix A, should be considered approximate. Elevations of the test borings were not surveyed as a part of the investigation. However, the boring elevations were estimated based on the referenced Conceptual Grading Plan and are included on the boring logs.

In accordance with the boring permits issued by the County of San Diego, the test borings were backfilled with neat cement. The neat cement backfill was capped with asphalt cold patch. Some settlement should be anticipated at the boring locations.

4.1.3 Soil Sampling: Standard penetration tests were conducted in the test borings, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1⅜-inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by pushing or driving a California modified split barrel ring sampler into the soil. The soil was retained in brass rings, 2.5 inches O.D. and 1-inch in height. The lower 6-inch portion of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory. Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

4.1.4 Percolation Test Holes and Testing: In accordance with our proposal, two (2) percolation tests were installed at depths of about 3½ feet and 8¼ feet BSG (P-1 and P-2, respectively). The approximate test hole locations are shown on Drawing No. 2 in Appendix A.

The percolation tests were installed with a PVC pipe in the borings and the annular space in the bottom of each boring was packed with gravel to stabilize the boreholes. The details of the test hole construction are shown on the percolation test sheets enclosed in Appendix D of this report.

Percolation testing was attempted on December 23, 2019. Further discussion of site percolation properties is provided in Section 5.8 of this report.

4.2 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and relatively undisturbed samples representative of the subsurface materials.

The results of laboratory tests are summarized in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

5.0 FINDINGS AND RESULTS

The findings and results of the research, field exploration and laboratory testing are summarized in the following subsections.

5.1 Geologic Setting: The site is located within the Peninsular Ranges geomorphic province. The project site is located on the southern edge of Mission Valley, which is a narrow valley cut by the west flowing San Diego River drainage. The San Diego River has cut the Mission Valley through older geologic formations which are described in the following sections. The river is also responsible for fluvial sediments deposited within the valley, including a part of the site. As discussed in Section 3.2 of this report, the site was graded in the early 50's and 60's. The referenced 1928 aerial image shows the pre-grading condition with the former toe of a north facing slope trending northeast-southwest across the north portion of the site. The image suggests that the majority of the site was cut to achieve the existing grade.

Based on the “Geologic Map of the San Diego 30’ x 60’ Quadrangle, California,” prepared by the California Geological Survey and compiled by Michael P. Kennedy and Siang S. Tan, dated 2005, the south portion of the site is mapped as being underlain by older Mission Valley Formation (Middle Eocene), and the northern portion of the site is shown to be underlain by young colluvial deposits (Holocene and late Pleistocene). The map also indicates numerous bedding dips in the site region, measured in the Mission Valley Formation, the underlying Stadium Conglomerate, and the overlying San Diego Formation. These bedding dips predominantly range from about 2 to 5 degrees from horizontal.

Descriptions of local formations presented in Bulletin 200 - “Geology of the San Diego Metropolitan Area,” prepared by Michael P. Kennedy and the California Division of Mines and Geology, dated 1975, indicate the Mission Valley Formation is a marine sandstone unit which is soft and friable with cobble conglomerate tongues comprising up to 30 percent of the section mapped. The formation description also indicates that interbeds and tongues of claystone of brackish water origin locally comprise 20 percent of the section.

The younger colluvial deposits along the north boundary of the site are described as poorly consolidated, poorly sorted, permeable flood-plain deposits of sandy, silty or clay-bearing alluvium.

Drawing No. 3 in Appendix A shows the regional geology of the site area.

Based on the scope and results of this investigation, the soils encountered could not be strictly differentiated between alluvial/colluvial soils and the marine/non marine sediments of the older Mission Valley Formation based on lithology alone. However, based on the relatively low N-values obtained during drilling, the upper soils encountered to depth of about 35 feet BSG on the north side of the site (boring B-1) are interpreted to be colluvial/alluvial sediments, with older Mission Valley formation sediments below. Conversely, based on the relatively higher N-values obtained throughout the drilling of borings B-2 and B-3 in the south portion of the site (borings B-2 and B-3), the soils encountered throughout these borings are interpreted to be Mission Valley Formation sediments. Thus, variable soil conditions occur from north to south across the site and proposed building area.

Our interpretation of the distribution of geologic units across the site is provided on the Site Geologic Map, Drawing No. 4 in Appendix A of this report.

5.2 City of San Diego Seismic Safety Study, "Geologic Hazards and Faults: The City of San Diego Seismic Safety Study, "Geologic Hazards and Faults", was reviewed. The site is located on Grid Map 21 of the Hazard Map Series. The map shows the northern portion of the site, in the area where the north portion of the proposed building is planned, is located within a zone of high potential liquefaction (category 31). The south portion of the proposed building is located outside the area of high liquefaction potential.

Based on Grid Map 21, the ascending slope area in the south portion of the site is located in a zone indicated as "sloping terrain, unfavorable geologic structure, low to moderate risk" (category 53). The condition and stability of the slope south are discussed below under Section 5.3 of this report.

5.3 Conditions and Stability of Slope Located South of Site: The subject site is located at the base of an ascending slope that forms the south flank of Mission Valley. This slope is off-site to the south. Similar slope conditions occur within numerous developed properties to the east and west of the subject site. The slope includes a lower cut slope portion and an upper native slope portion. The lower cut slope portion of the slope was cut as part of grading conducted in the 1950s and includes several terraces.

The lower (cut) portion of the slope is about 75 feet high with an overall average gradient of about 2 horizontal (H) to 1 vertical (V). However, the gradients of the intermediate slopes between existing benches are steeper than 2H:1V. Above the cut portion of the slope, the slope appears to be native. The height of the native slope extending upward from the top of lower (cut) varies, with a maximum slope height of about 210 vertical feet where the slope extends up to a residential neighborhood. The overall average gradient of the upper (native) portion of the slope, based on limited topographic information, is about 1.5H to 1V.

The cut (lower terraced) portion and the native (upper) portion of the slope area located south of the site were observed. The cut (lower) portion of the slope was vegetated with native grasses and shrubs, and a few trees. Evidence of rill type erosion, shallow soil slips, remedial erosion control measures and slope repairs, and accumulated sediment was noted on the terraced slopes to the east of the subject property (adjacent to the proposed Home Depot development). However, evidence of significant erosion, shallow slope instability, and/or deeper global instability were not noted on the cut slopes adjacent to the subject site.

The upper, north facing native slope (above the existing graded cut slope) was observed to be covered with native grasses, dense bushes, and trees. Based on our site observations, we did not identify any significant soil slips or excessive erosion within the native slope which would require repair. The native slopes appeared to be performing well.

The conditions and stability of the slope located south of the site, and the portion of this slope which extends east from the site are described in detail in the referenced preliminary geotechnical report for the adjacent Home Depot development. As part of the proposed Home Depot development, it is our understanding this off-site slope south of the subject site is proposed to be improved and will be maintained by others as it is not within the subject property.

5.4 **Existing Pavement Thickness**: The pavement thicknesses measured in the test borings ranged from about 3.0 to 3¾ inches of asphalt concrete, underlain by about 1 to 2¼ inches of aggregate base.

5.5 **Soil Profile:** The near surface soils encountered below the existing pavements predominantly consisted of silty sands and clayey sands in the upper 10 to 15 feet BSG. Below about 15 feet BSG, sandy silts and sandy lean clays were predominant with subordinate silty sand layers extending to the maximum depth explored of 50 feet BSG. However, a silty clay was encountered in the south portion of the site, at depths of about 4 feet and less than 1 foot in borings B-2 and B-7, respectively. Abundant cobbles were encountered in boring B-4 at a depth of about 5 feet BSG.

5.6 **Soil Engineering Properties:** The engineering properties of the subsurface soils encountered during this investigation are summarized below. As described in Section 5.1 of this report, borings B-1 and B-4 which were drilled in the northern portion of the site generally had lower standard penetration test, N-values in the upper 30 feet BSG compared with borings B-2 and B-3 which were drilled in the south portion of the site.

The granular soils encountered were generally medium dense with the exception that loose granular soils were encountered in borings B-1 and B-4 at depths of 8½ and 10 feet, respectively. Dense silty sands were encountered in boring B-1 at a depth of 48½ feet BSG, and in boring B-2 at a depth of 35 feet BSG. Auger and sampler refusal were encountered in boring B-2 at a depth of about 40 feet BSG.

The sandy silts, silty clays and sandy lean clays generally ranged from medium stiff to very stiff.

Expansion Index Testing: The results of testing of the near surface silty clay soils indicated a low expansion potential, with an expansion index 31.

Atterberg Limits Testing: Plasticity indices and liquid limits testing conducted on samples of the silty and clayey soils ranged from 10 to 18 and 36 to 37, respectively.

Direct Shear: The results of direct shear testing conducted on near surface samples of silty clay and silty sand indicated an internal angle of friction of 32 degrees for both samples, with cohesion values of 280 and 240 pounds per square foot for the silty clay and silty sand samples, respectively.

Consolidation: Consolidation testing was conducted on two (2) samples collected from depths of between 5 and 6½ BSG. The results indicated total consolidations of about 6.2 percent and 7.4 percent at 16 kips per square foot normal load.

Moisture/Density Relationships: One (1) maximum density/optimum moisture determination test was conducted on a sample of silty sand and clayey sand (mixed) collected from depths of ½ to 3 feet BSG. The results indicated a maximum dry density of 123.2 pounds per cubic foot, with an optimum moisture content of 11.7 percent.

R-Value: R value tests were conducted on bulk samples of soil collected from depths of ½ to 3 feet BSG. The results of testing a sample of silty sand and clayey sand (mixed) from boring B-6 indicated an R-value of 20. The results of testing a sample of silty clay from boring B-7 indicated an R-value of less than 5.

Chemical Tests: The results of chemical testing performed on a near surface soil sample indicated a pH value of 8.9; a minimum resistivity value of 1,868 ohm-centimeter; a 0.0024 percent by weight concentration of sulfate; and a "not detected concentration of chloride (less than 0.00060 percent by weight).

The foregoing is a general summary of the soil conditions encountered in the test borings drilled for this investigation. Detailed descriptions of the soils encountered at each test boring are presented in the logs of borings in Appendix B. The stratification lines in the logs represent the approximate boundary soil types; the actual in-situ transition may be gradual.

5.7 **Groundwater Conditions:** Groundwater was encountered in borings B-1 and B-2 at a depth of about 29 feet BSG.

Based on our review of California Department of Water Resources Control Board Geotracker data, two sites were identified within ½ mile of the site that included groundwater data. Review of the data identified five (5) monitoring wells installed at a fuel station about one-half (½) mile northeast of the site with groundwater depths ranging from about 27 to 28 feet BSG in 2004. These wells are on a property near the San Diego River, which has a similar elevation to the site of about 58 feet AMSL. A second site about one-half (½) mile west of the site indicated groundwater depths ranging from about 15 to 21 feet BSG for monitoring events from 2006 to 2012. The monitoring well site elevations range from about 37 to 41 feet AMSL and the groundwater gradient is indicated to be relatively flat (groundwater elevations reported from 22 to 23 feet AMSL). Considering that the ground surface elevation at the north side of the proposed building area at the subject is about 45 feet AMSL, projecting this data to the site would indicate a groundwater depth greater than 20 feet BSG.

Groundwater data on the Department of Water Resources Water Well Data Library was also reviewed. The nearest well to the site in the database (16S03W13Q004S) includes groundwater elevation measurements from 1978 to 1990 and is located about ½ mile north of the site (north of the San Diego River). The elevations of groundwater in this well ranged from elevation 33 feet in 1980 to elevation 25 feet in 1989. Based on the ground surface elevation of 45 feet BSG at the north side of the proposed building area and considering a groundwater elevation of 25 feet from the 1989 data from the DWR well near the river, a groundwater depth of 20 feet would be projected at the subject site.

Considering the locations and elevations of researched well data, and the range of groundwater depths encountered during the field investigation, an historic high groundwater of about 20 feet was considered for this report.

It should be recognized that groundwater elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation/measurements may vary from those encountered both during the construction phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

5.8 **Percolation of Near Surface Soils:** In accordance with our proposal, percolation tests were installed in an area being considered for stormwater infiltration, as described in Section 4.1.4 of this report. The test holes were installed to depths of about 3½ feet and 8¼ feet BSG at test locations P-1 and P-2, respectively. The approximate test hole locations are shown on Drawing No. 2 in Appendix A.

The near surface soil types encountered in the test holes include silty sands with clay, which were underlain by sandy lean clay extending to a depth of about 10 feet. The sandy lean clay was underlain by silty sand and sandy lean clay to the maximum depth explored in the nearby borings of about 20 feet BSG (see logs of borings B-5, P-1 and P-2 in Appendix A).

No groundwater was encountered in the borings B-5, P-1 and P-2 at the time of drilling. Based on other borings drilled nearby, the depth to groundwater at the percolation test holes was estimated to be about 29 feet BSG on December 17th, 2019.

On December 23, 2019, due to apparent recent runoff from rain events, both percolation holes contained standing water at a depth of about 1 to 2½ feet below the ground surface. Although it was not raining at the time of our percolation testing, about ½ inch of precipitation occurred in the site area during the 24 hour period prior to the percolation testing. Prior to testing, about 1.5 gallons of water was added to percolation hole P-1 to achieve the appropriate water column height for testing. Due to the standing water height in test P-2, no water was added prior to taking water level measurements. Measurements conducted on December 23, 2019 indicated no measurable drop in the level of the water in test hole P-2 over a period of about 6 hours. Measurements conducted on December 23, 2019 initially indicated very slow percolation in test hole P-1, with no measurable drop in the level of the water in test hole P-1 over the final measurement interval of 81 minutes. The percolation test measurements are included in Appendix D of this report.

On December 27, 2019, a Moore Twining geotechnical engineer observed the upper, visible, portions of the test holes and the pavement surface conditions exposed in the test holes. It was noted that some minor sloughing of the aggregate base materials below the asphalt concrete had sloughed into the test hole. In addition, evidence of some water seepage from the aggregate base section below pavements appeared to be seeping into the percolation test holes.

Considering that the water levels in the test holes did not decline appreciably during our measurements on December 23, 2019, the near surface soils in this area of the site have a negligible potential for infiltration of stormwater.

6.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for project design and preparation of geotechnical related construction specifications are summarized in the following subsections. The evaluations were based upon the subsurface conditions determined from the investigation, review of available maps and reports, and our understanding of the proposed construction. The conclusions obtained from the results of our evaluations are described in Section 7.0 of this report (Conclusions).

6.1 **Existing Surface Conditions:** The subject site is fully developed and includes an existing structure, pavements, site walls, underground utilities and other site improvements. Due to the existing site development, as part of the site preparation, existing foundations, slabs-on-grade, utilities and other improvements will need to be removed and the resulting excavations properly prepared and backfilled. All existing surface and subsurface structures, such as shallow foundations, floor slabs, utilities, etc., should be removed entirely and not buried in place. Areas with existing improvements should be over-excavated to at least 12 inches below the bottom of the existing improvements to be removed, or to the depth to remove disturbed soils from the demolition activity, whichever is greater. All excavations conducted as part of the demolition should be backfilled with engineered fill.

6.2 **Wet, Unstable Soils:** During the December 2019 field investigation, the moisture contents of several samples collected from near the ground surface to a depth of about 6 feet BSG were estimated to significantly exceed optimum moisture contents for compaction. Moisture contents of several tests conducted on near surface samples ranged from 14.3 to 21.3 percent, exceeding the measured optimum moisture content of 11.7 percent determined for a mixture of near surface clayey sands and silty sands. In addition, as noted in this report, some seepage was noted within the aggregate base section after periods of precipitation below the asphalt concrete pavement in the percolation tests holes. Accordingly, it is anticipated that some of the soils excavated during site grading will need to be aerated, i.e. dried, to meet the moisture conditioning requirements of this report to allow compaction of the soils as engineered fill. Due to the high soil moisture contents, wet soils could be exported from the site, or spread and repeatedly mixed/disced to dry, or chemically treated to dry the soils in order to achieve proper compaction.

In addition, where wet, unstable soil conditions are encountered, methods such as aeration, mixing wet soils with drier soils, chemical treatment, or the use of aggregate base or crushed rock and a geotextile stabilization fabric may be required to achieve a stable condition at the bottom of the excavations and in areas that require subgrade preparation.

6.3 **Oversize Rock / Soil Processing:** Fine to coarse gravel and cobbles were encountered in some of the borings and are common to the geologic nature of the subsurface materials. In order for the onsite soils to be used as engineered fill on the site, removal of over-sized rock should be anticipated. Screening should be anticipated due to the presence of cobbles and coarse gravel. In order to reduce export of materials screened from the soils, it may be possible to crush the oversized material on-site to sizes suitable for use in engineered fill.

6.4 **Static Settlement and Bearing Capacity of Shallow Foundations:** The potential for excessive total and differential static settlement of foundations and slabs-on-grade was evaluated for the proposed building site. The increases in effective stress to underlying soils which can occur from new foundations and structures and placement of fill, etc. can cause vertical deformation of the soils, which can result in damage to the overlying structure and improvements. The differential component of the settlement is often the most damaging. In addition, the allowable bearing pressures of the soils supporting the foundations were evaluated for shear and punching type failure of the soils resulting from the imposed foundation loads.

In order to reduce the potential for excessive differential static settlement of the new foundations, this report recommends that the new building pad be prepared by over-excavating the entire pad to provide a uniform layer of engineered fill at least 2 feet thick below the new foundations. In addition, considering that demolition of the existing building and site improvements will result in disturbance of the near surface soils, the soils disturbed from site demolition will need to be excavated to expose undisturbed soils. The allowable soil bearing pressure for spread foundations supported on engineered fill is 2,500 pounds per square foot for dead-plus-live loads. Based on this bearing capacity, the following static settlements are anticipated for the foundations and slabs on grade: 1) a total static settlement of 1 inch and 2) a differential static settlement of ½ inch in 40 linear feet.

6.5 **<u>Expansive Soils</u>:** One of the potential geotechnical hazards evaluated at this site is the expansion potential of the near surface soils. Over time, expansive soils will experience cyclic drying and wetting as the dry and wet seasons pass. Expansive soils experience volumetric changes (shrink/swell) as the moisture content of the clayey soils fluctuate. These shrink/swell cycles can impact foundations and lightly loaded slabs-on-grade when not designed for the anticipated expansive soil pressures. Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). Expansion potential may not manifest itself until months or years after construction. The potential for damage to slabs-on-grade supported on expansive soils can be reduced by placing non-expansive fill underlying the slabs-on-grade and extending the perimeter foundations to depths necessary to establish a moisture cutoff.

Expansion index (swell) testing was performed on a sample of the near surface silty clay soils. The tests indicated a low expansion potential, with expansion index values of 31. It is recommended to support floor slabs on at least 6 inches of aggregate base material.

6.6 **<u>Seismic Ground Rupture and Design Parameters</u>:** The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The closest active fault is the Rose Canyon Fault Zone, which is located approximately 4 miles west of the site. The City of San Diego Seismic Safety Study indicates a concealed segment of the Texas Street fault is located about 700 feet northeast of the site. The fault category for the Texas Street fault is described as "potentially active, inactive, presumed inactive, or activity unknown." Accordingly, the potential for ground rupture at the site is considered low.

Seismic coefficients and spectral response acceleration values were developed in accordance with the 2019 California Building Code (CBC). The CBC methodology for determining design ground motion values is based on U.S. Geological Survey seismic hazard maps, which incorporate both probabilistic and deterministic seismic ground motion.

A table providing the recommended seismic coefficient and earthquake spectral response acceleration values for the project site is included in the "Seismic Factors" recommendations section of this report. The standard penetration test results indicate a Site Class D based upon N-values between 15 and 50 blows per foot, for the upper 100 feet BSG. These field N-value results indicate the subgrade soils are considered a stiff soil site based on the method included in ASCE 7-16, Section 20.4.2.

A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.624g was determined for the site using the Ground Motion Parameter Calculator from the Structural Engineer's Associates of California (https://seismicmaps.org). A Maximum Considered Earthquake magnitude of 6.89 was determined for the site based on deaggregation analysis (United States Geological Survey deaggregation website (https://earthquake.usgs.gov/hazards/interactive/).

6.7 Liquefaction and Seismic Settlement: Based on Grid Tile 21 of the City of San Diego Seismic Safety Study Geologic Hazards and Faults, dated April 3, 2008, prepared by the City of San Diego Development Services Department, the northern portion of the subject site (including the northern portion of the building) is located in a liquefaction hazard zone.

Liquefaction and seismic settlements are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing can result. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and a particularly long duration of ground shaking are the common characteristics for liquefaction.

Liquefaction and seismic settlement analyses were conducted based on soil properties revealed by the test borings and the results of laboratory testing. The analyses were conducted for soils encountered in the deeper borings B-1 and B-2, using the software program LiquefyPro developed by CivilTech. A horizontal ground acceleration of 0.624g, a maximum considered earthquake of 6.89 and a high groundwater depth of 20 feet were used in the analysis. The N-values generated were used to determine the cyclic stress ratio needed to initiate liquefaction. Soil parameters, such as wet unit weight, N-value, fines content, and depth of N-value tests, were input for the soil layers encountered throughout the depths explored (see test boring logs, Appendix B). A hammer energy correction of 1.2 was applied to the field N-value results based on the results of equipment specific hammer energy calibrations. The hammer energy ratio correction was based on overall transfer efficiency of 71 percent for the hammer as indicated in a report prepared by SPT CAL for the CME-95 drill rig used, dated September 13, 2019 (included in Appendix E of this report).

One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils, however, seismic settlements are typically largest where liquefaction occurs (saturated soils).

Potential liquefaction of fine-grained soils were evaluated considering the plasticity index guidelines included in the document entitled: "Liquefaction Susceptibility Criteria for Silts and Clays", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, November 2006 (Ross W. Boulanger and I.M. Idriss). The referenced journal article states: "For practical purposes, fine-grained soils can confidently be expected to exhibit clay-like behavior if they have PI $\geq$ 7." Based on the plasticity index values obtained for fine-grained soils (silts and clays) based on the laboratory testing conducted herein, the fine-grained soil layers were not considered susceptible to liquefaction.

As indicated in Section 5.1 of this report, variable soil conditions were encountered in the building pad area. The results of the liquefaction analyses indicate that the lower 5 foot portion of the silty sand layer, encountered at a depth of about 38½ to 43½ feet BSG in test boring B-1, would be susceptible to liquefaction. The associated seismic settlement was estimated to be about 1¾ inches. Analysis of the other deep boring (B-2) drilled about 200 feet southwest of boring B-1, did not identify any zones of liquefaction or significant seismic settlement.

Given the depth where liquefaction is expected to occur, it is not expected that the loss of strength associated would impact the ability of the soils to support shallow spread foundations. However, 1¾ inches of total seismic settlement was estimated for the proposed building based on the design level earthquake. Given the depth and nature of the soils susceptible to liquefaction, design of the building may be based on a differential seismic settlement of 1 inch in 40 feet in addition to the static settlement. In the event that the differential seismic settlement predicted exceeds tolerable limits for design, it is recommended that additional exploration, such as use of CPT testing, be conducted in the proposed building pad area to better quantify the differential seismic settlement for design of the building. It should be noted that conducting CPTs may be difficult due to potential to encounter gravel and cobbles.

The liquefaction and seismic settlement analysis output are included in Appendix E of this report.

6.8 **<u>Stability of Off-Site Slope</u>:** The existing hillside south of the site is located in City of San Diego geologic hazard category 53, which indicates: "sloping terrain, unfavorable geologic structure, low to moderate risk." The general slope configuration and conditions are described in Section 5.3 of this report.

As indicated in Section 5.3 of this report, significant evidence of erosion, shallow slope instability, and/or deeper global instability were not noted on the cut slopes adjacent to the site. Also, we did not identify any significant soil slips or excessive erosion within the native slope (above the cut portion) which would require repair.

As part of the proposed adjacent development, we understand this off-site slope is proposed to be improved and maintained by others as part of the Home Depot development. The referenced preliminary geotechnical report for the adjacent Home Depot development addresses the slopes south of the proposed Home Depot and Scottish Rite developments and states: *"The existing north facing* *native slope was evaluated to identify unfavorable geologic structures as a part of this investigation. No unfavorable geologic structures were identified and this upper native slope has been performing well for quite some time. Geologic mapping referenced herein indicates bedding local to the site is neutral with respect to gross stability. Therefore, the slope is considered stable and potential gross instability of the upper native slope is low."*

6.9 **<u>Asphaltic Concrete (AC) Pavements</u>:** Recommendations for onsite asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highways Design Manual. The analysis was based on traffic index values ranging from 5.0 to 8.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

It should be noted that if the pavements are constructed prior to the building construction, the additional construction truck traffic should be considered in the selection of the traffic index value. If more frequent or heavier traffic is anticipated and higher Traffic Index values are needed, Moore Twining should be contacted to provide additional pavement section designs.

Based on the results of the testing and the procedures in the Caltrans Highway Design Manual, an R-value of 5 was used for the pavement design.

6.10 **<u>Portland Cement Concrete (PCC) Pavements</u>:** Recommendations for Portland cement concrete (PCC) pavement structural sections are presented in the "Recommendations" section of this report. The PCC pavement sections are based upon the amount and type of traffic loads being considered and the strength of the subgrade soils which will support the pavement. The measure of the amount and type of traffic loads are based upon an index of equivalent axle loads (EAL) from the loading of heavy trucks called a traffic index (T.I).

The results of R-value testing performed in accordance with California Test Method 301 were used to estimate the pavement subgrade modulus. A modulus of subgrade reaction, K-value, for the pavement section, of 110 psi/in was used for the pavement design, when considering the aggregate base section.

The recommendations provided in this report for PCC pavements are based on traffic indices ranging from 5.0 and 8.0 and the design procedures contained in the Portland Cement Association "Thickness Design of Highway and Street Pavements."

The PCC pavement sections were designed for a life of 20 years and a load safety factor of 1.1. The section thicknesses for a traffic index of 5.0 were evaluated for light passenger vehicular loading and traffic indices of 5.0 to 8.0 were evaluated based on typical axle loads for a garbage truck (single axle weight of 20,000 pounds and a tandem axle weight of 35,000 pounds).

6.11 **Stormwater Infiltration**: Based on the results of the percolation testing, the near surface soils in the area tested have negligible potential for infiltration of stormwater (less than 0.01 inches per hour). Thus, the predicted infiltration would not meet City of San Diego Storm Water Standards for infiltration type systems.

In the event that storm water systems which allow infiltration of water into the soils are used, these systems should be setback at least 30 feet from the structure and building foundations to reduce potential impacts to the proposed structure from expansive soil movement and/or settlement.

6.12 **Soil Corrosion:** The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The results of a soil sample analysis of near surface silty clay soils indicate a minimum resistivity value of 1,868 ohm-centimeter (results included in Appendix C of this report). The National Association of Corrosion Engineers (NACE) provides corrosion severity ratings listed in the Table No. 1 below.

Table No. 1
Soil Resistivity and Corrosion Potential Ratings

Soil Resistivity (ohm cm)	Corrosion Potential Rating
>20,000	Essentially non-corrosive
10,000 - 20,000	Mildly corrosive
5,000 - 10,000	Moderately corrosive
3,000 - 5,000	Corrosive
1,000 - 3,000	Highly corrosive
<1,000	Extremely corrosive

Therefore, the near-surface soils exhibit a "highly corrosive" potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the "highly corrosive" corrosion potential. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

6.13 **Sulfate Attack of Concrete:** Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil/groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with concrete is to perform testing to determine the sulfates present in the soils. The results of the sulfate analysis of a near surface soil sample indicates 0.0024 percent by weight. This result should be compared with the provisions of ACI 318, section 4.3 to provide guidelines for concrete exposed to sulfate-containing solutions. Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios. The test results are included in Appendix C of this report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

The soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters.

7.0 CONCLUSIONS

Based on the data collected during the field and laboratory investigations, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, we present the following general conclusions.

7.1 The site is considered geotechnically and geologically suitable for the proposed construction with regard to support of the proposed improvements, provided the recommendations contained in this report, and future design level geotechnical investigation reports, are followed. It should be noted that the recommended design consultation and observations during construction by Moore Twining are integral to this conclusion.

7.2 In general, the near surface soils encountered predominantly consisted of silty sands and clayey sands in the upper 10 to 15 feet BSG. Below about 15 feet BSG, sandy silts and sandy lean clays were predominant with subordinate silty sand layers extending to the maximum depth explored of 50 feet BSG. Abundant cobble material was encountered in boring B-4 at a depth of about 5 feet BSG.

Based on natural deposition and previous grading of the site, the north side of the site is in an area more prone to colluvial and alluvial deposition, relative to the south side of the site where older formational soils are interpreted. These conditions are reflected in the standard penetration test, N-value, results from testing conducted in the borings drilled for this investigation.

7.3 In order to reduce the potential for excessive differential static settlement of the new foundations, he new building pad should be prepared by over-excavating the entire pad to provide a uniform layer of engineered fill at least 2 feet thick below the new foundations.

7.4 Expansion index (swell) testing was performed on a sample of the near surface silty clay soils. The tests indicated a low expansion potential, with expansion index values of 31. It is recommended to support floor slabs on at least 6 inches of aggregate base material.

7.5 Groundwater was encountered in borings B-1 and B-2 at a depth of about 29 feet BSG.

7.6 The existing north facing native slope (off-site) adjacent to the south side of the site was evaluated by Moore Twining as a part of the evaluation for the adjacent development. The preliminary geotechnical report for the Home Depot development (Project #D050R0.01, dated January 10, 2019) indicates that the slope is considered stable and potential gross instability of the upper native slope is considered low.

7.7 Based on our site observations and the results of percolation tests, the near surface soils have negligible potential for infiltration of stormwater.

7.8 Variable soil conditions were encountered in the building pad area. The results of the liquefaction analyses indicate that the lower 5 foot portion of the silty sand layer, encountered at a depth of about 38½ to 43½ feet BSG in test boring B-1, would be susceptible to liquefaction with a total seismic settlement estimated to be about 1.75 inches. Analysis of the other deep boring (B-2) drilled about 200 feet southwest of boring B-1, did not identify any zones of liquefaction or significant seismic settlement. Given the depth and nature of the soils susceptible to liquefaction, design of the building may be based on a differential seismic settlement of 1 inch in 40 feet in addition to the static settlement. In the event that the differential seismic settlement predicted exceeds tolerable limits for design, it is recommended that CPT testing be conducted in the proposed building pad area to better quantify the differential seismic settlement for design of the building. It should be noted that conducting CPTs may be difficult due to potential to encounter gravel and cobbles.

7.9 Variable amounts of fine to coarse gravel and cobbles are anticipated in the near surface soils. Due to the oversize rock, the cobble material will need to be removed such as by screening prior to placement and compaction as engineered fill.

7.10 The site is not located in a mapped fault rupture hazard zone. The potential for fault rupture on the site is estimated to be low.

7.11 The analytical results of a soil sample analysis indicate that the near-surface soils exhibit a “highly corrosive” corrosion potential to buried metal objects.

7.12 Chemical analyses indicated a “negligible” potential for sulfate attack on concrete placed in contact with the near surface soils.

8.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, the following recommendations are presented for use in the project design and construction. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered.

Where the requirements of a governing agency, utility agency or pipe manufacturer differ from the recommendations of this report, the more stringent recommendations should be applied to the project.

8.1 General

8.1.1 Grading and drainage plans, and foundation plans, when available, should be provided to Moore Twining for review to determine if the following recommendations need to be updated or revised. Once these details are provided, a design level geotechnical report should be prepared to provide specific recommendations for final design prior to bidding and construction. In addition, in the event the estimated seismic settlements are considered excessive for design, a supplemental investigation should be conducted to further evaluate the estimated seismic settlement. The recommendations presented in this report could change depending on the extent of proposed grading, etc. Therefore, it is critical that updated improvement plans, when available, be provided to Moore Twining for review.

8.1.2 Once the foundation loads are available, this information should be provided to Moore Twining for review to determine if the recommendations for site preparation are suitable for the actual design loads.

8.1.3 A preconstruction meeting including, as a minimum, the owner, general contractor, earthwork contractor, contractor's land surveyor, foundation and paving subcontractors, and Moore Twining should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling.

8.1.4 A demolition plan should be developed to identify the existing surface and subsurface improvements to be removed and those which are to remain.

8.1.5 The Contractor(s) bidding on this project should determine if the information included in the construction documents and this geotechnical engineering investigation report are sufficient for accurate bid purposes. If the data are not sufficient, the Contractor shall notify the client in writing that insufficient data are available to prepare an accurate bid for the project.

8.1.6 Contractors should be aware that wet soils are anticipated that will likely be significantly above the optimum moisture content required for proper compaction and could require soil drying or chemical treatment for stabilization to achieve the required relative compaction. In addition, measures such as placement of geotextile stabilization fabric and aggregate base may be required in areas of wet soils to achieve stable conditions.

8.1.7 Appropriate construction methods and equipment, such as low vibration equipment, should be used adjacent to the existing improvements (such as retaining walls) so as not to damage existing improvements which are to remain.

8.2 <u>Building Slope Setbacks, Site Grading, and Drainage for Building Pad</u>

8.2.1 The proposed building should be setback horizontally a minimum of 30 feet from the toe (or existing retaining wall constructed at the base) of the north facing cut slope.

8.2.2 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of five percent for a distance of at least ten feet away from the structures to preclude ponding of water adjacent to foundations. Adjacent exterior grades which are paved should be sloped at least 2 percent away from the foundations.

8.2.3 Landscaping after construction should direct rainfall and irrigation runoff away from the structure and not promote ponding of water adjacent to the structures. Care should be taken to maintain a leak-free sprinkler system.

8.2.4 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with low water requirements are recommended.

8.2.5 Perimeter curbs should be extended to the bottom of the aggregate base section, where irrigated landscape areas meet pavements.

8.2.6 It is recommended that landscape planted areas, etc. not be placed adjacent to the building foundations and/or interior slabs-on-grade. Trees should be setback from proposed structures at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

8.2.7 Rain gutters and roof drains should be provided, and connected directly to the site storm drain system. As an alternative, the roof drains should extend a minimum of 5 feet away from the structures and the resulting runoff directed away from the structures.

8.2.8 In general, due to the potential for expansion related heave, or settlement from the introduction of water and long term saturation, stormwater systems which concentrate surface or subsurface water below or adjacent to improvements are not recommended. If stormwater systems which allow wetting of the soils (such as retention or infiltration systems) are required, sufficient setbacks to existing improvements and slopes should be maintained. Alternatively, specific measures such as deepened curbs, cutoffs, liners, etc. could be incorporated in the designs to reduce the potential for excessive settlement of improvements due to moisture and free-water migration from storm water systems. Where onsite stormwater system features that allow wetting of the soils are required for the project by a regulatory agency, these systems should be setback as far as possible from the proposed structures and improvements which are sensitive to settlement. At a minimum, it is recommended that storm water disposal systems which allow wetting of the underling soils be setback at least 30 feet from the proposed building and all foundations.

8.3 Site Preparation

8.3.1 Existing surface and subsurface improvements (including the building, foundations, pavements, canopies, light poles etc.) in the areas of new construction should be excavated and removed from the site and all soils disturbed from the demolition and removal of these improvements should be over-excavated to expose undisturbed soils. Where present, existing utility trench backfill soils should be excavated from within a zone extending from 1 foot below the wall, foundation, or pipe at a 1H to 1V slope to the ground surface. Foundations, walls and utilities lines should be completely removed and disposed of off-site. Excavations to remove existing improvements should extend to at least 12 inches below the bottom of the improvements to be removed or to the depth required to remove all soils disturbed from demolition, whichever is greater. After over-excavation, and prior to backfill, the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted as engineered fill. Any existing deep foundations encountered during the demolition activities should be removed to a depth of at least 5 feet below finished grade, 5 feet below the bottom of foundations and to the depth necessary to allow for installation of the proposed improvements, whichever is deeper.

8.3.2 All surface topsoil, vegetation, trees, roots, organics, surface and subsurface improvements (if any) should be removed from all work areas. The general depth of stripping should be sufficiently deep to remove the root systems and organic top soils. All roots larger than ¼ inch in diameter or any accumulation of organic matter that will result in an organic content more than 3 percent should be removed and not used as engineered fill. The depth of stripping should be reviewed by our firm at the time of construction.

8.3.3 Abundant cobble material was encountered in boring B-4 at a depth of about 5 feet BSG. Where encountered during grading, oversized (cobble) materials (exceeding 3 inches in diameter) should not be used as engineered fill within 36 inches of the final pad grade or for trench backfill. Also, it should be expected that additional effort may be required to excavate these layers or dense gravels and cobbles during mass grading and installation of deeper utilities. Further, if the native soils are to be used as engineered fill, screening of the excavated soils should be anticipated to remove oversize materials that will allow the placement, compaction, and testing of the processed soils and provide uniform support of foundations and floor slabs.

8.3.4 After stripping and removal of the existing surface and subsurface improvements, the proposed building pad area should be over-excavated to at least 2 feet below the pre-construction site grade, 2 feet below the bottom of the proposed foundations, to the depths required to remove all existing surface and subsurface improvements; and to the depth required to remove all undocumented fill and all soils disturbed from demolition, whichever requires the deeper excavation.

The limits of the over-excavation for the building pad should include the footprint of the entire building, all foundations, concrete slabs on grade adjacent to the building, and a minimum of five (5) feet beyond the edges of these improvements and all the foundations. It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for surveying and measuring to verify the horizontal and vertical extent of over-excavation and compaction. The contractor should verify in writing to the owner and Moore Twining that the horizontal and vertical over-excavation limits were completed in conformance with the recommendations of this report, the project plans, and the specifications (the most stringent applies). This verification should be performed by a licensed surveyor and should include a scaled plan showing the "as-graded" limits (i.e., horizontal and vertical extent) in relation to the proposed pad improvements and the elevations of the bottom of the over-excavation. This verification should be provided prior to placing fill and prior to requesting pad certification from Moore Twining or excavating for foundations. Upon approval of the over-excavation limits (horizontal and vertical) by Moore Twining based on survey data by a licensed surveyor provided by the contractor, the soils exposed at the bottom of the excavation should be should be scarified to a minimum depth of 8 inches, aerated or moisture conditioned to between one (1) and four (4) percent above optimum moisture content, and compacted as engineered fill to achieve a stable condition in accordance with the recommendations of this report.

8.3.5 Contractors should be aware that wet soils are anticipated that will likely be significantly above the optimum moisture content required for proper compaction and could require soil drying or chemical treatment for stabilization to achieve the required relative compaction. In addition, measures such as placement of geotextile stabilization fabric and aggregate base may be required in areas of wet soils to achieve stable conditions.

8.3.6 The subgrade below the interior concrete slabs-on-grade within the building pad limits should be underlain by 6 inches of aggregate base compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.3.7 After footing excavations are completed, the moisture content and compaction should be maintained until the reinforcement and concrete are placed.

8.3.8 After stripping and removal of existing improvements and undocumented fills (if encountered), pavement areas, exterior slabs outside the building pad preparation limits and areas to receive fill outside the building pad limits should be prepared by over-excavation to at least 12 inches below the pre-construction subgrade elevation, and to the depth required to remove soils disturbed during the demolition activity, whichever is greater. Following excavation, the exposed subgrade soils shall be scarified to a minimum depth of 8 inches, moisture conditioned to between one (1) and four (4) percent above optimum moisture content and compacted as engineered fill.

8.3.9 For retaining walls and miscellaneous lightly loaded foundations for non-building structures, after stripping and removal of existing improvements and undocumented fills, the native subgrade should be prepared by over-excavation to at least 12 inches below the pre-construction site grade, to the depth required to remove undocumented fill (if any), and to 12 inches below the bottom of the foundations, whichever is deeper. Following excavation, the exposed subgrade soils shall be scarified to a minimum depth of 8 inches, moisture conditioned to between one (1) and four (4) percent above optimum moisture content and compacted as engineered fill. The moisture content of the subgrade soils should be maintained until placement of the aggregate base.

8.3.10 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

8.3.11 The moisture content and density of the compacted soils should be maintained until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.

8.3.12 The Contractor should use appropriate equipment, such as low pressure equipment, to achieve the required over-excavation, compaction and subgrade stabilization to prevent rutting and subgrade instability.

8.3.13 Final grading should produce a building pad and prepared subgrade ready to receive the slab-on-grade which is smooth, planar, and resistant to rutting. Both the finished pad (before aggregate base is placed) and the aggregate base section should not depress more than one-half (½) inch under the wheels of a fully loaded concrete truck. If depressions more than one-half (½) inch occur, the contractor shall perform remedial grading to achieve this requirement at no cost to the Owner.

8.3.14 The Contractor should be responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable. The Contractor should be responsible to perform the tests, assessments, etc. to determine the appropriate method of disposal. In addition, the Contractor is responsible for all costs to dispose of these materials in a legal manner.

8.4 Engineered Fill

8.4.1 Interior and exterior concrete slabs on grade within the building pad preparation limits (which includes the building floor slab and all concrete slabs adjacent to the building) should be supported on a minimum of 6 inches of non-recycled aggregate base over subgrade soils prepared in accordance with Section 8.3 of this report. Exterior concrete slabs-on-grade and PCC paving outside the building pad preparation limits should be supported on a minimum of 6 inches of aggregate base over subgrade soils prepared in accordance with Section 8.3 of this report.

8.4.2 For the building pad and pavement sections, the on-site soils may be used as engineered fill below the recommended aggregate base, provided the soils have an expansion index of 35 or lower, the soils are conditioned/dried to the moisture contents recommended in this report, the soils do not contain more than 3 percent organics, and are processed so the soils do not contain particles larger than 3-inches. Also, if soils with abundant gravels or cobbles are encountered, these materials should be processed such that a minimum of 70 percent passing a 3/4 inch sieve, are free of debris and are properly aerated/moisture conditioned to achieve the recommendations of this report. Screening and crushing of the rock fraction may be required to achieve the gradation requirements for reuse of the onsite soils as engineered fill.

8.4.3 Flyash may not be used for treatment of soils on the project.

8.4.4 If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

8.4.5 The compactability of the native soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, it is recommended that they be evaluated by the contractor during preparation of bids and construction of the project.

8.4.6 Import fill soil (if any) should be non-recycled, non-expansive and granular in nature with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	75 - 100
Percent Passing No. 200 Sieve	10 - 40
Expansion Index (ASTM D4829)	Less than 20
Plasticity Index (ASTM D4318)	Less than 15
Organics	Less than 3 percent by weight
Sulfates	< 0.05 percent by weight
Resistivity	> 3,000 ohm-cm
R-value	≥25

Prior to importing fill, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Owner) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. The Contractor shall pay for the environmental testing required to determine compliance with the requirements of this report. This certification shall consist of, as a minimum, recent analytical data specific to the source of the import material including proper chain-of-custody documentation. Moore Twining will sample and test the material after the environmental certification submittal is approved to verify that the proposed material complies with the geotechnical engineering recommendations of this report. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested for the geotechnical properties.

8.4.7 On-site non-plastic granular soils or imported granular soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between optimum and three (3) percent above optimum moisture content, and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of subgrade below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts of fill should not be placed if the previous lift or subgrade is not stable.

8.4.8 On-site, processed clayey soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between one (1) and four (4) percent above optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of subgrade below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts of fill should not be placed if the previous lift or subgrade is not stable.

8.4.9 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at the minimum frequency listed in Table No. 2, below.

Table No. 2
Minimum In-place Density Test Frequency

Area	**Minimum Test Frequency**
Building Pad	1 test per 2,500 square feet per lift
Pavements	1 test per 5,000 square feet per lift
Utility Pipe and Structure Backfill	1 test per 100 linear feet of trench per compacted lift

8.4.10 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency or pipe manufacturer for use as backfill, or for stabilization of trenches, all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. In addition, periodic slurry cutoffs should be provided along trenches where gravel is placed to reduce potential impacts from groundwater migration through the gravel materials. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Materials such as crushed rock should be placed in thin (less than 8 inches) lifts and each lift should be compacted with a minimum of three (3) passes with a vibratory compactor.

8.4.11 Aggregate base below the building slab should comply with State of California Department of Transportation requirements for a non-recycled Class 2 aggregate base or Crushed Aggregate Base (CAB) from the Standard Specifications for Public Works Construction. The aggregate base used below the building pad should not contain recycled materials. However, a recycled aggregate base may be used for pavement areas outside the building pad, provided that the recycled materials are accepted by the Owner and adequate quality control testing is conducted. Aggregate base should be compacted to a minimum relative compaction of 95 percent. Prior to importing the aggregate base material, the contractor should submit documentation demonstrating that the material meets all the quality requirements (i.e., gradation, R-value, sand equivalent, durability, etc.) for the applicable aggregate base. Documentation should be provided to the Owner, Architect and Moore Twining and reviewed and approved prior to delivery of the aggregate base to the site.

8.5 Foundations

8.5.1 Spread and continuous footings supported on engineered fill soils prepared as recommended in this report may be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads. The weight of the footing may be ignored in design. The building pad should be prepared in accordance with the recommendations included in the "Site Preparation" section of this report.

8.5.2 Perimeter foundations should extend to a minimum depth of 18 inches below the top of the floor slab and the lowest adjacent grade, whichever is deeper. Interior footings should extend to a minimum of 24 inches below the top of the interior floor slab. All footings should have a minimum width of 15 inches, regardless of load.

8.5.3 The foundations should be designed and reinforced for the anticipated settlements and for temperature and shrinkage effects. A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations. Structural deign should be based on a total static settlement of 1 inch and a differential static settlement of ½ inch in 40 feet. In addition, a differential seismic settlement of 1 inch in 40 feet should be considered in design.

8.5.4 The foundations should be continuous around the perimeter of the structure to reduce moisture migration beneath the structure. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.

8.5.5 Site lighting (if any) may be supported on a drilled-cast-in-hole reinforced concrete foundations (piers). An allowable skin friction of 150 pounds per square foot per foot of embedment may be used to resist axial loads. Lateral load resistance may be estimated using the CBC non-constrained design. A value of 150 pounds per square foot per foot of depth may be used.

8.5.6 At the time of pier construction and until the concrete is placed, the shaft excavation should have stable sidewalls and all sloughed soil should be completely removed from the bottom of the excavation. If the drilled hole exhibits instability, it should be cased.

8.5.7 Moore Twining should observe the bottom of foundation excavations prior to the placement of reinforcing steel and utilities. The Contractor shall provide a minimum of 48 hours notice for these observations.

8.6 Seismic Design Factors

The following seismic factors were developed for the site using the Ground Motion Parameter Calculator provided by SEOAC and OSHPD (http://seismicmaps.org), based upon a site latitude of 32.7667 degrees and a site longitude of -117.1460 degrees. The data provided in Table No. 3 are based upon the procedures of Sections 1613.2.1 through 1613.2.4 of the 2019 California Building Code, ASCE 7-16 Chapter 11 and Supplement No. 1. The data in Table No. 3 were not determined based upon a ground motion hazard analysis. The structural engineer should review the values in Table No. 3 and determine whether a ground motion hazard analysis is required for the project considering the seismic design category, structural details, and requirements of ASCE 7-16 (Section 11.4.8 and other applicable sections). If required, Moore Twining should be notified and requested to conduct the additional analysis, develop updated seismic factors for the project, and update the following values.

Table No. 3
Seismic Design Parameters

Item	CBC Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.624
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration ASCE 7-16 (PGA)	0.568
Spectral Response At Short Period (0.2 Second), Ss	1.258
Spectral Response At 1-Second Period, S_1	0.433
Site Coefficient (based on Spectral Response Short Period), Fa	1.0
Site Coefficient, (based on Spectral Response 1-Second Period) Fv	See Note 1
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	1.258
Maximum considered earthquake spectral response acceleration for 1-second period, S_{M1}	See Note 1
Five percent damped design spectral response acceleration for short period, S_{DS}	0.839

Note 1: Requires ground motion hazard analysis per ASCE Section 21.2 (ASCE 7-16, Section 11.4.8), unless the structural engineer determines that an exception of Section 11.4.8 of ASCE 7-16 is applicable for the project design.

8.7 Interior Concrete Slabs-on-Grade

8.7.1 The recommendations provided herein are intended only for the design of concrete slabs on grade within the building pad and their proposed uses, which do not include construction loading. The building contractor should assess the slab section and determine its adequacy to support any proposed construction loading.

8.7.2 A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed floor slab. Concrete slabs on grade supported on the aggregate base and subgrade soils prepared as recommended in this report should be designed for a total settlement and heave of 1 inch total and ½ inch differential over 40 feet.

8.7.3 Concrete slabs on grade within the building pad should be supported on a minimum of 6 inches of non-recycled Class 2 aggregate base placed over subgrade soils prepared as indicated in Section 8.3 of this report. The minimum thickness of AB is recommended directly below the slabs-on-grade to improve the slab support characteristics and for construction stability purposes.

8.7.4 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

8.7.5 The moisture content of the subgrade below the aggregate base section should be verified to be in compliance with the recommendations for engineered fill within 48 hours prior to placing the overlying layer.

8.7.6 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarder when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The vapor retarder should overlay the compacted aggregate base. It should be noted that placing the PCC slab directly on the vapor barrier will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the aggregate base and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with the current

ASTM C 755, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to the current ASTM E 154 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier selection and installation conform to the current ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R), Addendum, Vapor Retarder Location and current ASTM E 1643, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of the floor covering and floor covering adhesive be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

8.7.7 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with the manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be caulked per manufacturer's recommendations.

8.7.8 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations.

8.7.9 The moisture retarding membrane is not required beneath exposed concrete floors, such as warehouses and garages, provided that moisture intrusions into the structure are permissible for the design life of the structure.

8.7.10 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.52 lb./lb. or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or pavements adjacent to the structure, 4) providing adequate drainage away from the structure, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structure.

8.7.11 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity of the floor slab, etc., at a frequency and method as specified by the flooring manufacturer, adhesive manufacturer, underlayment manufacturer, etc. or as required by the plans and specifications, whichever is most stringent. The tests should be conducted in accordance with the applicable ASTM test methods. The results of vapor transmission tests, pH tests, internal relative humidity tests of the floor slab, ambient building conditions, etc. should be within floor manufacturer's, adhesive manufacturer's and underlayment manufacturer's specifications at the time the floor is placed. It is recommended that the floor, adhesive and underlayment manufacturers and subcontractor review and approve the test data prior to floor covering installation.

8.7.12 To reduce the potential for damaging slabs during construction the following recommendations are presented: 1) use perimeter pour-strips at tilt-wall locations to avoid damage to slab-wall connections; 2) design for a differential slab movement of ½ inch relative to interior columns; 3) provide aggregate base below the slabs, 4) it is expected that erection of concrete tilt-up wall panels and roof steel may require cranes. The loaded track and/or pad pressure of any crane which will operate on slabs or pavements should be evaluated by the contractor prior to loading the slab.

8.7.13 For tilt up construction, a perimeter pour strip between the wall footing and the adjacent interior slab should be incorporated into the project design. After the walls are erected and a majority of the differential movement has occurred, the pour strip should be placed.

8.7.14 Backfill the zone above the top of footings at interior column locations, building perimeters, and below the bottom of slabs with an approved backfill and/or an aggregate base section as recommended herein for the area below interior slabs-on-grade. This procedure should provide more uniform support for the slabs which may reduce the potential for cracking.

8.7.15 If the pad subgrade or the aggregate base will be used as a working surface, the Contractor should determine an adequate aggregate base section thickness for the type and methods of construction proposed for the project. The proposed compacted subgrade can experience instability under construction loading.

8.8 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, rather lightly loaded sidewalks, curbs, and planters, etc.

8.8.1 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with recommendations presented in this report for interior slabs-on-grade. Moore Twining can provide alternative design recommendations for exterior slabs, if requested.

8.8.2 Subgrade soils for exterior slabs should be prepared as recommended in the "Site Preparation" section of this report. Upon completion of the over-excavation and compaction of subgrade soils, the exterior slabs adjacent to the building should be supported on 6 inches of aggregate base over subgrade soils prepared in accordance with the recommendations provided in the "Site Preparation" section of this report. Exterior slabs on grade that are not located adjacent to the building (i.e., outside of the building pad limits defined in this report) should be supported on 6 inches of aggregate base placed over subgrade soils prepared in accordance with the recommendations provided in the "Site Preparation" section of this report.

8.8.3 The moisture content of the subgrade soils should be verified to be in compliance with the recommendations for engineered fill within 48 hours of placement of the slab-on-grade. In addition, the density and stability of the prepared subgrade should be verified prior to placement of the aggregate base. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.8.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend to at least a depth of 6 inches below the bottom of the slabs-on-grade.

8.8.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing concrete walks and finish work over dry or slightly moist subgrade should be avoided. It is recommended that the general contractor notify Moore Twining to conduct in-place moisture and density tests prior to placing concrete flatwork. Written test results indicating passing density and moisture tests should be in the general contractor's possession prior to placing concrete for exterior flatwork.

8.9 Asphaltic Concrete (AC) Pavements

8.9.1 Areas for AC pavement should be prepared in accordance with the recommendations section entitled, "Site Preparation." The upper 12 inches of subgrade beneath the aggregate base should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.9.2 The following pavement sections are based on an R-value of 5, a minimum asphalt concrete thickness of 3 inches and traffic index values ranging from 5.0 to 8.0. It should be noted that if pavements are constructed prior to the building construction, the traffic index value should account for construction traffic. The actual traffic index values applicable to the site should be determined by the project civil engineer.

Table No. 4
Two-Layer Asphaltic Concrete Pavements

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
5.0	3.0	10.0	12
5.5	3.0	11.5	12
6.0	3.0	13.5	12
6.5	3.5	14.5	12
7.0	4.0	15.5	12
7.5	4.0	17.5	12
8.0	4.5	18.5	12

AC - Asphaltic Concrete compacted in accordance with Section 8.9.7 of this report
AB - Class 2 aggregate base, CAB, or CMB compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Minimum depth of subgrade soils prepared and compacted in accordance with the recommendations in the Site Preparation section of this report.

8.9.3 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

8.9.4 If the actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions. If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement sections should be re-evaluated for the anticipated traffic.

8.9.5 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

8.9.6 Pavement materials and construction method should conform to the current State of California Standard Specifications.

8.9.7 The asphaltic concrete, including the joint density, should be compacted to an average relative compaction of 93 percent, with no single test value being below a relative compaction of 91 percent and no single test value being above a relative compaction of 97 percent of the referenced laboratory density according to ASTM D2041.

8.10 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural sections are presented in the following subsections. The PCC pavement design assumes a minimum modulus of rupture of 500 psi for the Portland cement concrete. It is recommended that PCC pavements have a minimum compressive strength of 3,500 pounds per square inch.

8.10.1 The subgrade soils for Portland cement concrete pavements should be over-excavated and compacted as recommended in the "Site Preparation" section of the recommendations in this report. The moisture content of the upper 12 inches of subgrade soils (engineered fill) below the slabs-on-grade should be confirmed to be in compliance with the recommendations of this report by testing within 48 hours prior to placement of the slab-on-grade. The Contractor should obtain written confirmation of in-situ moisture and density test results from Moore Twining prior to pouring the slab.

8.10.2 Final grading should produce a compacted subgrade which is smooth, planar, and resistant to rutting. Proof rolling of the finished subgrade should be conducted to assess stability prior to slab construction. The finished subgrade shall not depress more than one-half (½) inch under the wheels of a fully loaded water truck, or equivalent loading.

8.10.3 The following PCC pavement section thicknesses were prepared based on a design k-value of 110 psi/in for the subgrade soils and traffic index values ranging from 5.0 to 8.0. The design thicknesses were prepared based on the procedures outlined in the Portland Cement Association (PCA) document, "Thickness Design for Concrete Highway and Street Pavements," assuming the following: 1) minimum modulus of rupture of 500 psi for the concrete, 2) load transfer by aggregate interlock or dowels, 3) a concrete shoulder, 4) a load safety factor of 1.1, 5) vehicular loading only for a traffic index of 5.0 and 6) truck loading consisting of 1 single axle load of 20 kips and one tandem axle load of 35 kips for traffic indices of 5.0, 6.0, 7.0, and 8.0.

Table No. 5
Portland Cement Concrete Pavement Sections

Traffic Index	ADTT (Trucks/day)	PCC thickness (inches)	AB thickness (inches)	Compacted Subgrade[2] (inches)
5.0[1]	N/A[1]	5.0	6.0	12.0
5.0	0.4	7.0	6.0	12.0
6.0	1.6	7.0	6.0	12.0
7.0	6	7.5	6.0	12.0
8.0	19	8.0	6.0	12.0

1 - Passenger Vehicular Loading Only
2 - Subgrade soils compacted to at least 95 percent relative compaction at a minimum moisture content of optimum moisture (ASTM D-1557)

8.10.4 The PCC pavement should be constructed in accordance with American Concrete Institute requirements, the requirements of the project plans and specifications, whichever is the most stringent. The pavement design engineer should include appropriate construction details and specifications for construction joints, contraction joints, joint filler, concrete specifications, curing methods, etc.

8.10.5 Other than load transfer at joints, there are no special geotechnical engineering design requirements for reinforcement of exterior concrete slabs. However, the use of temperature and shrinkage steel may be desirable to reduce the potential for shrinkage cracking in areas of PCC paving which are also used as paths of travel for pedestrians. The final design details and specifications should be determined by the applicable design consultant.

8.10.6 Concrete used for PCC pavements shall possess a minimum flexural strength (modulus of rupture) of 500 pounds per square inch. A minimum compressive strength of 3,500 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, curing methods, etc. should be provided by the designer of the PCC slabs.

8.10.7 The pavement section thickness design provided above assumes the design and construction will include sufficient load transfer at construction joints. Coated dowels or keyed joints are recommended for construction joints to transfer loads. The joint details should be detailed by the pavement design engineer and provided on the plans.

8.10.8 Exposed contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade soils. The type of joint filler should be specified by the pavement designer. The joint sealer and filler material should be maintained throughout the life of the pavement.

8.10.9 Contraction joints should have a depth of at least one-fourth the slab thickness, e.g., 1.5-inch for a 6-inch slab. Specifications for contraction joint spacing, timing and depth of sawcuts should be included in the plans and specifications.

8.10.10 Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.

8.10.11 Joint spacing should be in accordance with an accepted standard such as the ACI Concrete Manual of Concrete Practice. However, regardless of slab thickness, joint spacing should not exceed 15 feet.

8.10.12 Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.

8.10.13 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas.

8.10.14 Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

8.11 Temporary Excavations

8.11.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. The contractor is responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades, classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures.

8.11.2 Temporary excavations should be constructed in accordance with CAL OSHA requirements. Temporary cut slopes should not be steeper than 1.5:1, horizontal to vertical, and flatter if possible. If excavations cannot meet these criteria, the temporary excavations should be shored.

8.11.3 In no case should excavations extend below a 1.5H to 1V zone below utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 1.5H to 1V envelope should be shored to support the soils, foundations, and slabs.

8.11.4 Shoring should be designed by an engineer with experience in designing shoring systems and registered in the State of California. Moore Twining should be provided with the shoring plan to assess whether the plan incorporates the recommendations in the geotechnical report.

8.11.5 Excavation stability should be monitored by the contractor. Slope gradient estimates provided in this report do not relieve the contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the structure occurs, during or after excavation, the owners and Moore Twining should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury.

8.12 Utility Trenches

8.12.1 The utility trench subgrade should be prepared by excavation of a neat trench without disturbance to the bottom of the trench. If sidewalls are unstable the Contractor shall either slope the excavation to create a stable sidewall or shore the excavation. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 92 percent relative compaction prior to placement of bedding material. The Contractor is responsible for notifying Moore Twining when these conditions occur and arrange for Moore Twining to observe and test these areas prior to placement of pipe bedding. The Contractor shall use such equipment as necessary to achieve a smooth undisturbed native soil surface at the bottom of the trench with no loose material at the bottom of the trench. The Contractor shall either remove all loose soils or compact the loose soils as engineered fill prior to placement of pipe and backfill of the trench.

8.12.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The bottom of the trench should be compacted as engineered fill prior to placement of the pipe bedding. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported, non-expansive materials moisture conditioned and compacted as engineered fill. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.12.3 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand should be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321 listed in the table below (minimum manufacturer requirements). As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Table No. 4
Minimum Trench Widths for HDPE Pipe with
Select Sand Bedding Initial Backfill

Inside Diameter of HDPE Pipe (inches)	Outside Diameter of HDPE Pipe (inches)	Minimum Trench Width (inches) per ASTM D2321
12	14.2	30
18	21.5	39
24	28.4	48
36	41.4	64
48	55	80
60	67.3	96

8.12.4 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to reduce the potential for migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining.

8.12.5 Utility trench backfill should be moisture conditioned and compacted as engineered fill. The Contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

8.12.6 On-site soils and approved imported engineered fill may be used as final backfill in trenches.

8.12.7 Jetting of trench backfill is not allowed to compact the backfill soils.

8.12.8 Where utility trenches extend from the exterior to the interior limits of a building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

8.12.9 Storm drains and/or utility lines should be designed to be watertight. If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil heave causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. It is recommended that the pipelines, stormwater, sewer, water, retaining wall drains, etc. be inspected by video inspection prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are watertight. The Contractor shall provide to the Owner and Moore Twining a copy of video tape and a written description of the pipe condition prepared by the video inspection firm prior to placement of improvements above the utilities. In addition, the Contractor is required to inspect and test the utility lines as required by the pipe manufacturer and governing agencies.

8.12.10 Utility trenches should be a minimum of 24 inches in width to allow for in-place density testing by traditional (nuclear density test) methods and the backfill should be compacted in accordance with the recommendations for engineered fill.

8.12.11 Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 1.5 horizontal to 1 vertical downward from the bottom of building foundations.

8.12.12 The project Civil Engineer should include slurry type cutoff collars along utility trenches at critical locations to prevent the surface water and groundwater from draining along the trench backfill/bedding material.

8.13 <u>Corrosion Protection</u>

8.13.1 Based on the National Association of Corrosion Engineers corrosion severity rating listed in Section 6.11 of this report, the analytical results of sample analyses indicate a "highly corrosive" corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on these conditions. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

8.13.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentration of sulfates determined for the near-surface soils (negligible exposure). According to provisions of ACI 318, section 4.3, the sulfate concentration falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. Therefore, no restrictions are required regarding the type, water-to-cement ratio, or strength of the concrete used for foundation and slabs due to the sulfate content. However, a low water to cement ratio is recommended for slabs on grade as recommended for exposed concrete slabs to reduce shrinkage.

8.13.3 These soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Moore Twining is not a corrosion engineer; thus, cannot provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

9.0 DESIGN CONSULTATION

9.1 Moore Twining should be retained to review those portions of the contract drawings and specifications that pertain to earthwork operations and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not part of this current contractual agreement.

9.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

9.3 If Moore Twining is not retained for the plan review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

10.0 CONSTRUCTION MONITORING

10.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.

10.2 Moore Twining can conduct the necessary observation and field testing to provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing and conclusions will be provided regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

10.3 In the event that the earthwork operations for this project are conducted such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) it is recommended that the exposed subgrade that will receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.

10.4 The construction monitoring is an integral part of this investigation. This phase of the work provides Moore Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

10.5 If Moore Twining is not retained to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

10.6 Upon the completion of work, a final report should be prepared by Moore Twining. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to prepare a final report summarizing the observations during site preparation activities relative to the recommendations of this report. This service is not, however, part of this current contractual agreement.

11.0 NOTIFICATION AND LIMITATIONS

11.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of subsurface variations between borings may not become evident until construction.

11.2 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

11.3 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (over 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

11.4 Changed site conditions, or relocation of proposed structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

11.5 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction section of this report. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in this report is not recommended. The entity or entities that use or cause to use this report or any portion thereof for other structures or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

11.6 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

11.7 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

11.8 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

11.9 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

We appreciate the opportunity to be of service to Cushman & Wakefield on this project. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience at (800) 268-7201.

Sincerely,
MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

REGISTERED GEOLOGIST
KENNETH JAMES CLARK
No EG 1884
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA
EXP 5-31-15

Kenneth J. Clark, CEG
Senior Engineering Geologist

Read L. Andersen, RGE
Manager

REGISTERED PROFESSIONAL ENGINEER
READ L. ANDERSEN
2810
EXP. 12-31-20
GEOTECHNICAL
STATE OF CALIFORNIA

List of References

California Division of Mines and Geology (CDMG) Open File report 95-03 titled Landslide Identification Map No. 33; Landslide Hazards in the Southern Part of the San Diego Metropolitan Area by Tan (1995).

CDMG Bulletin 200 - "Geology of the San Diego Metropolitan Area," prepared by Michael P. Kennedy and the California Division of Mines and Geology, (1975)

City of San Diego, Guidelines for Geotechnical Reports, (2018)

City of San Diego, Seismic Safety Study, "Geologic Hazards and Faults" (Grid 21 of the Hazard Map Series) dated April 3, 2008.

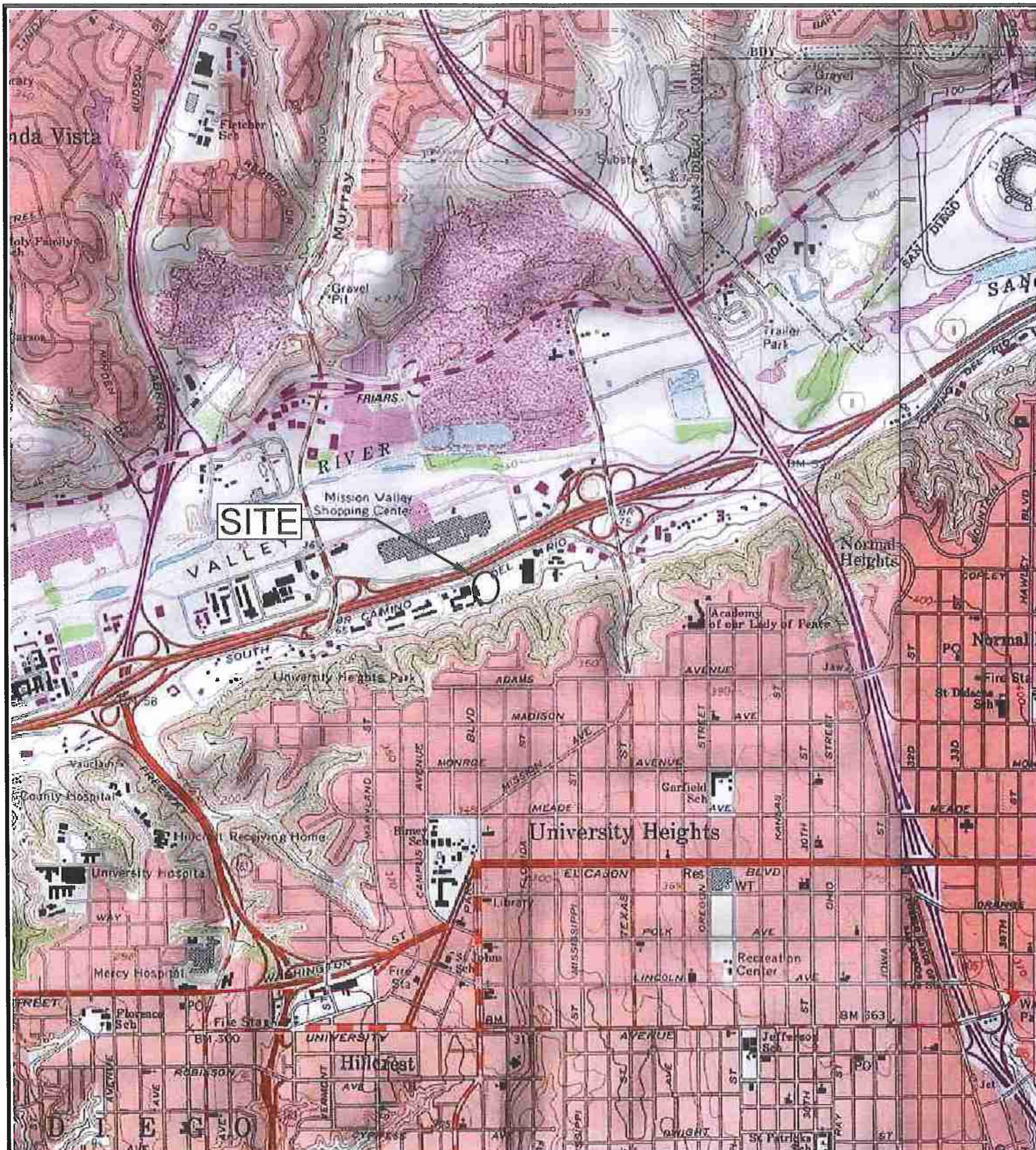
City of San Diego, Storm Water Standard - prepared by Geosyntec, effective date October 1, 2018

EDR - Aerial Photo Decade Package for the years 1928, 1949, 1953, 1964, 1966, 1970, 1979, and 1985 of the site (1561 Camino Del Rio South)

Geologic Map of the San Diego 30'x60' Quadrangle, California, Regional Geologic Map Series, prepared by the California Geological Survey and compiled by Michael P. Kennedy and Siang S. Tan, dated 2008

APPENDIX A

DRAWINGS

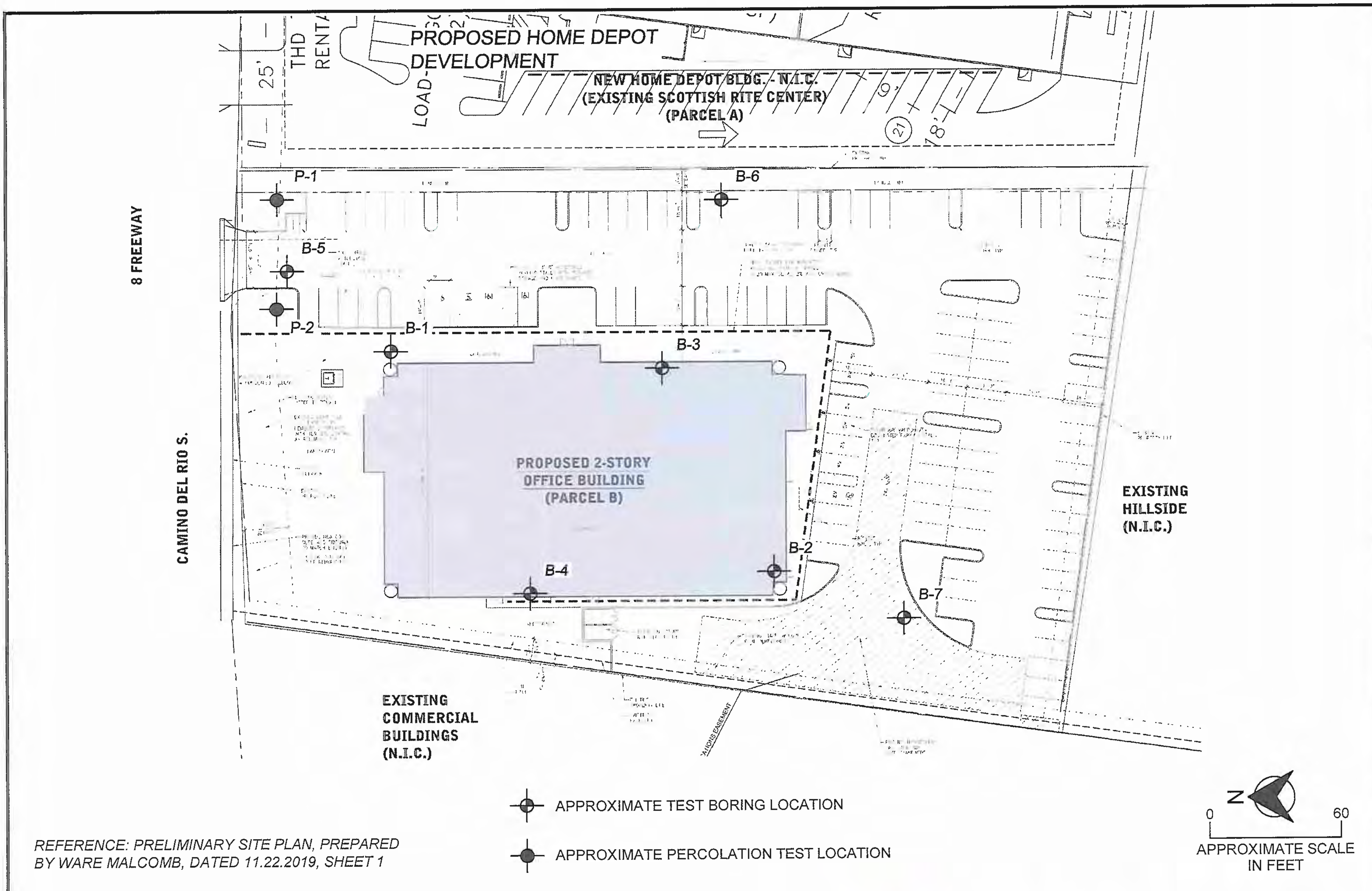


SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
LA JOLLA, CALIFORNIA QUADRANGLE 1996

0 2000
APPROXIMATE SCALE IN FEET

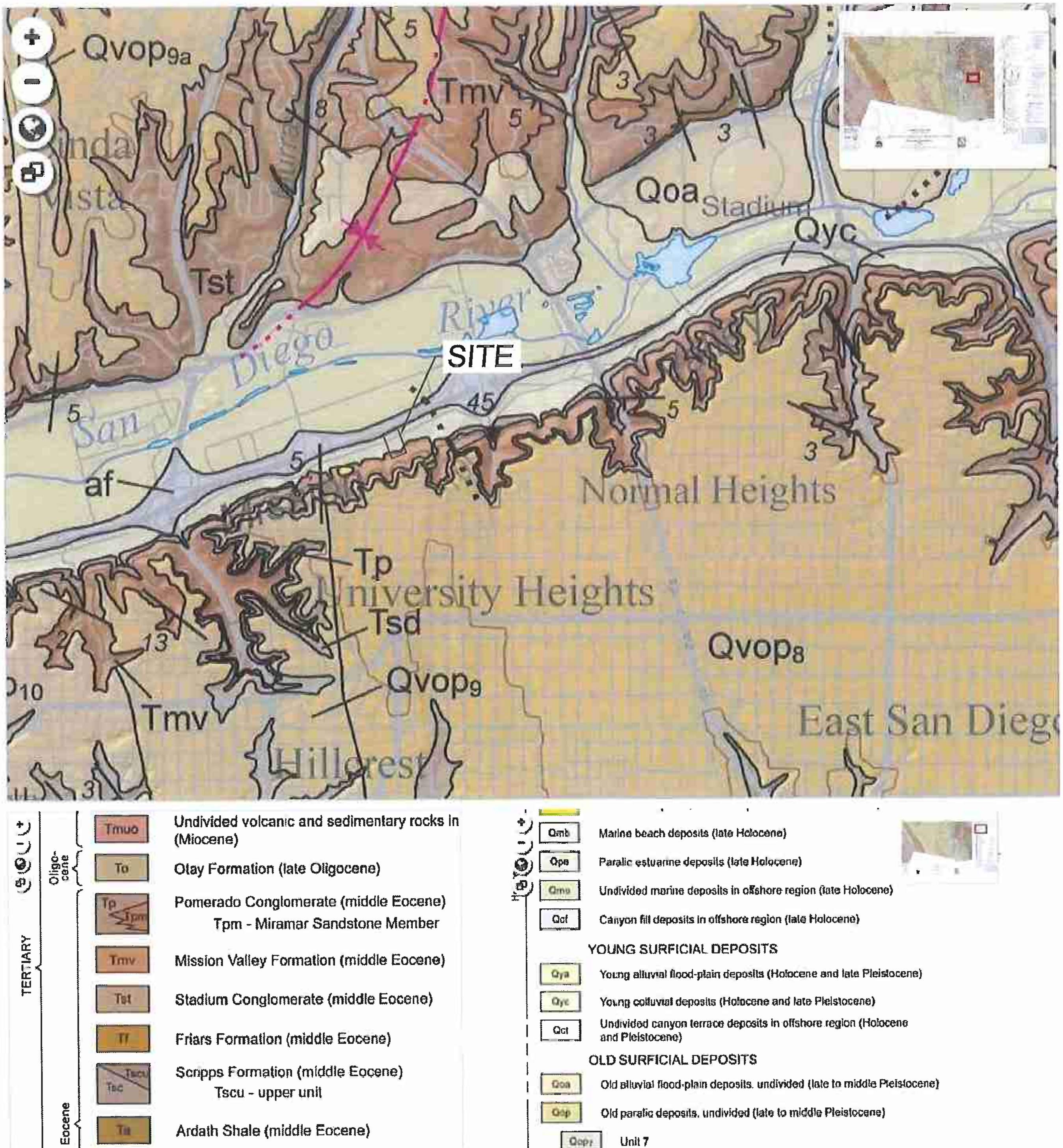
SITE LOCATION MAP PROPOSED 2-STORY OFFICE BUILDING SCOTTISH RITE CORPORATE BUSINESS CENTER 1561 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO.: 84101-01-01	DATE: 01/13/20
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. G84101.01	DRAWING NO. 1

MOORE TWINING ASSOCIATES, INC.



REFERENCE: PRELIMINARY SITE PLAN, PREPARED BY WARE MALCOMB, DATED 11.22.2019, SHEET 1

TEST BORING LOCATION MAP PROPOSED 2-STORY OFFICE BUILDING SCOTTISH RITE CORPORATE BUSINESS CENTER 1561 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 84101-01-02	DATE DRAWN: 01/13/20	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. G84101.01	DRAWING NO. 2	



GEOLOGIC MAP OF THE SAN DIEGO 30' x 60' QUADRANGLE, CALIFORNIA

By

Michael P. Kennedy[1] and Siang S. Tan[1]

2008

Digital preparation by

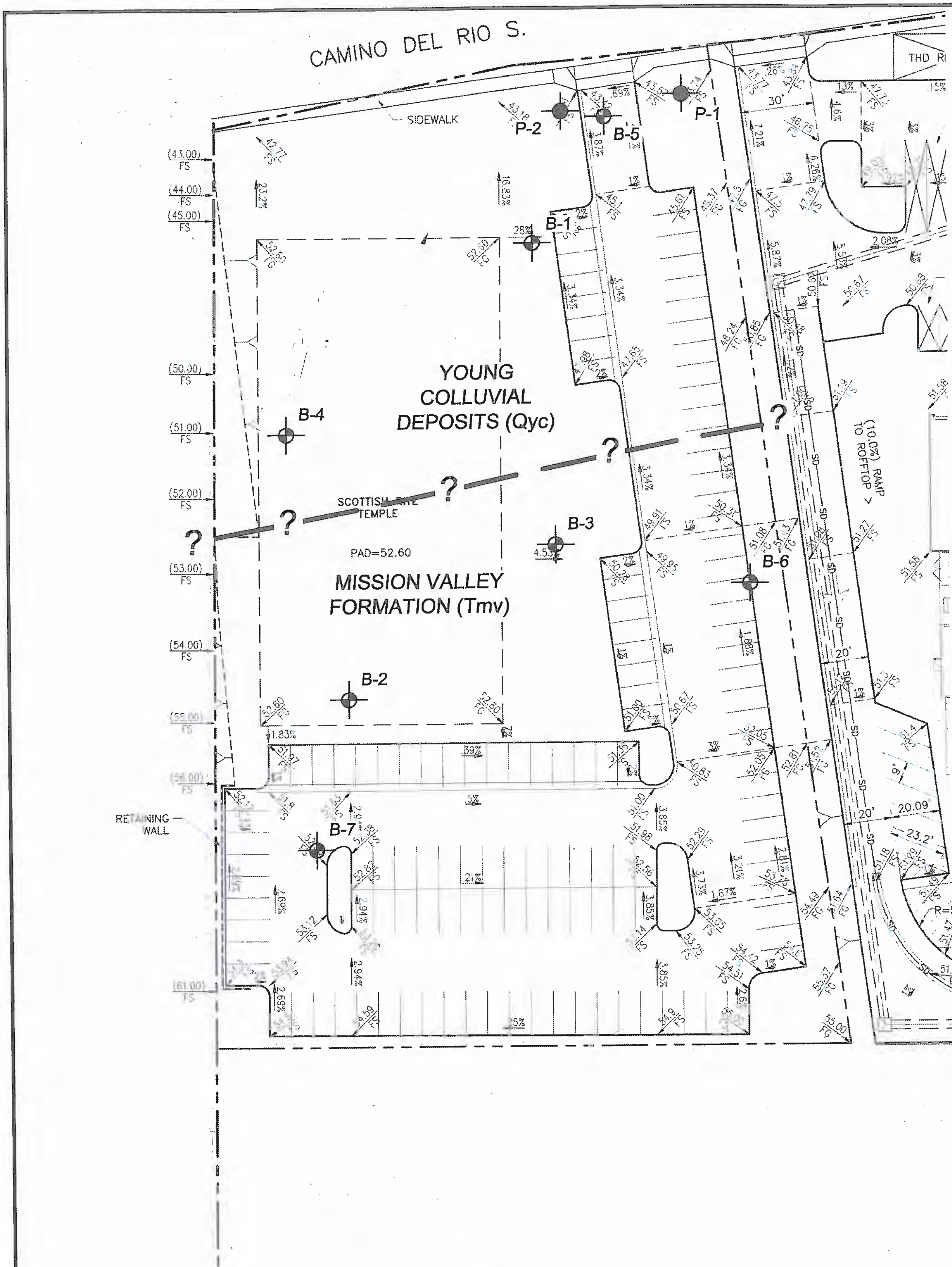
Kelly R. Bovard[2], Anne G. Garcia[2], Diane Burns[2], and Carlos I. Gutierrez[1]

N

0 3500

APPROXIMATE SCALE IN FEET

SITE GEOLOGIC MAP PROPOSED 2-STORY OFFICE BUILDING SCOTTISH RITE CORPORATE BUSINESS CENTER 1561 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 84101-01-02	DATE DRAWN: 01/13/20	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. G84101.01	DRAWING NO. 3	



REFERENCE: *CONCEPTIUAL GRADING PLAN SHEET 2 of 2, DATED FEBRUARY 15, 2019, PREPARED BY* SAN DIEGUITO ENGINEERING INC.

APPROXIMATE TEST BORING LOCATION

APPROXIMATE PERCOLATION TEST LOCATION

N

0 60

APPROXIMATE SCALE IN FEET

SITE GEOLOGIC MAP PROPOSED 2-STORY OFFICE BUILDING SCOTTISH RITE CORPORATE BUSINESS CENTER 1561 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 84101-01-02	DATE DRAWN: 01/13/20	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. G84101.01	DRAWING NO. 4	

APPENDIX B

LOGS OF BORINGS

This appendix contains the final logs of borings. These logs represent our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these test boring locations. Also, the passage of time may result in changes in the soil conditions at these test boring locations.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Scottish Rite Building

Project Number: G84101.01

Drilled By: Baja Exploration

Drill Type: CME-95

Auger Type: 8-inch hollow-stem

Hammer Type: 140 lb auto-trip

Logged By: JC

Date: December 17, 2019

Elevation: 45 feet AMSL (Approx.)

Depth to Groundwater First Encountered During Drilling: 29 feet BSG

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
45 / 0		SC	3.25 inches of AC over 1.25 inches of AB			
	26/6 14/6 7/6		Clayey Sand: medium dense, damp, fine grained, light brown, with fine gravel and trace of clay	-#200=35.9% SAND=54.2% +#4=9.9% DD=112.9 pcf	21	14.3
40 / 5	13/6 27/6 41/6		At 3.5 to 5 feet: 4 inch dia. cobble in sample shoe*		68*	10.8
35 / 10	3/6 3/6 4/6	SM	Silty Sand: moist, fine grained, dark brown loose		7	
30 / 15	4/6 5/6 6/6		medium dense	-#200=30.8% SAND=69.2% +#4=0.0%	11	
25 / 20	3/6 3/6 6/6	ML	Sandy Silt, stiff, very moist, fine grained, dark brown	-#200=61.1% SAND=38.9% +#4=0.0% PI=10 LL=36	9	
20 / 25	2/6 2/6 3/6	CL	Sandy Lean Clay: medium stiff, very moist, dark brown, medium plasticity	-#200=58.4% SAND=41.6% +#4=0.0% PI=14 LL=37	5	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Scottish Rite Building

Project Number: G84101.01

Drilled By: Baja Exploration

Drill Type: CME-95

Auger Type: 8-inch hollow-stem

Hammer Type: 140 lb auto-trip

Logged By: JC

Date: December 17, 2019

Elevation: 45 feet AMSL (Approx.)

Depth to Groundwater First Encountered During Drilling: 29 feet BSG

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
15 — 30	2/6 7/6 9/6	ML	stiff Sandy Silt: stiff, wet, brown		16	
	2/6 2/6 4/6		medium stiff	-#200=51.1% SAND=48.9% +#4=0.0%	6	
10 — 35	4/6 4/6 6/6	ML	medium plasticity	-#200=64.0% SAND=36.0% +#4=0.0% PI=11 LL=36	10	
	9/6 15/6 15/6				30	
		SM	Silty Sand: medium dense, very fine grained, brown			
5 — 40	6/6 8/6 12/6				20	
	9/6 9/6 9/6		interbedded with lean clay, very fine grained, brown		18	
0 — 45	3/6 3/6 5/6	CL	Sandy Lean Clay: medium stiff, medium plasticity, wet, dark gray	PI=18 LL=37	8	
	9/6 12/6 15/6				27	
		SM	Silty Sand: medium dense, wet, fine grained, brown			
-5 — 50	11/6 14/6 20/6		dense		34	
			Bottom of boring at 50 feet BSG			
-10 — 55						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-2

Project: Scottish Rite Building

Project Number: G84101.01

Drilled By: Baja Exploration

Drill Type: CME-95

Auger Type: 8-inch hollow-stem

Hammer Type: 140 lb auto-trip

Logged By: JC

Date: December 17, 2019

Elevation: 51 feet AMSL (Approx.)

Depth to Groundwater First Encountered During Drilling: 29 feet BSG

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
51 / 0			3.1 inches of AC over 1.4 inches of AB			
50	11/6 14/6 12/6	SM	Silty Sand: medium dense, damp, fine to medium grained, light brown, with fine gravel, cobble encountered at 1.5 feet BSG		26	3.2
5 / 45	19/6 11/6 18/6	CL-ML	Silty Clay: very stiff, damp, low plasticity, brown	DD=97.2 pcf ø=32° C=280 psf	29	21.3
10 / 40	9/6 8/6 8/6		interbedded with clayey sand, fine grained, brown		16	
15 / 35	5/6 10/6 11/6	ML	Sandy Silt, stiff, very moist, dark brown, with layers of silty sand		21	
		CL	Sandy Lean Clay: damp, fine sand, brown, with fine and coarse gravel, medium plasticity			
20 / 30	5/6 7/6 10/6		very stiff		17	
25	5/6 8/6 11/6		dark brown, trace of fine gravel	-#200=55.7% SAND=38.1% +#4=6.2% PI=14 LL=37	19	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Scottish Rite Building

Project Number: G84101.01

Drilled By: Baja Exploration

Drill Type: CME-95

Auger Type: 8-inch hollow-stem

Hammer Type: 140 lb auto-trip

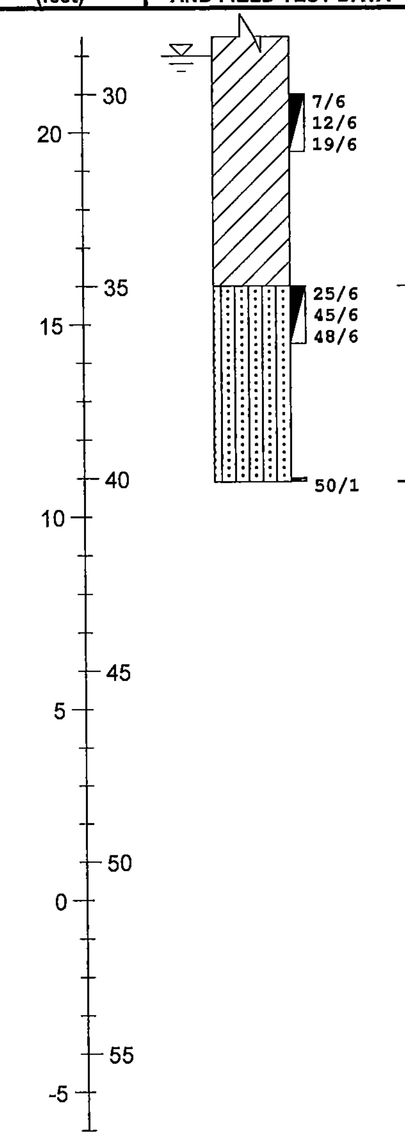
Logged By: JC

Date: December 17, 2019

Elevation: 51 feet AMSL (Approx.)

Depth to Groundwater First Encountered During Drilling: 29 feet BSG

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
	7/6 12/6 19/6		very stiff, brown		31	
			very hard drilling			
	25/6 45/6 48/6	SM	Silty Sand: very dense, wet, fine to coarse grained, brown		93	
	50/1		Auger and sample refusal at approx. 40.1 feet BSG		>50	



Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-3

Project: Scottish Rite Building
Project Number: G84101.01
Drilled By: Baja Exploration
Drill Type: CME-95
Auger Type: 8-inch hollow-stem
Hammer Type: 140 lb auto-trip

Logged By: JC
Date: December 18, 2019
Elevation: 49.5 feet AMSL (Approx.)
Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		SM	2.6 inches of AC over 1.1 inches of AB			
	7/6 11/6 11/6		Silty Sand: dry, reddish brown, fine grained, with trace of clay		22	2.7
45 / 5	10/6 12/6 14/6		At 2 feet BSG: medium dense, fine to coarse grained, light brown damp, trace of clay	DD=113.1 pcf ø=32° C=240 psf	26	3.3
40 / 10	6/6 13/6 13/6	SC	Clayey Sand: interebedded with silty sand, medium dense, damp, fine grained, light brown		26	
35 / 15	8/6 8/6 6/6	SM	Silty Sand: medium dense, damp, fine grained, brown, with clay		14	
30 / 20	6/6 14/6 17/6		interbedded with clayey sand, dense, damp, fine grained, light brown to brown		31	
			Bottom of boring at 20 feet BSG			
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-4

Project: Scottish Rite Building

Project Number: G84101.01

Drilled By: Baja Exploration

Drill Type: CME-95

Auger Type: 8-inch hollow-stem

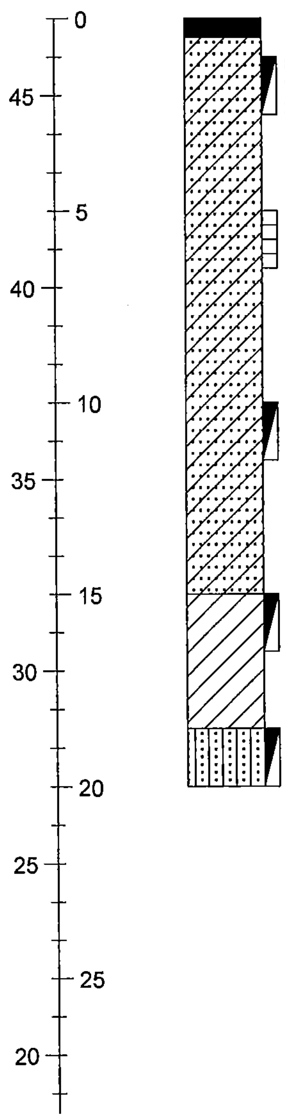
Hammer Type: 140 lb auto-trip

Logged By: JC

Date: December 18, 2019

Elevation: 47 feet AMSL (Approx.)

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			3 inches of AC over 2.25 inches of AB			
	8/6 7/6 11/6	SC	Clayey Sand: damp, light brown, fine grained, with fine gravel and trace of coarse gravel		18	9.8
5	42/6 18/6 21/6		very rocky drilling, increase in coarse gravel with fractured cobbles in cuttings	No recovery	39	18.4
10	3/6 3/6 4/6		interbedded with silty sand, loose, damp, fine grained, light brown		7	
15	2/6 3/6 4/6	CL	Sandy Lean Clay: medium stiff, damp, medium plasticity. dark gray		7	
	6/6 6/6 5/6	SM	Silty Sand: medium dense. damp, gray brown, fine grained, trace of clay		11	
20			Bottom of boring at 20 feet BSG			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-5

Project: Scottish Rite Building

Project Number: G84101.01

Drilled By: Baja Exploration

Drill Type: CME-95

Auger Type: 8-inch hollow-stem

Hammer Type: 140 lb auto-trip

Logged By: JC

Date: December 18, 2019

Elevation: 52.5 feet AMSL (Approx.)

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			3.1 inches of AC over 1.25 inches of AB			
50	8/6 8/6 5/6	SM	Silty Sand: damp to moist, gray brown, fine to medium grained, with trace fine and coarse gravel and trace of clay medium dense		13	
5	2/6 4/6 10/6			No recovery Cobble in sampler	14	17.0
45		CL	Sandy Lean Clay: soft (est.), moist, medium plasticity, brown At 8 feet: black, damp			
10	7/6 4/6 4/6	SM	Silty Sand: loose, damp, fine grained, dark brown, trace of clay		8	
40						
15	5/6 4/6 4/6				8	
35	2/6 2/6 4/6	CL	Sandy Lean Clay: interbedded with clayey sand, medium stiff, moist, low to medium plasticity, brown		6	
20			Bottom of boring at 20 feet BSG			
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Scottish Rite Building
Project Number: G84101.01
Drilled By: Baja Exploration
Drill Type: CME-95
Auger Type: 8-inch hollow-stem
Hammer Type: 140 lb auto-trip

Logged By: JC
Date: December 18, 2019
Elevation: 50 feet AMSL (Approx.)
Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0			3.8 inches of AC over 1 inch of AB	At 0.5 to 3': R=20 SR=1,868 ohm-cm Cl<0.0006% SS=0.0024% pH=8.9		
		SC	Clayey Sand: damp, brown, fine grained, with fine and coarse gravel and trace of clay			
		SM	Silty Sand, damp, fine grained, light brown, trace of clay			
			increase in clay content, brown			
45 / 5			Bottom of boring at 5 feet BSG			
40 / 10						
35 / 15						
30 / 20						
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-7

Project: Scottish Rite Building

Project Number: G84101.01

Drilled By: Baja Exploration

Drill Type: CME-95

Auger Type: 8-inch hollow-stem

Hammer Type: 140 lb auto-trip

Logged By: JC

Date: December 18, 2019

Elevation: 53 feet AMSL (Approx.)

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			2.25 inches of AC over 1.25 inches of AB			
		CL-ML	Silty Clay: damp, brown, with fine and coarse gravel and trace of clay scattered gravel At 3 feet: increase in clay content	At 0.25 to 3': EI=31 R <5		
5			Bottom of boring at 5 feet BSG			

Elevation scale: 50, 45, 40, 35, 30, 25; Depth scale: 0, 5, 10, 15, 20, 25

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-1

Project: Scottish Rite Building

Project Number: G84101.01

Drilled By: Baja Exploration

Drill Type: CME-95

Auger Type: 8-inch hollow-stem

Hammer Type: 140 lb auto-trip

Logged By: JC

Date: December 18, 2019

Elevation: 52.5 feet AMSL (Approx.)

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			3.1 inches of AC over 1.25 inches of AB			
		SM	Silty Sand: damp to moist, gray brown, fine to medium grained, with trace fine and gravel and trace of clay At 1.5 feet BSG: increase in clay content	-#200=38.7% SAND=57.6% +#4=3.7%		16.3
			Bottom of boring at 3.33 feet BSG			

Depth scale: 0, 5, 10, 15, 20, 25. Elevation scale: 50, 45, 40, 35, 30, 25.

Notes:

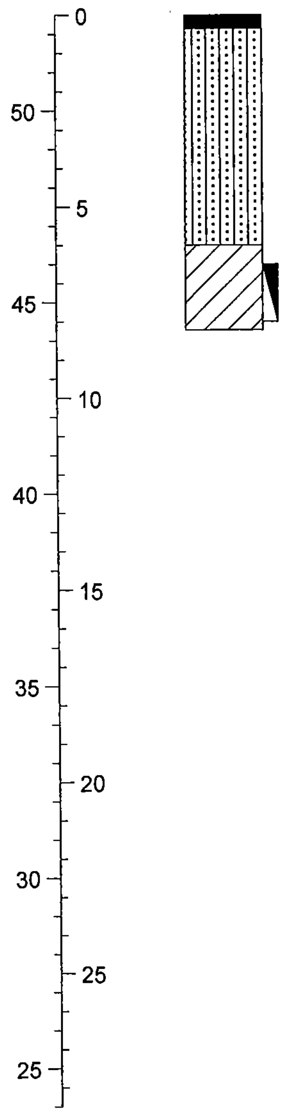
Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: P-2

Project: Scottish Rite Building
Project Number: G84101.01
Drilled By: Baja Exploration
Drill Type: CME-95
Auger Type: 8-inch hollow-stem
Hammer Type: 140 lb auto-trip

Logged By: JC
Date: December 18, 2019
Elevation: 52.5 feet AMSL (Approx.)
Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
			3.1 inches of AC over 1.25 inches of AB			
		SM	Silty Sand: damp to moist, gray brown, fine to medium grained, with trace fine and coarse gravel and trace of clay At 1.5 feet: increase in clay content			
		CL	Sandy Lean Clay: moist, low plasticity, brown	-#200=54.9% SAND=45.1% +#4=0.0		22
			Bottom of boring at 8.2 feet BSG			

Notes:

Figure Number

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Asphaltic Concrete
	Clayey sand
	Silty sand
	Silt
	Lean clay
	Silty low plasticity clay

Symbol	Description
Misc. Symbols	
	Boring continues
	Water table during drilling
Soil Samplers	
	Standard penetration test
	California Modified split barrel ring sampler
	Bulk/Grab sample

Notes:

1. Exploratory borings were dilled on December 17th and 18th using a CME-95 drill rig equipped with 8 inch dia. hollow-stem auger.

2. Groundwater was encountered in borings B-1 and B-2 at a depth of about 29 feet BSG.

3. Boring locations were located by using a steel tape or pacing.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. The "N-value" reported for the California Modified Split Barrel Sampler is the uncorrected field blow count. This value should not be interpreted as an equivalent N-value.

6. Results of tests conducted on samples recovered are reported on the logs. Abbreviations used are:

DD =	Natural dry density (pcf)	LL =	Liquid Limit (%)
UC =	Unconfined compression (psf)	PI =	Plasticity Index (%)
-4 =	Percent passing #4 sieve (%)	pH =	Soil pH
-200 =	Percent passing #200 sieve (%)	SS =	Soluble sulfates (%)
SR	Soil resistivity (ohm-cm)	Cl =	Soluble chlorides (%)
c =	Cohesion (psf)	ø =	Angle of internal friction (degrees)
N/A =	Not applicable		
pcf =	pounds per cubic foot	N/E =	Not encountered
psf =	pounds per square foot	AMSL =	Above Mean Sea Level
O.D. =	Outside Diameter		

APPENDIX C

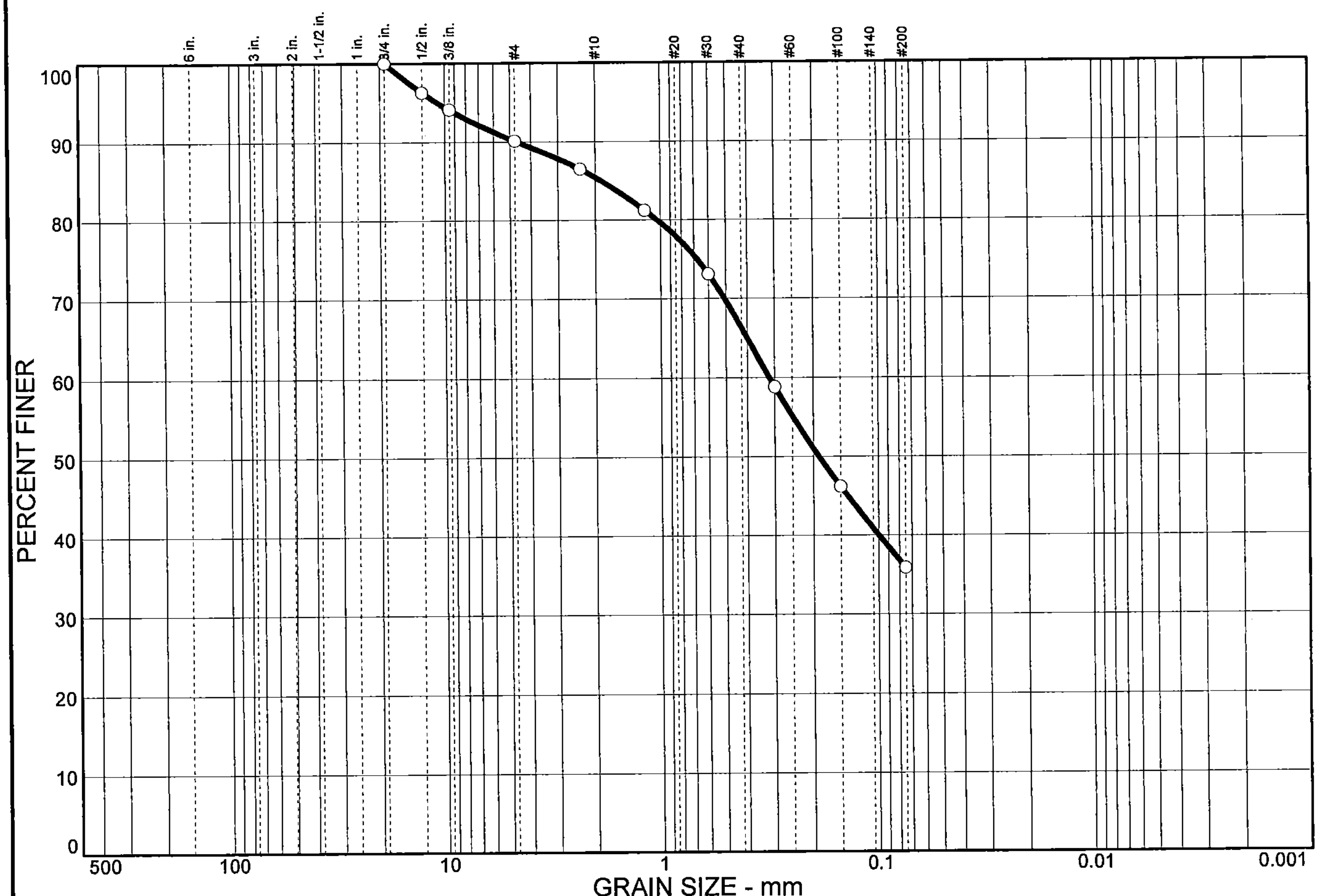
RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	To Determine:
Moisture Content (ASTM D2216)	Moisture contents representative of field conditions at the time the sample was taken.
Dry Density (ASTM D2937)	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Grain-Size Distribution (ASTM D422)	Size and distribution of soil particles, i.e., clay, silt, sand, and gravel.
Atterberg Limits (ASTM D4318)	Determines the moisture content at which the soil behaves as a viscous material (liquid limit) and the moisture content at which the soil reaches a plastic state.
Expansion Index (ASTM D4829)	Swell potential of soil with increases in moisture content.
Consolidation (ASTM D2435)	The amount and rate at which a soil sample compresses when loaded, and the influence of saturation on its behavior.
Direct Shear (ASTM D3080)	Soil shearing strength under varying loads and/or moisture conditions.

These Included:	To Determine:
Moisture-Density Relationship (ASTM D1557)	The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.
R-Value (ASTM D2844)	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Sulfate Content (ASTM D4327)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (ASTM D4327)	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM G187)	The potential of the soil to corrode metal.
pH (ASTM D4972)	The acidity or alkalinity of subgrade material.

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	9.9	54.2	35.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	96.3		
3/8 in.	94.1		
#4	90.1		
#8	86.5		
#16	81.2		
#30	73.1		
#50	58.8		
#100	46.2		
#200	35.9		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.88 D_{60}= 0.318 D_{50}= 0.189
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

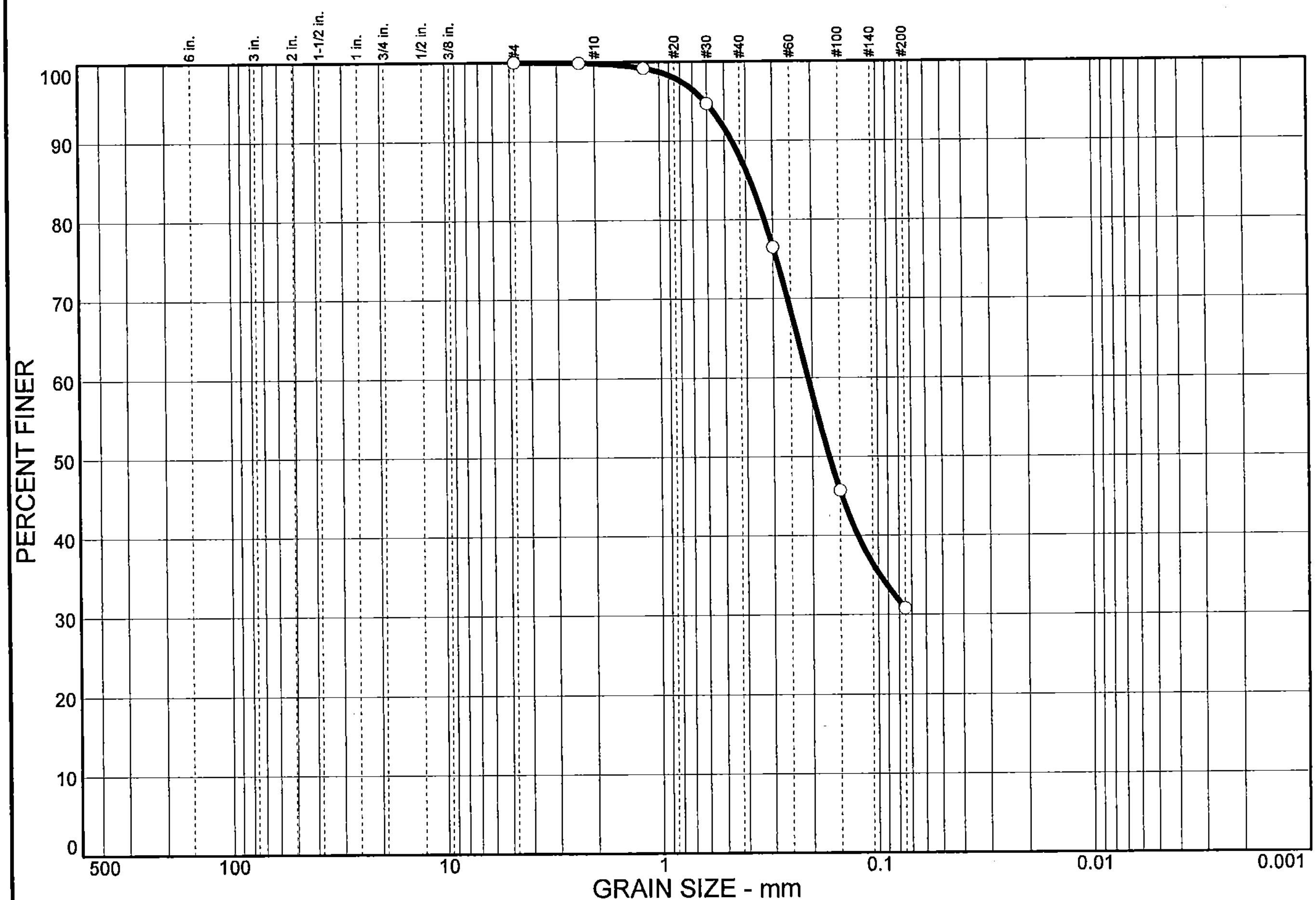
Sample No.: B-1 **Source of Sample:** **Date:** 12/17/19
Location: **Elev./Depth:** 2-3.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	69.2	30.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#16	99.2		
#30	94.7		
#50	76.5		
#100	45.7		
#200	30.8		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.382 D_{60}= 0.209 D_{50}= 0.168
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

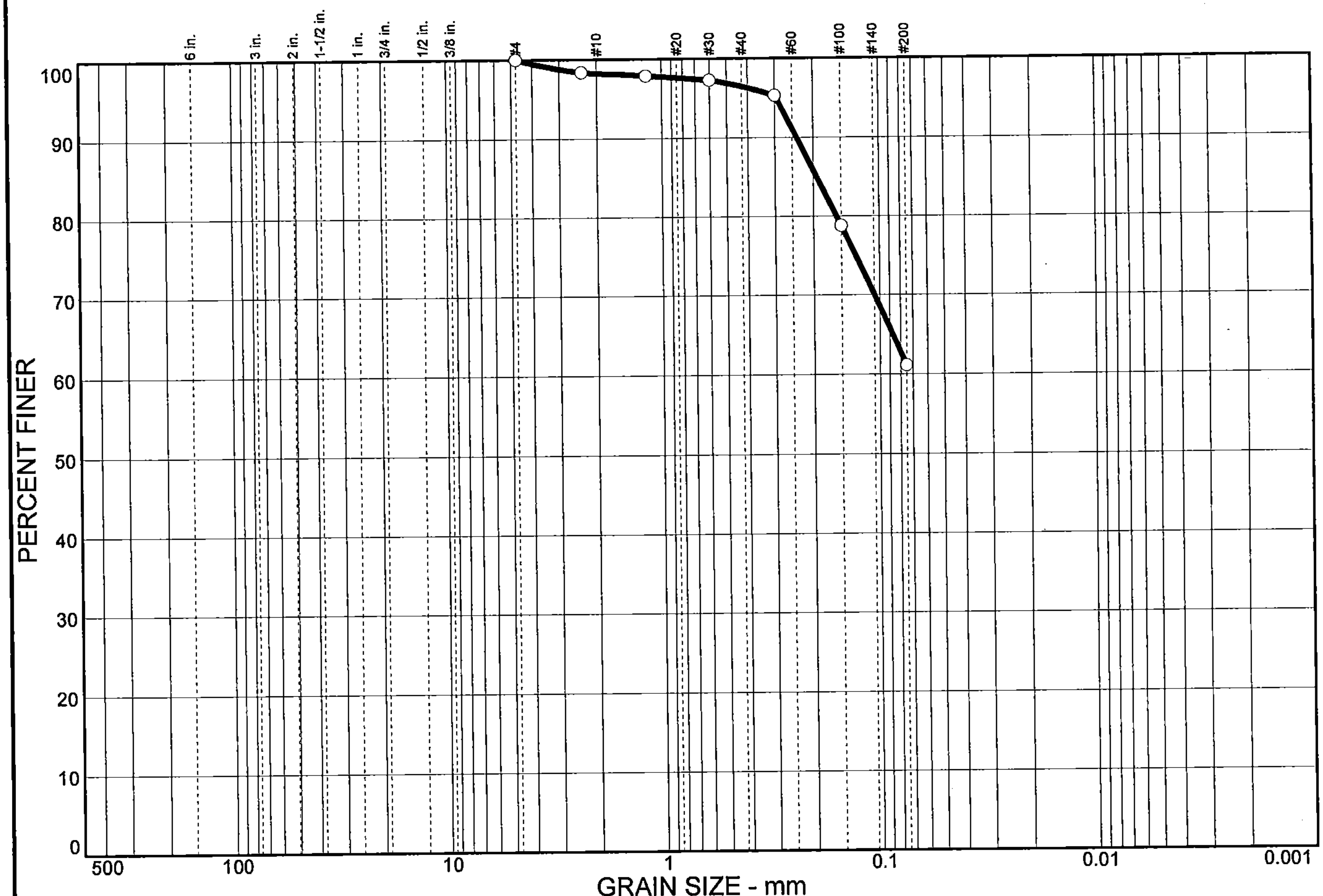
Sample No.: B-1
Location:
Source of Sample:
Date: 12/17/19
Elev./Depth: 13.5-15'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01
Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	38.9	61.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	98.4		
#16	97.9		
#30	97.3		
#50	95.3		
#100	78.8		
#200	61.1		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= 26 LL= 36 PI= 10

Coefficients

D_{85}= 0.193 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

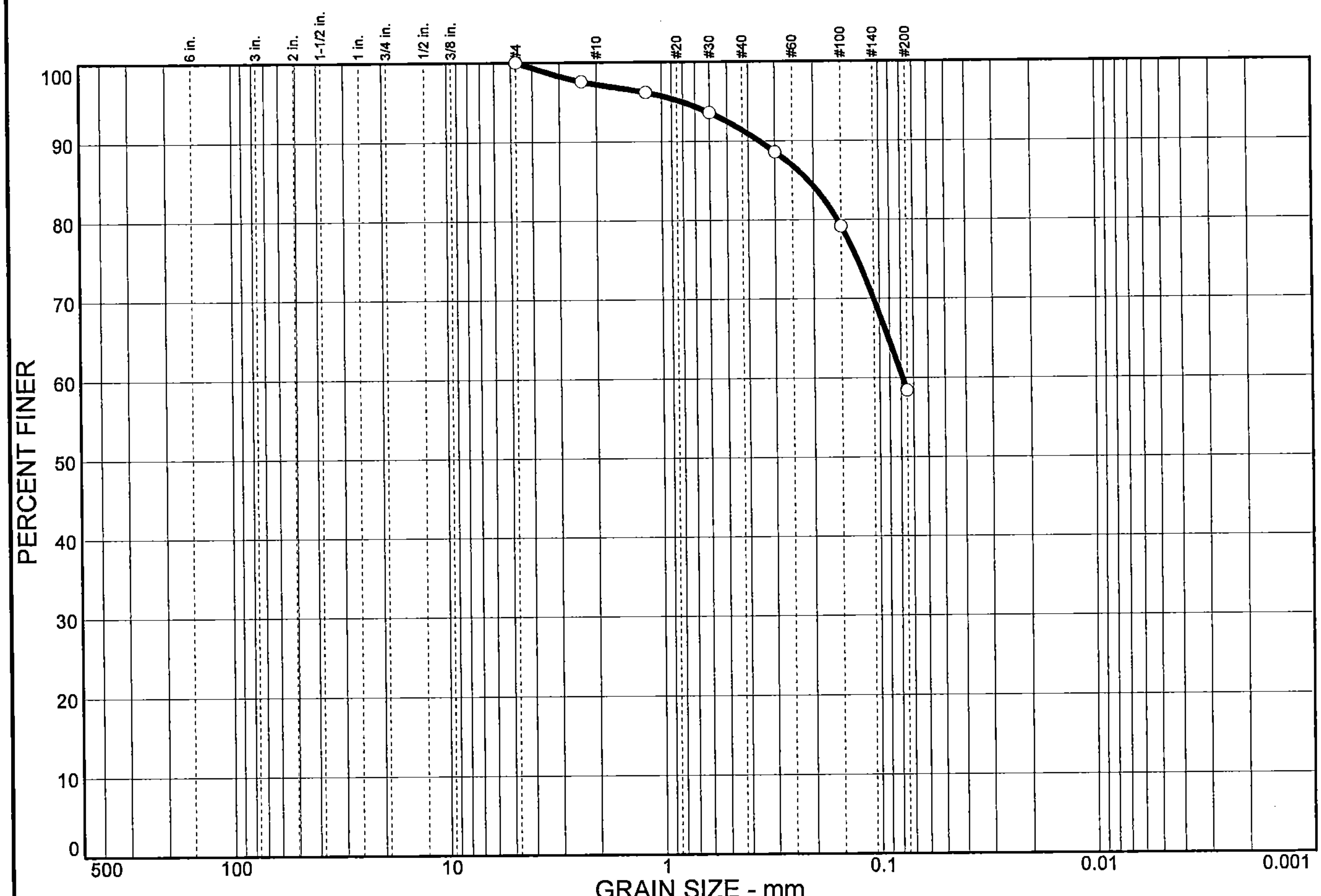
Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 12/17/19
Location: **Elev./Depth:** 18.5-20'

Moore Twining Associates, Inc.
Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	41.6	58.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	97.6		
#16	96.2		
#30	93.6		
#50	88.6		
#100	79.2		
#200	58.4		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= 23 LL= 37 PI= 14

Coefficients

D_{85}= 0.211 D_{60}= 0.0786 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

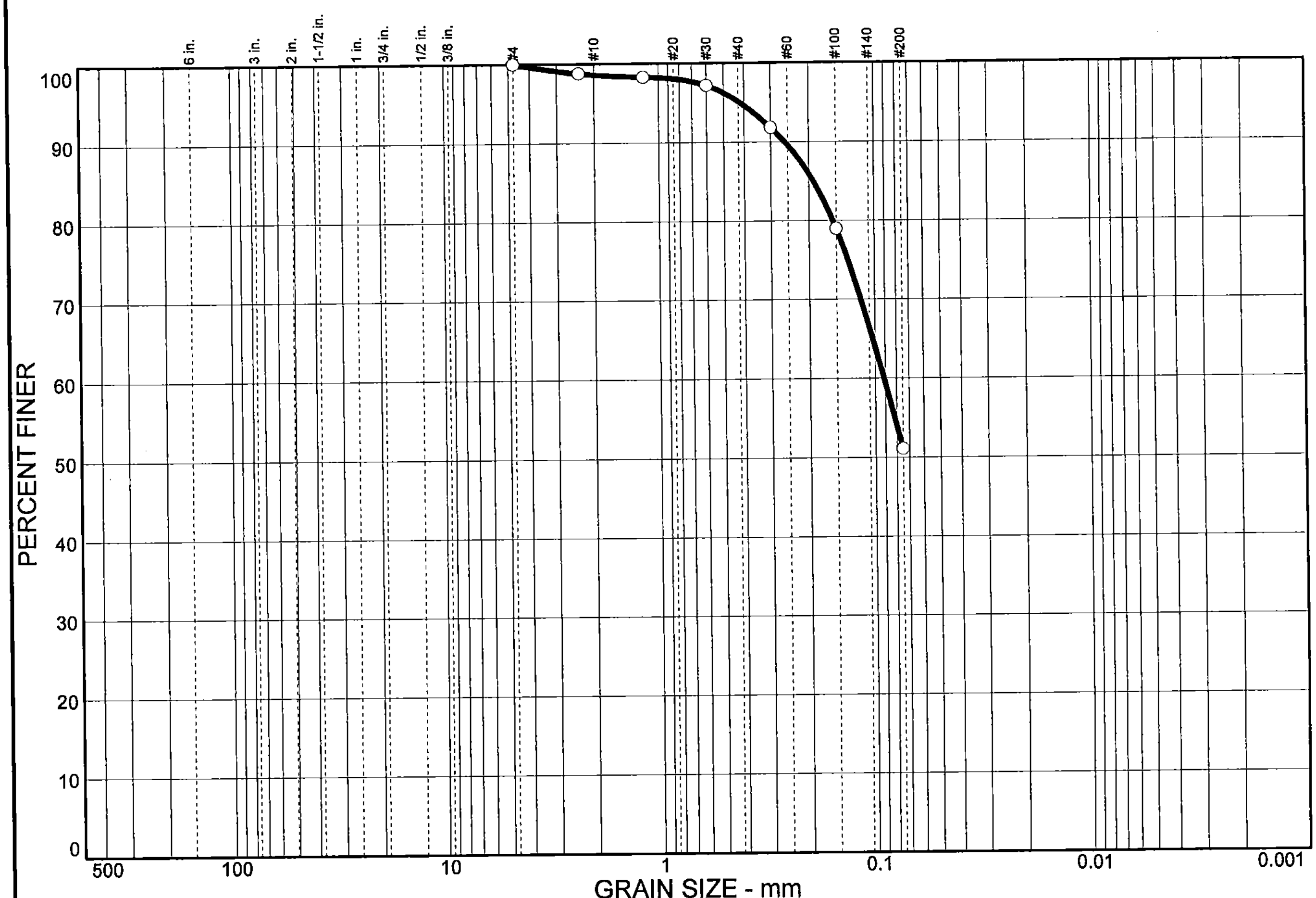
Sample No.: B-1 **Source of Sample:** **Date:** 12/17/19
Location: **Elev./Depth:** 23.5-25'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	48.9	51.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	98.8		
#16	98.3		
#30	97.2		
#50	91.8		
#100	79.0		
#200	51.1		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.191 D_{60}= 0.0914 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

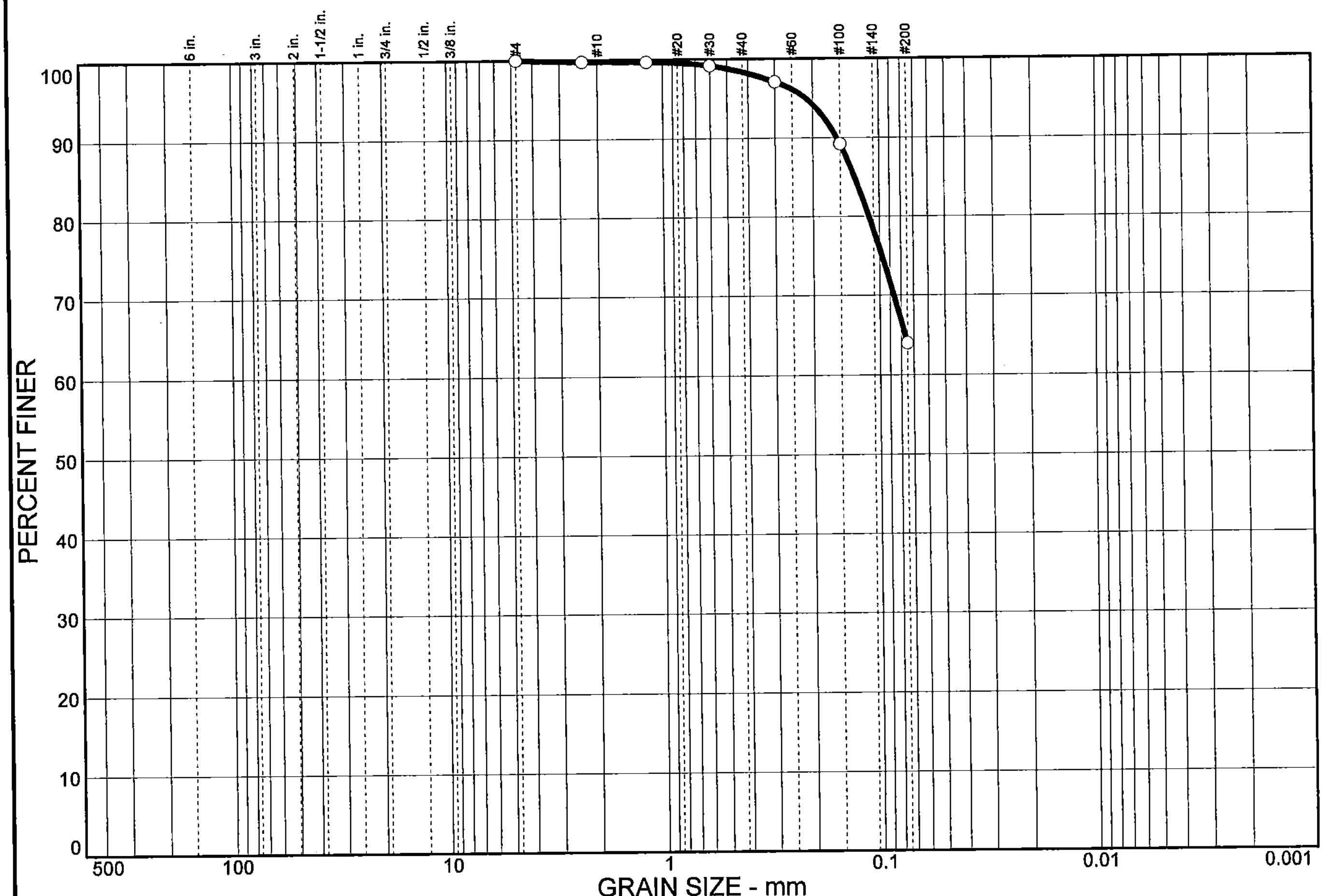
Sample No.: B-1 **Source of Sample:** **Date:** 12/17/19
Location: **Elev./Depth:** 31-32.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	36.0	64.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.8		
#16	99.7		
#30	99.2		
#50	97.1		
#100	89.2		
#200	64.0		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= 25 LL= 36 PI= 11

Coefficients

D_{85}= 0.129 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

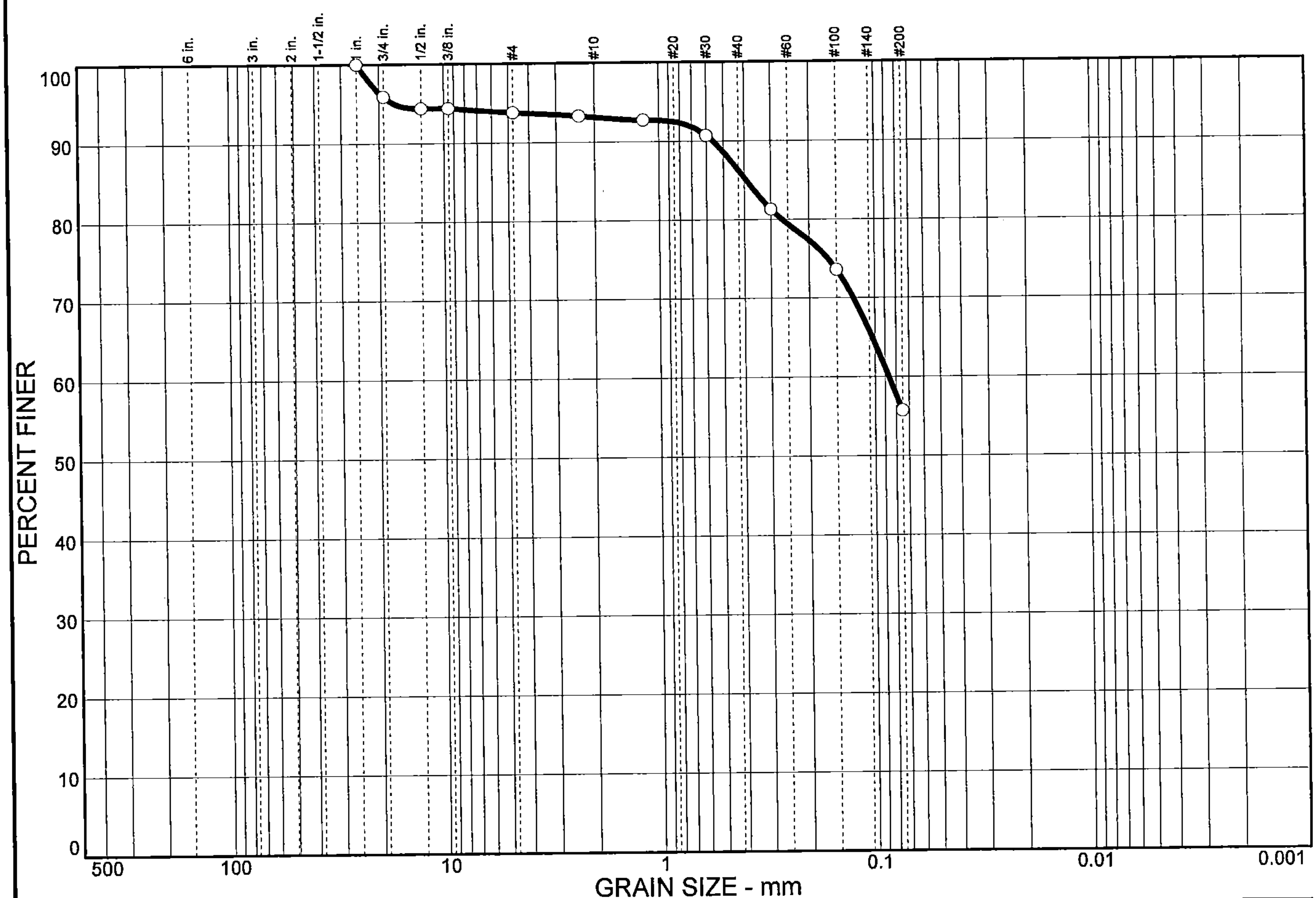
Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 12/17/19
Location: **Elev./Depth:** 33.5-35'

Moore Twining Associates, Inc.
Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	6.2	38.1	55.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	95.9		
1/2 in.	94.4		
3/8 in.	94.4		
#4	93.8		
#8	93.3		
#16	92.7		
#30	90.7		
#50	81.3		
#100	73.6		
#200	55.7		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= 21 LL= 38 PI= 17

Coefficients

D_{85}= 0.391 D_{60}= 0.0866 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

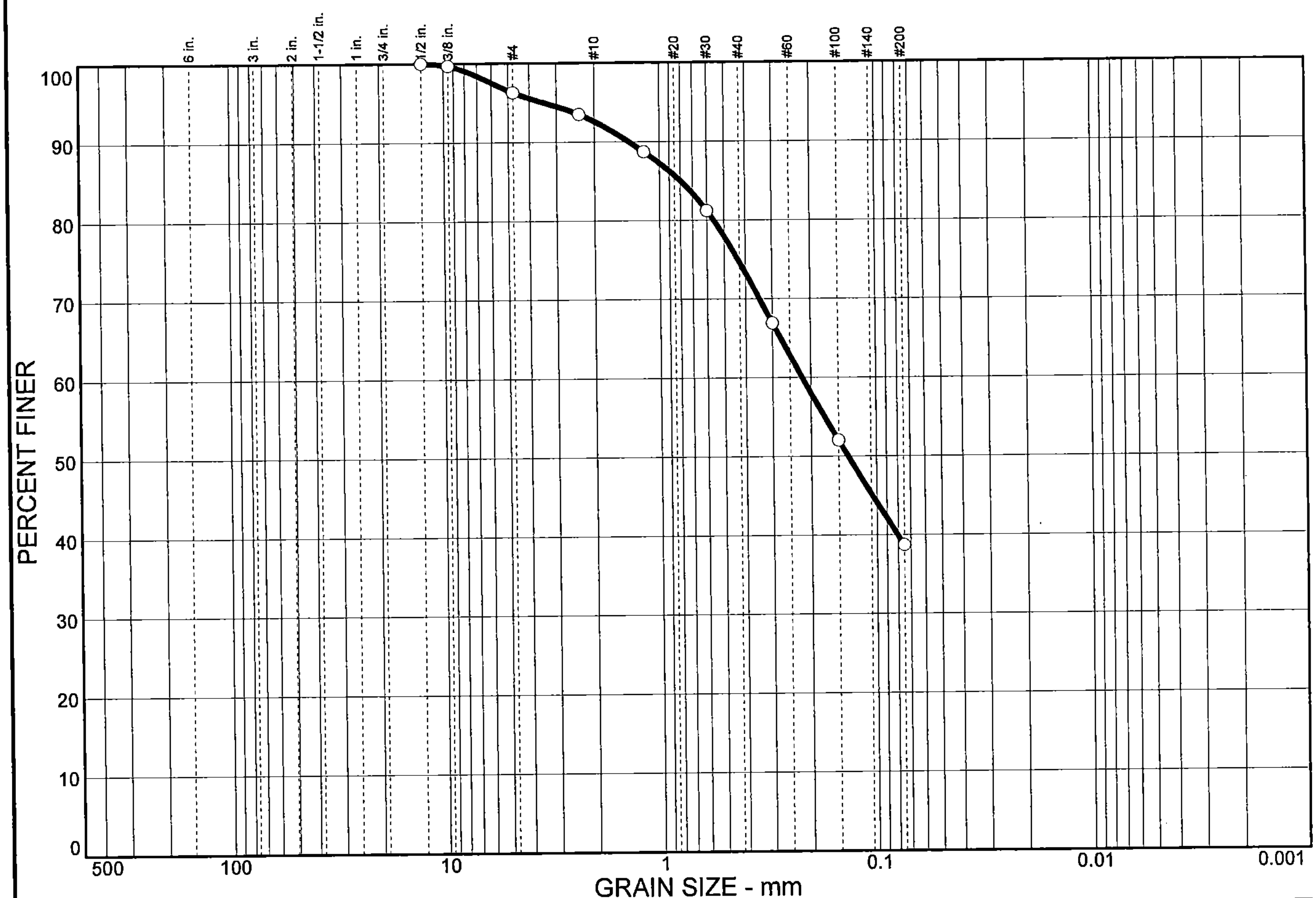
Sample No.: B-2 **Source of Sample:** **Date:** 12/17/19
Location: **Elev./Depth:** 25-26.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.7	57.6	38.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	99.7		
#4	96.3		
#8	93.5		
#16	88.7		
#30	81.2		
#50	66.9		
#100	52.0		
#200	38.7		

* (no specification provided)

Material Description

Silty sand trace of clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.796 D_{60}= 0.220 D_{50}= 0.136
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

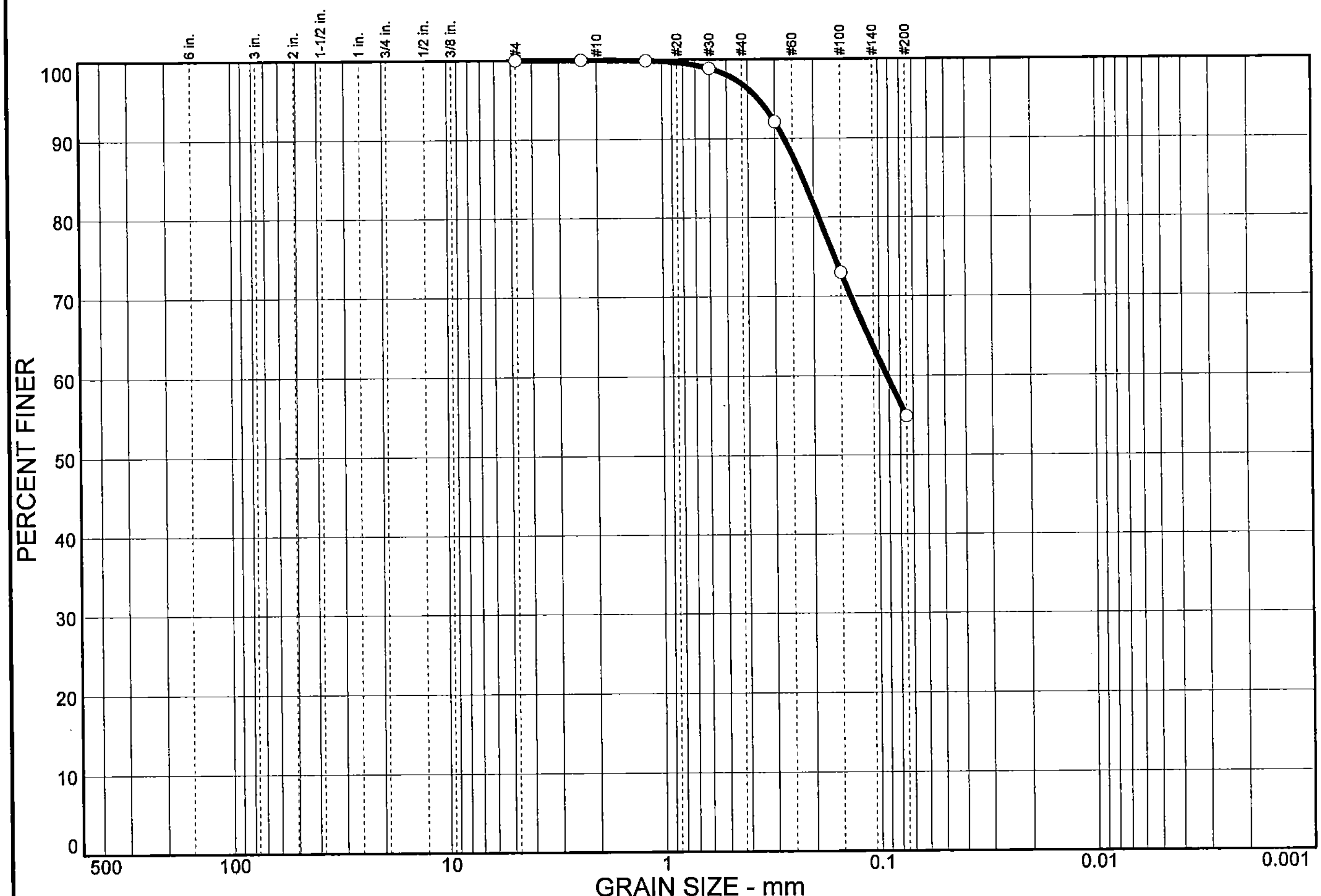
Sample No.: P-1 **Source of Sample:** **Date:** 12/18/19
Location: **Elev./Depth:** 1.5-3'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	45.1	54.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	99.9		
#30	98.9		
#50	92.1		
#100	73.0		
#200	54.9		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.225 D_{60}= 0.0922 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

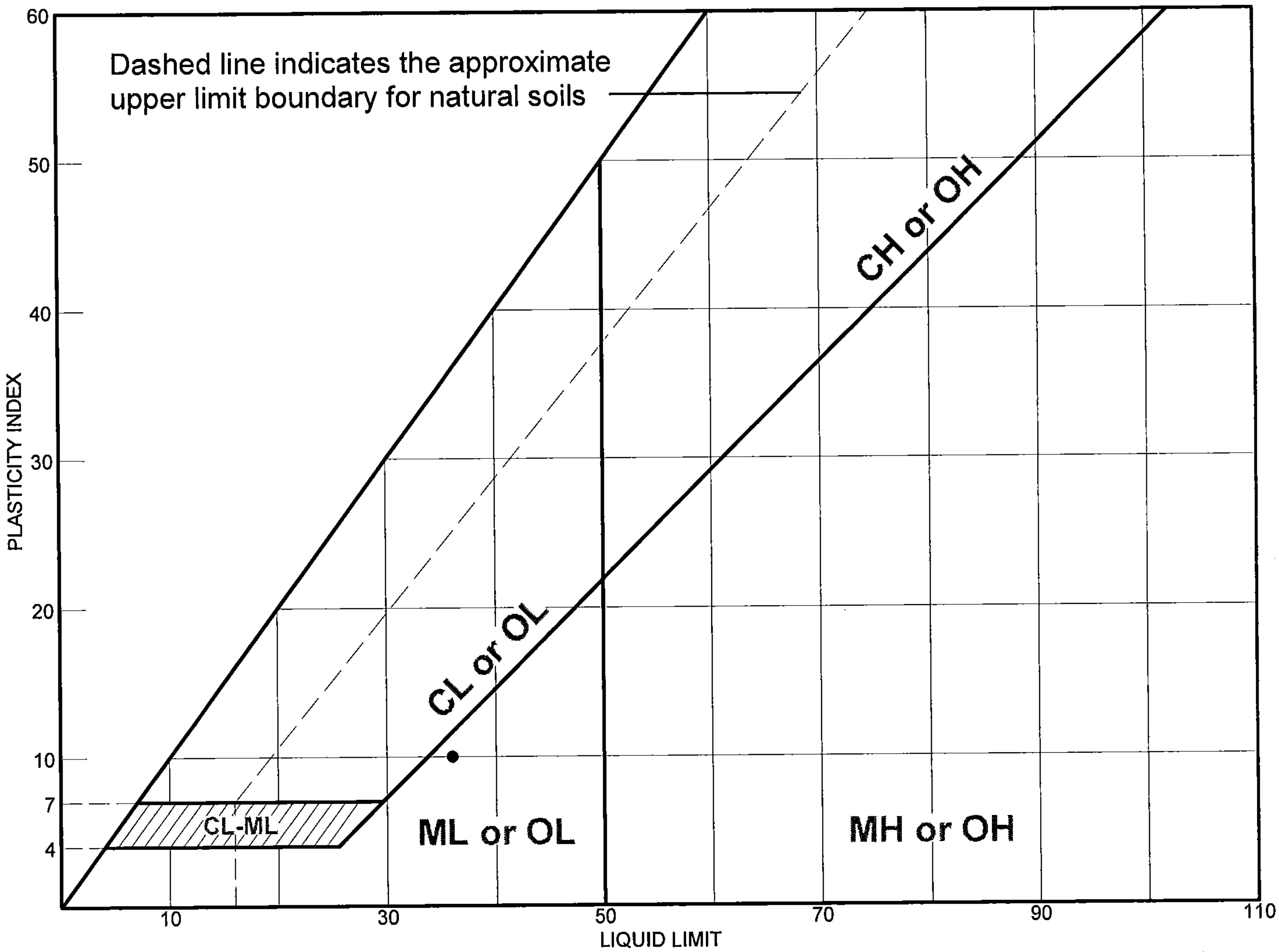
Sample No.: P-2 **Source of Sample:** **Date:** 12/18/19
Location: **Elev./Depth:** 6.5-8'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Scottish Rite Building
Project No: G84101.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy silt	36	26	10	96.5	61.1	ML

Project No. G84101.01 **Client:**

Project: Scottish Rite Building

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 18.5-20'

Remarks:

●

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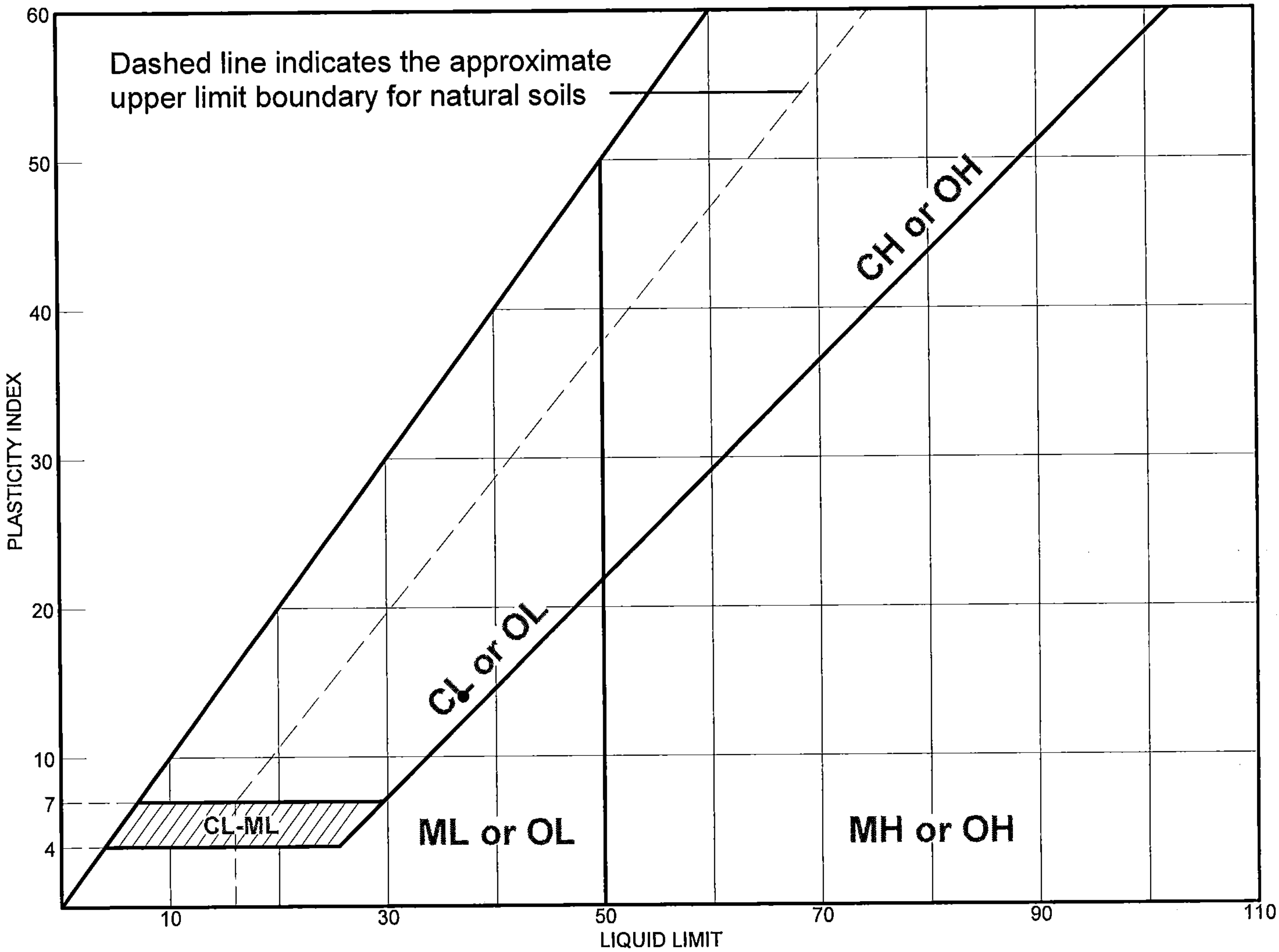
Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	37	23	14	91.4	58.4	CL

Project No. G84101.01 **Client:**

Project: Scottish Rite Building

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 23.5-25'

Remarks:

●

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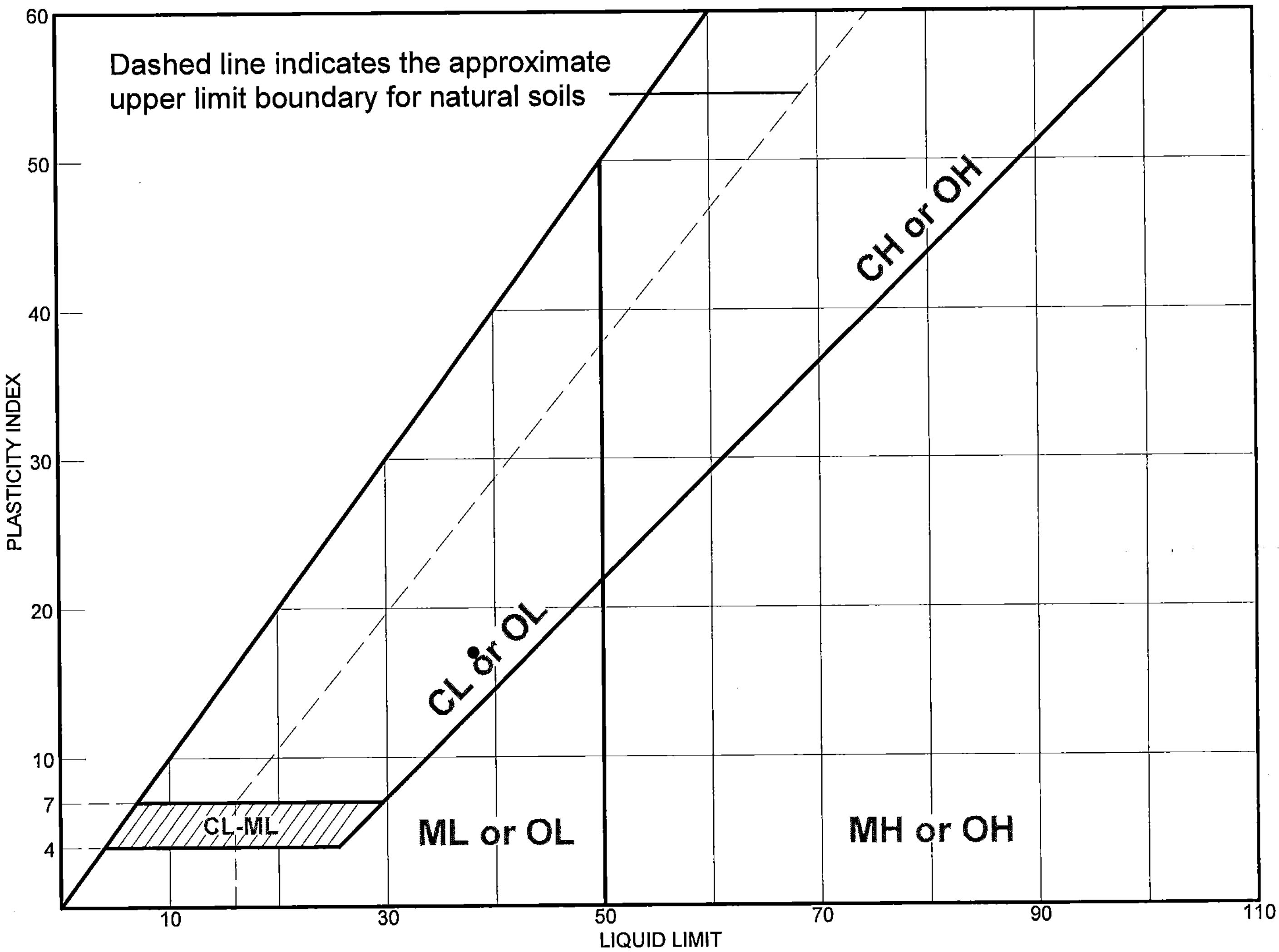
Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	38	21	17	86.3	55.7	CL

Project No. G84101.01 **Client:**

Project: Scottish Rite Building

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 25-26.5'

Remarks:

●

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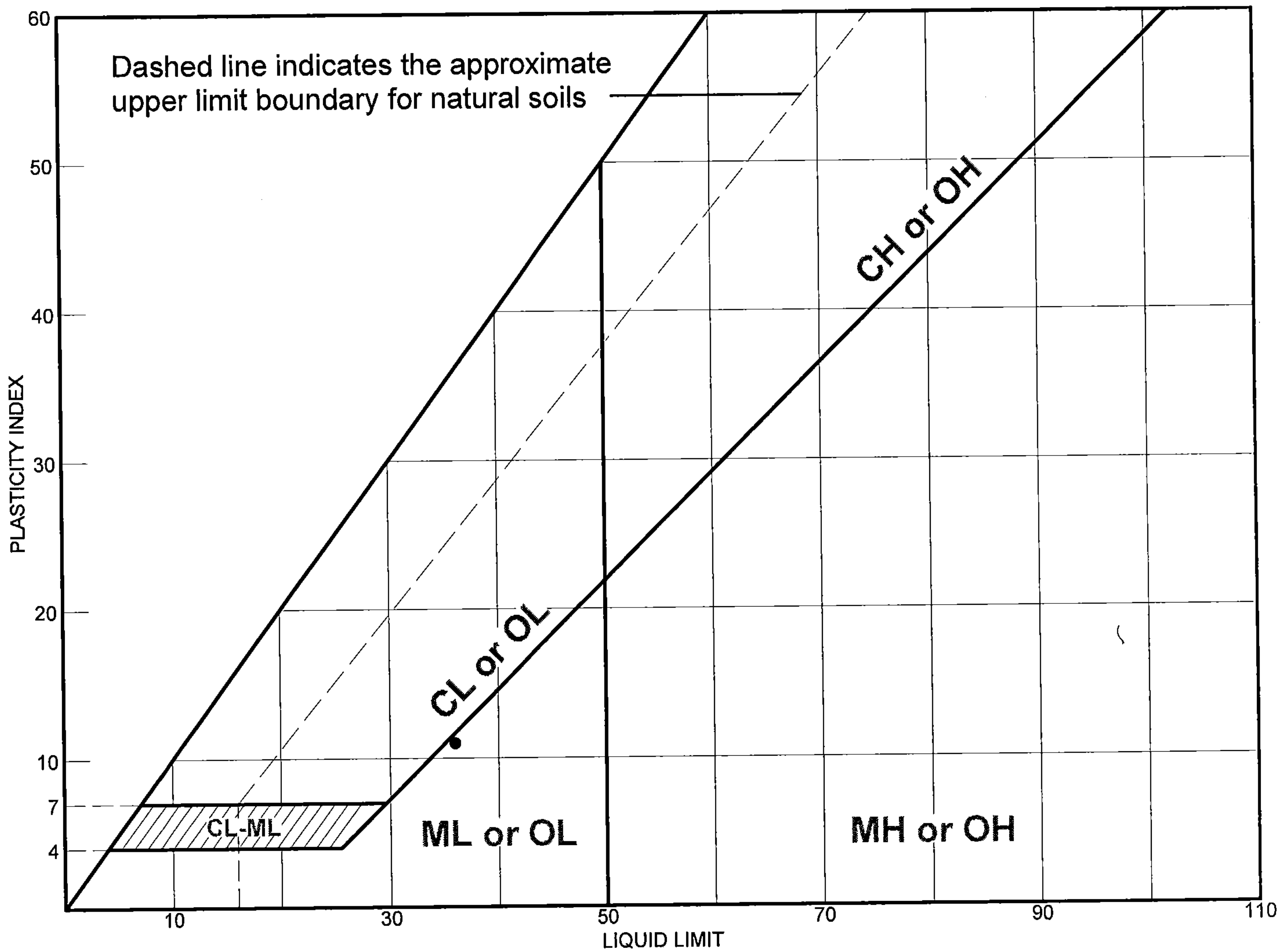
Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy silt	36	25	11	98.4	64.0	ML

Project No. G84101.01 **Client:**

Project: Scottish Rite Building

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 33.5-35'

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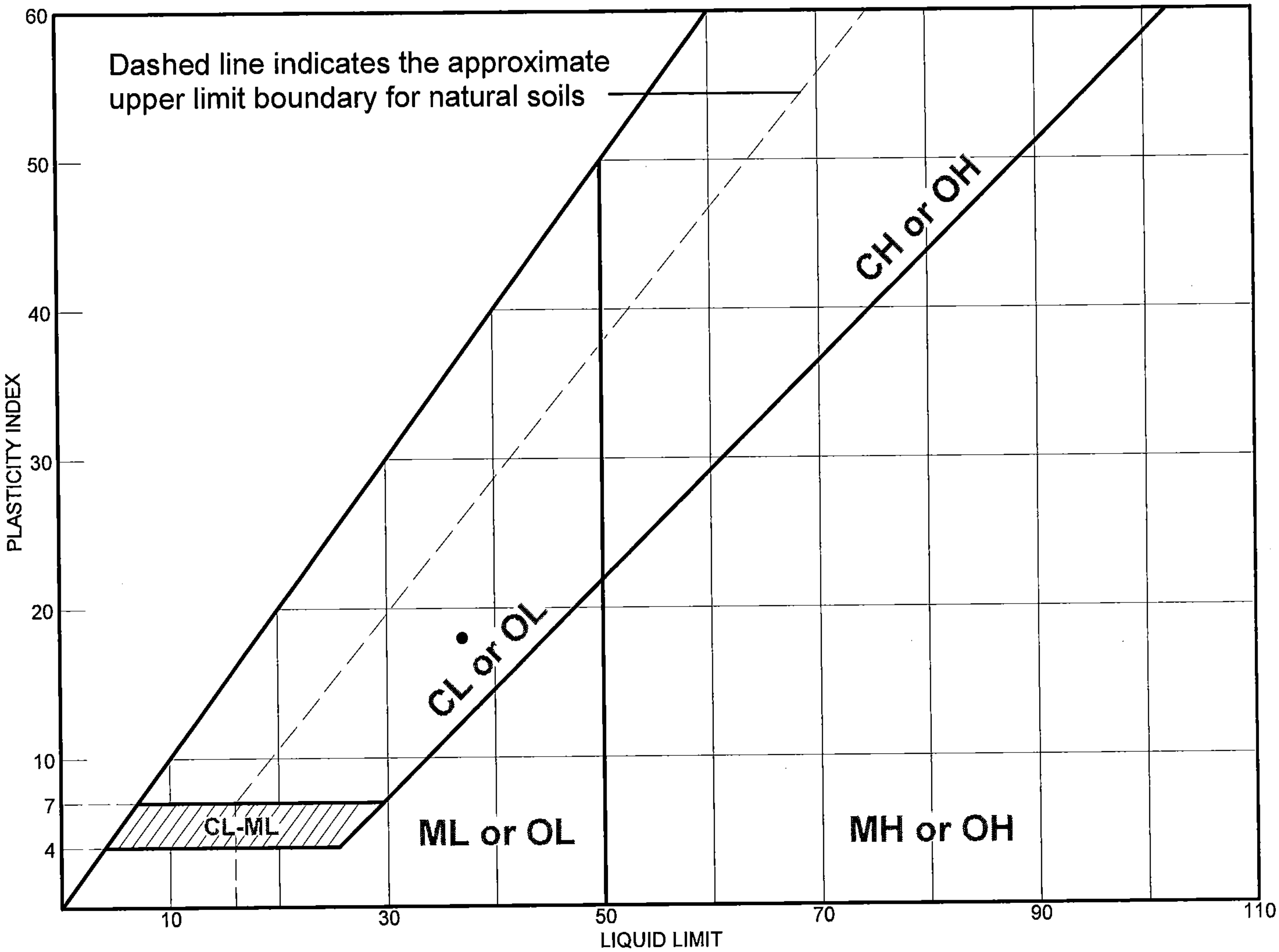
Remarks:

●

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	37	19	18			CL

Project No. G84101.01 **Client:**

Project: Scottish Rite Building

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 43.5-45'

Remarks:

●

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Fresno, CA

Figure

MOORE TWINING
ASSOCIATES, INC.

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Scottish Rite Building	REPORT DATE:	1/10/2017
		TEST DATE:	1/9/2017
MTA PROJECT NO.:	G84101.01		
SAMPLE I.D.:	B-7 @ 0.25-3'		
SAMPLED BY:	JC		
SAMPLE DATE:	12/17/2019	TESTED BY:	MA
MATERIALS DESCRIPTION:	Lean clay		
% PASSING # 4 SIEVE	100		

Initial Moisture Determination:

Pan + Wet Soil Wt., gm	250.0
Pan + Dry Soil Wt., gm	226.2
Pan Wt., gm	0.0
Initial % Moisture Content	10.5

Final Moisture Determination:

Wet Soil Wt., lbs	0.9511
Dry Soil Wt., lbs	0.7819
Final % Moisture Content	21.6

Initial Expansion Data:

Ring + Sample Wt., lbs	0.8642
Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8642
Remolded Wet Density, pcf	118.8
Remolded Dry Density, pcf	107.5

Final Expansion Data:

Ring + Sample Wt., lbs	0.9511
Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.9511
Remolded Wet Density, pcf	126.8
Remolded Dry Density, pcf	104.3

Initial Volume	Final Volume
0.00727222	0.007498

Expansion Data:

Initial Gage Reading, in:	0.0500
Final Gage Reading, in:	0.0811
Expansion, in:	0.0311
Expansion Index	31

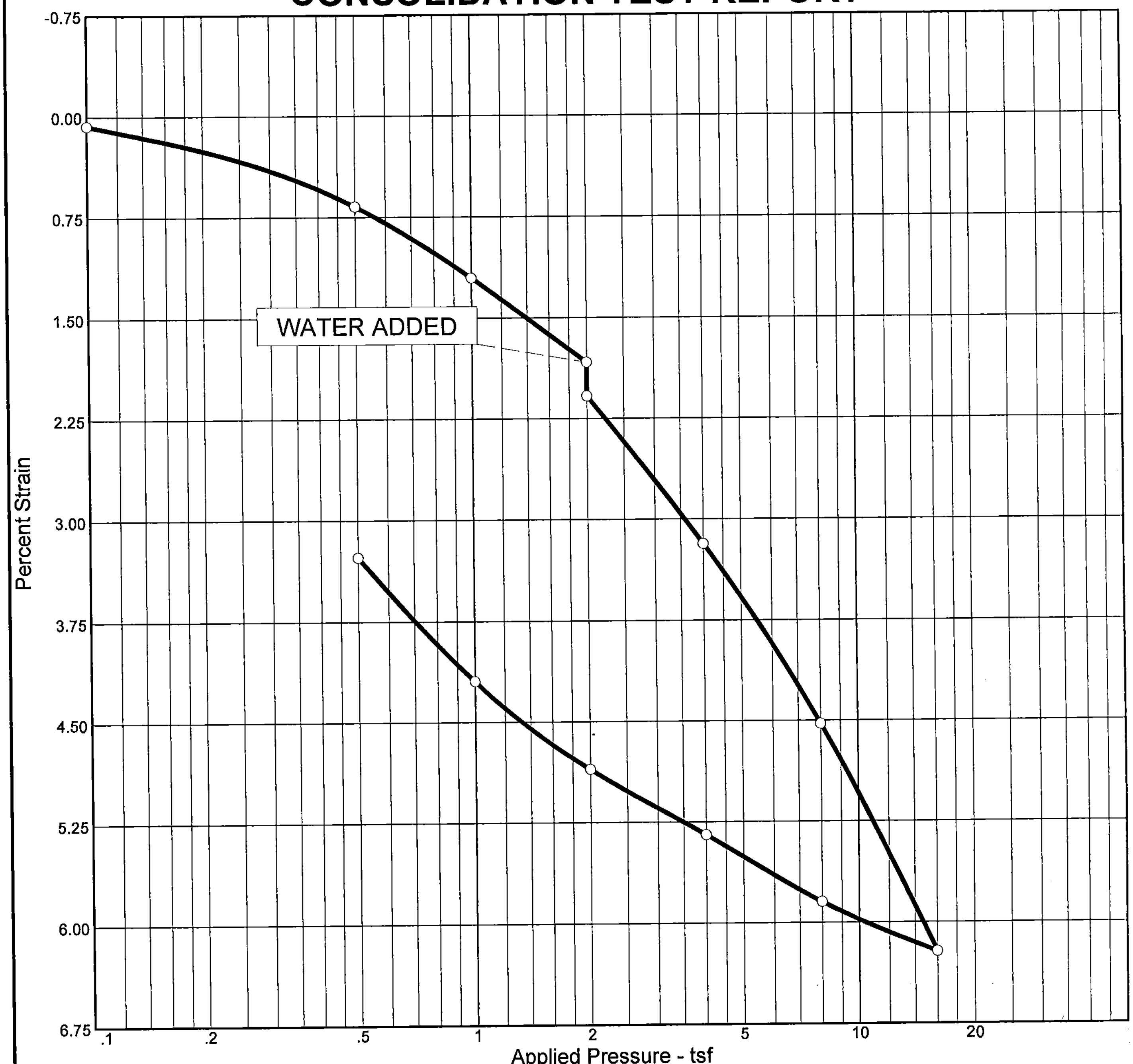
Comments: **Low Expansion Potential**

Classification of Expansive Soils. (Table No.1 From ASTM D4829)

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

www.mooretwining.com
PH: 800.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_s	Swell Press. (tsf)	Clpse. %	e_o
Sat.	Moist.											
71.0 %	18.6 %	97.7			2.65		8.23	0.10	0.03		0.3	0.693

MATERIAL DESCRIPTION	USCS	AASHTO
Silty clay		

Project No. G84101.01 **Client:**

Project: Scottish Rite Building

Source: **Sample No.:** B-2 **Elev./Depth:** 5-6.5'

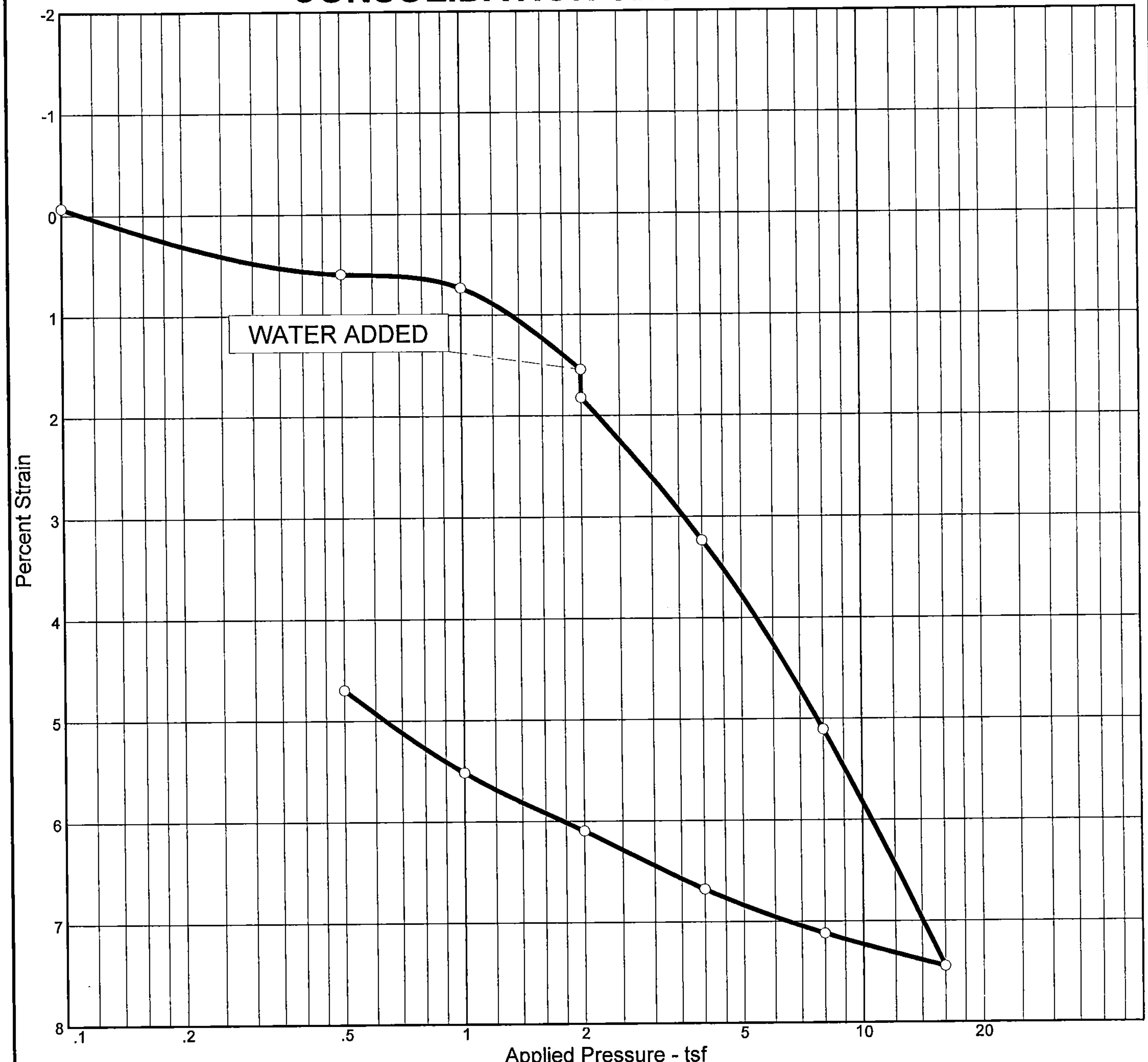
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_s	Swell Press. (tsf)	Clpse. %	e_o
Sat.	Moist.											
78.2 %	19.7 %	99.2			2.65		2.45	0.13	0.03		0.3	0.668

MATERIAL DESCRIPTION	USCS	AASHTO
Silty Sand		

Project No. G84101.01 **Client:**

Project: Scottish Rite Building

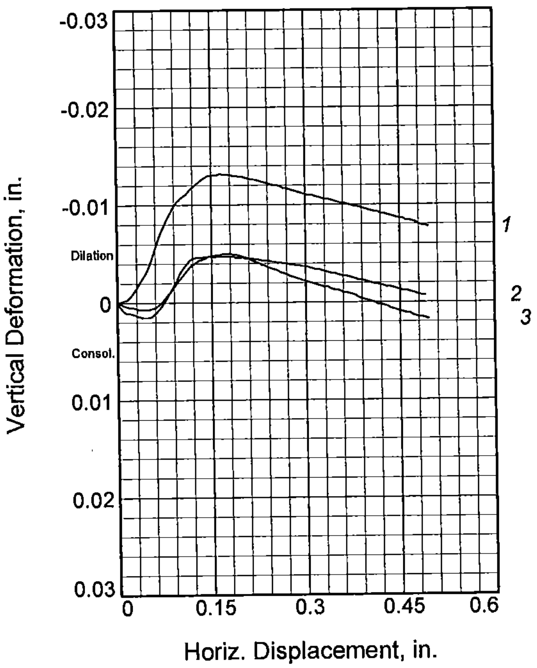
Source: **Sample No.:** B-3 **Elev./Depth:** 5-6.5'

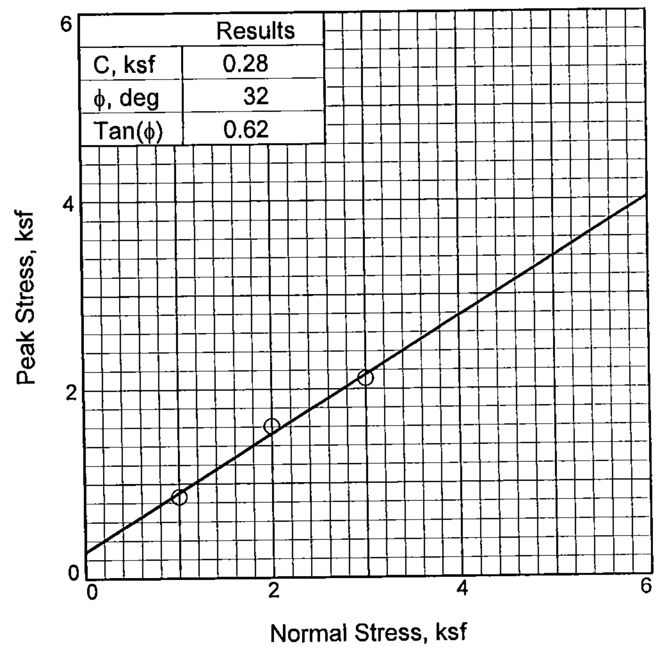
Moore Twining Associates, Inc.

Fresno, CA

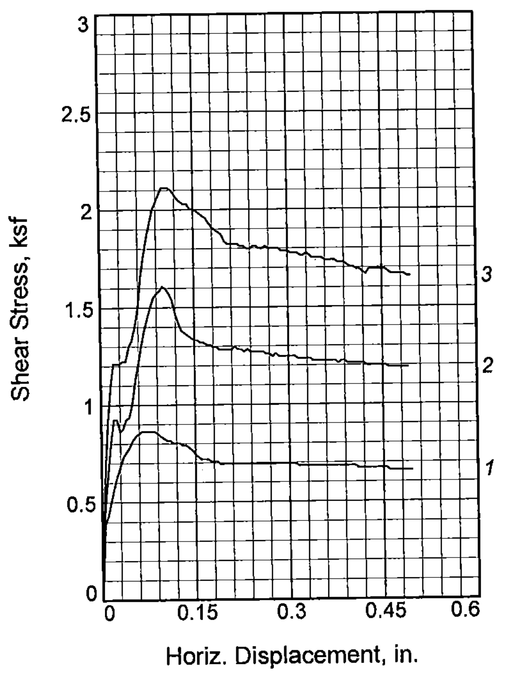
Remarks:

Figure





Results	
C, ksf	0.28
ϕ, deg	32
Tan(ϕ)	0.62



	Sample No.	1	2	3
Initial	Water Content, %	17.8	19.7	24.5
	Dry Density, pcf	96.2	97.2	95.9
	Saturation, %	65.5	74.3	89.6
	Void Ratio	0.7192	0.7016	0.7251
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	25.5	24.4	24.6
	Dry Density, pcf	98.3	99.8	99.4
	Saturation, %	98.8	98.6	98.3
	Void Ratio	0.6833	0.6571	0.6636
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.97	0.96
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.86	1.61	2.11
	Displacement, in.	0.07	0.10	0.10
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	2.7	4.1	4.1

Sample Type:
Description: Silty clay

Specific Gravity= 2.65
Remarks:

Figure ________

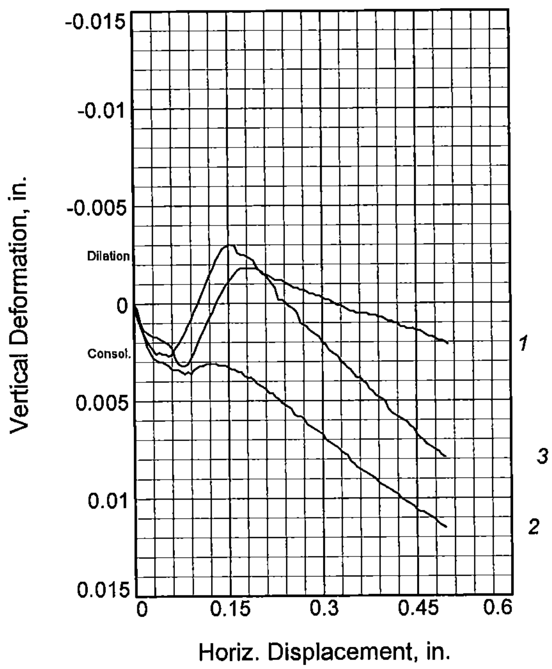
Client:

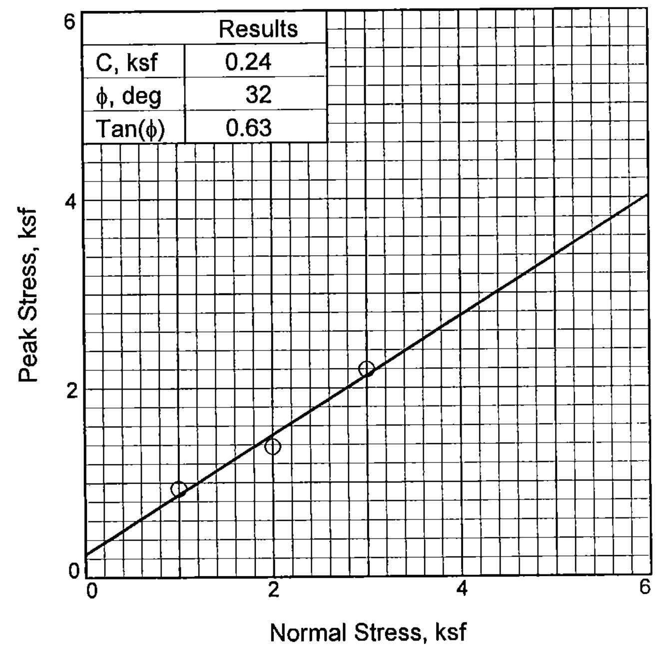
Project: Scottish Rite Building

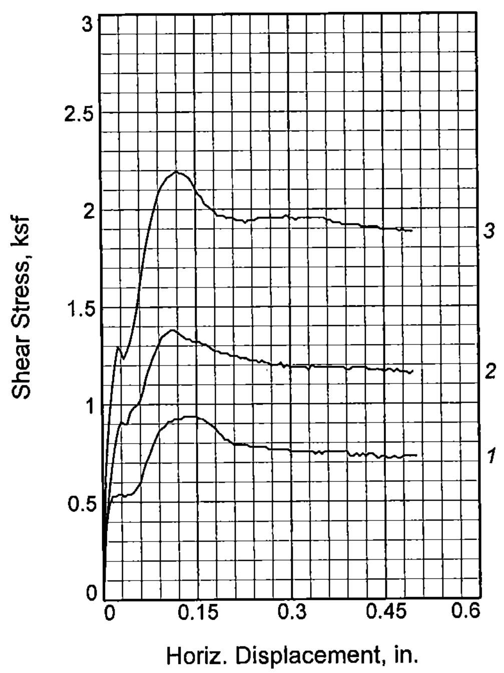
Sample Number: B-2 **Depth:** 5-6.5'

Proj. No.: G84101.01 **Date Sampled:** 12/17/19

DIRECT SHEAR TEST REPORT
Moore Twining Associates, Inc.
Fresno, CA







Sample No.		1	2	3
Initial	Water Content, %	16.0	24.5	16.6
	Dry Density, pcf	94.4	92.7	106.6
	Saturation, %	56.4	82.7	79.7
	Void Ratio	0.7525	0.7839	0.5517
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	26.7	27.0	17.5
	Dry Density, pcf	96.5	95.4	110.8
	Saturation, %	99.0	97.4	93.8
	Void Ratio	0.7137	0.7336	0.4932
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.97	0.96
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		0.94	1.38	2.20
Displacement, in.		0.13	0.11	0.12
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		5.4	4.6	5.0

Sample Type:

Description: Silty sand

Specific Gravity= 2.65

Remarks:

Figure ________

Client:

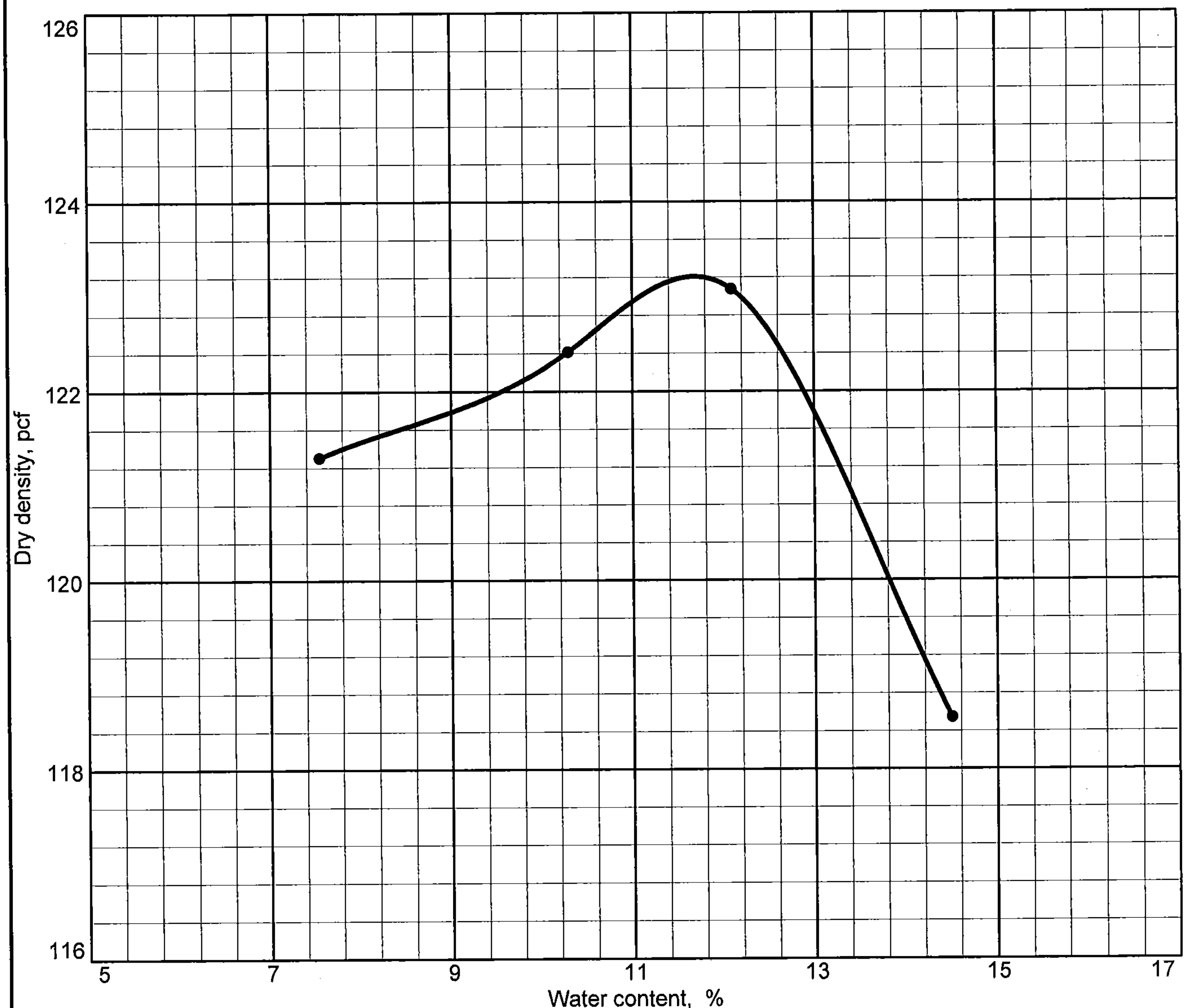
Project: Scottish Rite Building

Sample Number: B-3 **Depth:** 3.5-5'

Proj. No.: G84101.01 **Date Sampled:** 12/17/19

DIRECT SHEAR TEST REPORT
Moore Twining Associates, Inc.
Fresno, CA

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0.5-3'							3.3	

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 123.2 pcf Optimum moisture = 11.7 %	Clayey sand / Silty Sand Mix

Project No. G84101.01 **Client:**

Project: Scottish Rite Building

● **Source:** **Sample No.:** B-6 **Elev./Depth:** 0.5-3'

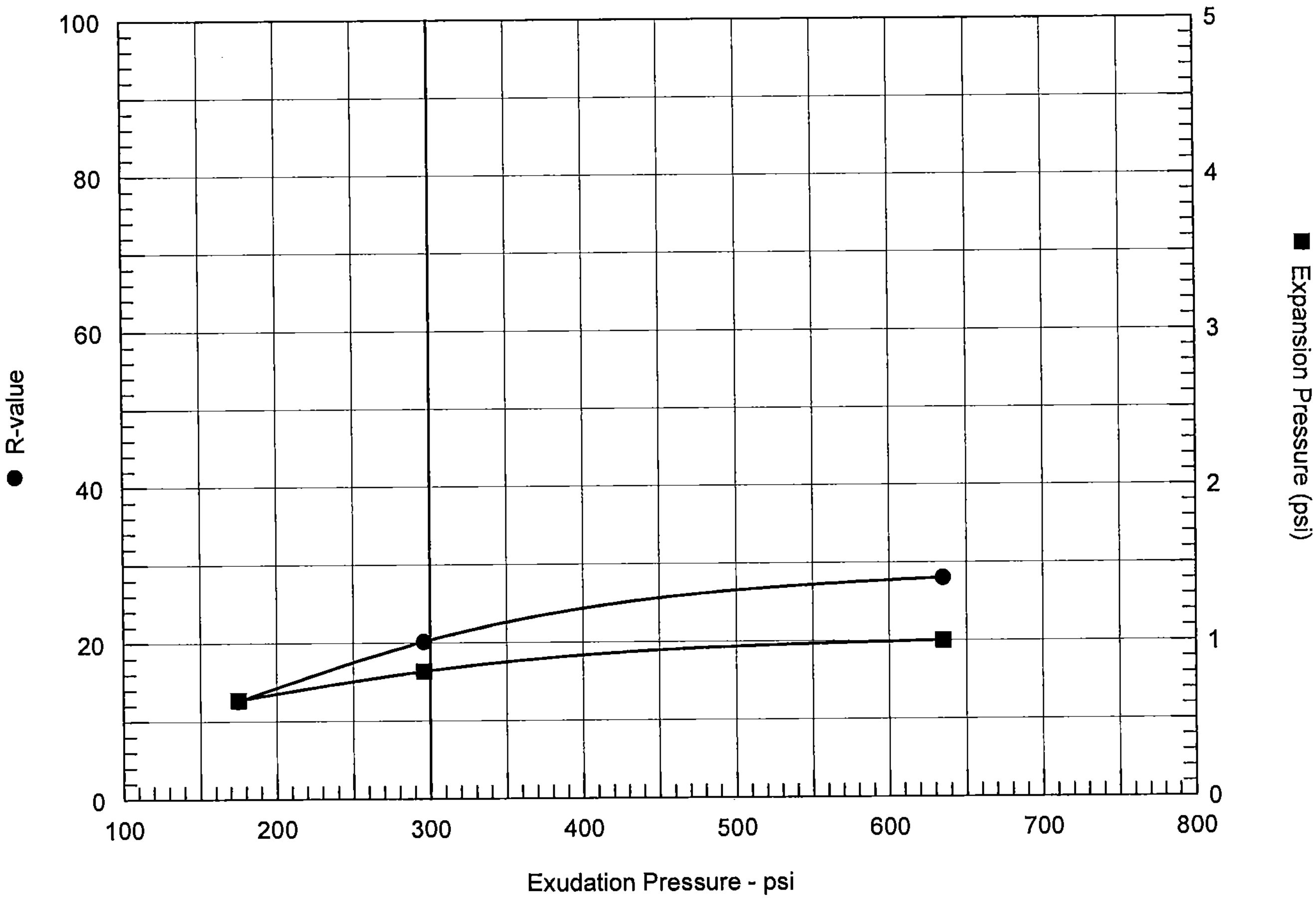
Remarks:

Moore Twining Associates, Inc.

Fresno, CA

Figure

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	100	112.1	17.8	0.64	125	2.46	175	13	13
2	300	118.3	15.6	1.00	89	2.33	635	32	28
3	200	115.0	16.7	0.82	106	2.39	296	22	20

Test Results	Material Description
R-value at 300 psi exudation pressure = 20 **Exp. pressure at 300 psi exudation pressure =** 0.82 psi	Clayey sand / Silty Sand Mix

Project No.: G84101.01

Project: Scottish Rite Building

Sample Number: B-6 **Depth:** 0.5-3'

Date: 1/13/2020

Tested by:

Checked by:

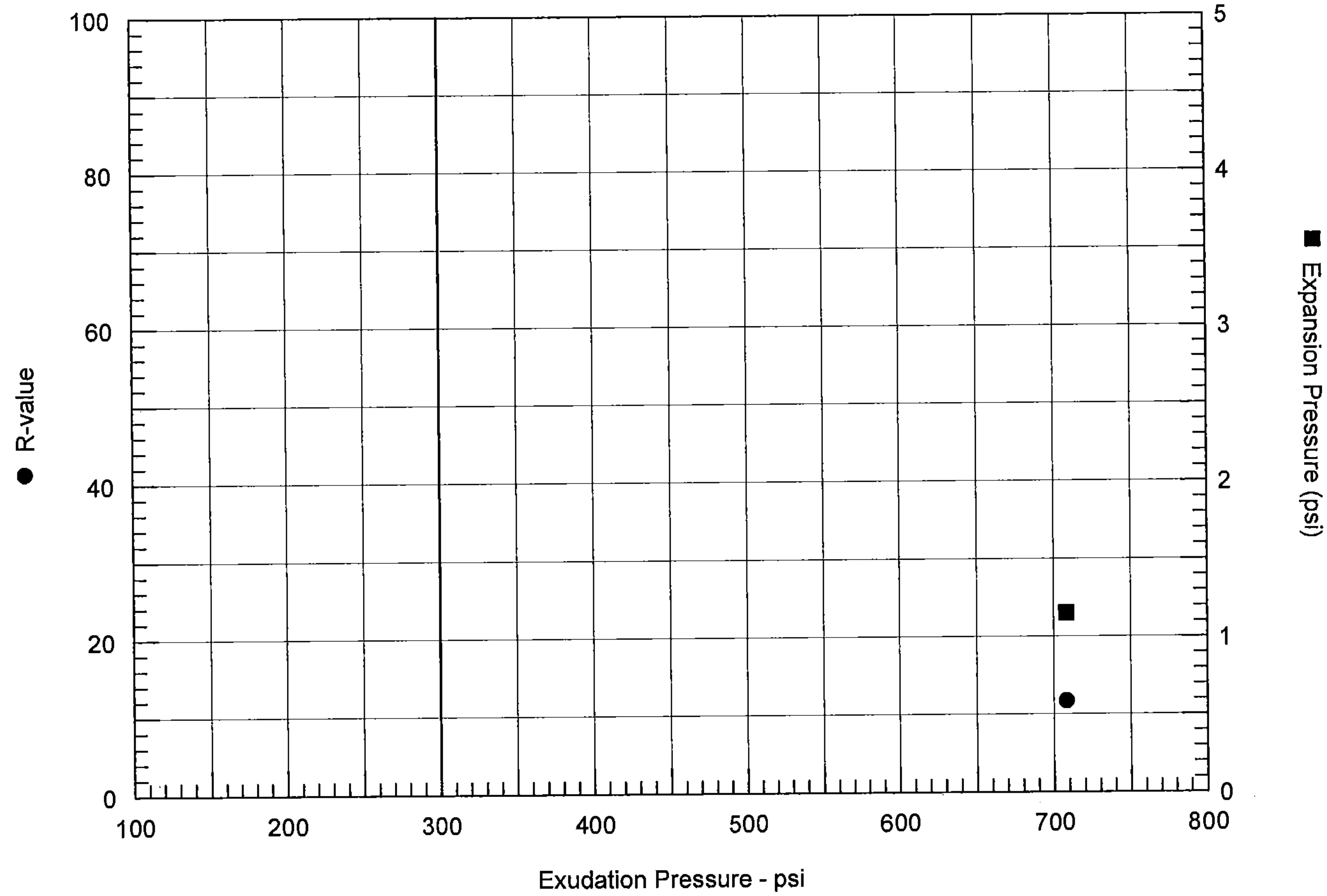
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure ______

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	50	104.5	20.2	1.15	127	2.42	708	12	12

Test Results	Material Description
R-value at 300 psi exudation pressure = n/a **Exp. pressure at 300 psi exudation pressure =** n/a psi	Silty clay

Project No.: G84101.01

Project: Scottish Rite Building

Sample Number: B-7 **Depth:** 0.5-3'

Date: 1/13/2020

Tested by:

Checked by:

Remarks:
R-V < 5; Sample Excuded

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure ______

MOORE TWINING ASSOCIATES, INC.

Since 1898

Project Name:	Scottish Rite Building
Project Number:	G84101.01
Subject:	Minimum Resistivity, ASTM G187
Material Description:	Silty clay
Location:	B-6 @ 0.5-3'

Report Date:	1/6/2020
Sample Date:	12/17/2019
Sampled By:	JC
Tested By:	MA
Test Date:	12/23/2019

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	90,045
100 mls	24,012
150 mls	22,678
200 mls	7,337
250 mls	1,868
300 mls	2,001

Remarks: Min. Resistivity is 1,868 Ohm-cm

PH: 800.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

MOORE TWINING
ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

December 27, 2019

Work Order #: **FL23019**

Ken Clark
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: Scottish Rite Building

Enclosed are the analytical results for samples received by our laboratory on **12/23/19** . For your reference, these analyses have been assigned laboratory work order number **FL23019.**

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.

Susan Federico

Susan Federico
Client Services Representative

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Scottish Rite Building **Project Number:** G84101.01 **Project Manager:** Ken Clark	**Reported:** 12/27/2019

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B-6 @ 0.5 - 3'		FL23019-01	Soil	12/17/19 00:00	12/23/19 12:30

MOORE TWINING
ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Scottish Rite Building **Project Number:** G84101.01 **Project Manager:** Ken Clark	**Reported:** 12/27/2019

B-6 @ 0.5 - 3'

FL23019-01 (Soil)   Sampled: 12/17/19 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		ND	6.0	mg/kg	3	B9L2604	12/26/19	12/26/19	ASTM D4327
Chloride		ND	0.00060	% by Weight	3	[CALC]	12/26/19	12/26/19	ASTM D4327
Sulfate as SO4		**0.0024**	0.00060	% by Weight	3	[CALC]	12/26/19	12/26/19	ASTM D4327
pH		**8.9**	0.10	pH Units	1	B9L2604	12/26/19	12/27/19	ASTM D4972 Mod
Sulfate as SO4		**24**	6.0	mg/kg	3	B9L2604	12/26/19	12/26/19	ASTM D4327

Notes and Definitions

µg/L	micrograms per liter (parts per billion concentration units)
mg/L	milligrams per liter (parts per million concentration units)
mg/kg	milligrams per kilogram (parts per million concentration units)
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference
	Analysis of pH, filtration, and residual chlorine is to take place immediately after sampling in the field. If the test was performed in the laboratory, the hold time was exceeded. **(for aqueous matrices only)**

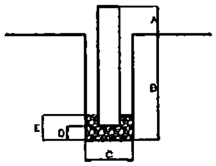
APPENDIX D

RESULTS OF PERCOLATION TESTS

PERCOLATION TEST LOG
P-1

Project: Percolation Testing - Scottish Rite
Location: San Diego, CA

Project No. G84101.01
Tested on 12/23/2019



A. Top of Pipe Above Ground	-8	Inches
B. Depth of Hole	40	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	2.5	Inches
E. Total Gravel Layer Depth	18.5	Inches
F. Pipe Length	30	Inches
G. Pipe Diameter	2	Inches

Pre-saturated:
Checked — Water at 2.32' below top of casing prior to adding water

Gravel Correction Factor: 2.6

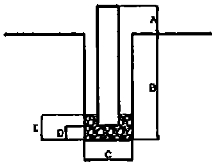
Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Percolation Rate Corrected for Gravel (minutes per inch)	Estimated Unfactored Infiltration Rate, (inches per hour)
1	Dec. 23, 2019	8:45:00	1.36				
	Dec. 23, 2019	9:15:00	1.38	30	0.24	319.9	0.01
2	Dec. 23, 2019	9:17:00	1.36				
	Dec. 23, 2019	9:47:00	1.38	30	0.24	319.9	0.02
3	Dec. 23, 2019	9:48:00	1.36				
	Dec. 23, 2019	10:18:00	1.38	30	0.24	319.9	0.02
4	Dec. 23, 2019	10:19:00	1.35				
	Dec. 23, 2019	10:49:00	1.36	30	0.12	639.9	0.01
5	Dec. 23, 2019	10:51:00	1.36				
	Dec. 23, 2019	11:21:00	1.37	30	0.12	639.9	0.01
6	Dec. 23, 2019	11:22:00	1.36				
	Dec. 23, 2019	11:52:00	1.37	30	0.12	639.9	0.01
7	Dec. 23, 2019	11:53:00	1.36				
	Dec. 23, 2019	12:23:00	1.37	30	0.12	639.9	0.01
8	Dec. 23, 2019	12:24:00	1.36				
	Dec. 23, 2019	12:54:00	1.37	30	0.12	639.9	0.01
9	Dec. 23, 2019	14:15:00	137	81	0	0.0	0.00

* Depth to water measured from top of pipe

PERCOLATION TEST LOG
P-2

Project: Percolation Testing - Scottish Rite
Location: San Diego, CA

Project No. G84101.01
Tested on: 12/23/19



A. Top of Pipe Above Ground	-11 Inches
B. Depth of Hole	98.5 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	17.5 Inches
F. Pipe Length	90 Inches
G. Pipe Diameter	2 Inches

Pre-saturated:
Checked

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Percolation Rate Corrected for Gravel (minutes per inch)	Estimated Unfactored Infiltration Rate, (inches per hour)
1	Dec. 23, 2019		0.8	NO MEASURABLE DROP IN WATER LEVEL DURING THE MONITORING INTERVAL OF APPROX. 6 HOURS			
2							

* Depth to water measured from top of pipe

APPENDIX E

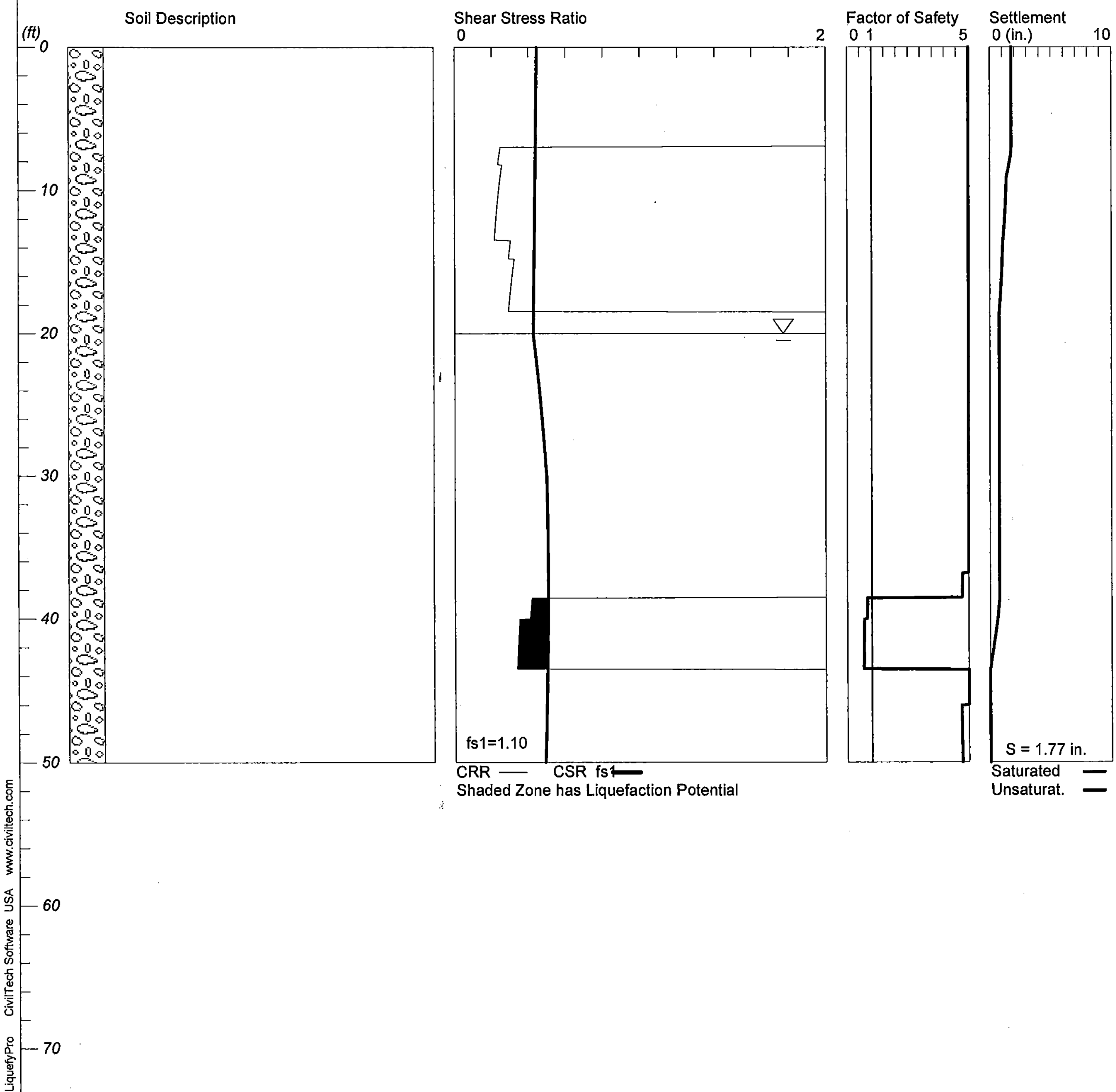
RESULTS OF LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

Scottish Rite Blding

Hole No.=B-1   Water Depth=20 ft

Magnitude=6.89
Acceleration=0.624g



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LIQUEFACTION ANALYSIS SUMMARY

****

Font: Courier New, Regular, Size 8 is recommended for this report.
Licensed to , 1/13/2020 5:40:19 PM

Input File Name: F:\ENG\Geotech\G84101.01 Scottish Rite San Diego\Liquefaction\B-1 WO SILTS and HE of 1.2.liq
Title: Scottish Rite Blding
Subtitle:

Surface Elev.=
Hole No.=B-1
Depth of Hole= 50.00 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 29.00 ft
Max. Acceleration= 0.62 g
Earthquake Magnitude= 6.89

Input Data:
Surface Elev.=
Hole No.=B-1
Depth of Hole=50.00 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 29.00 ft
Max. Acceleration=0.62 g
Earthquake Magnitude=6.89
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.2
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.1
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: No
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	21.00	125.00	35.90
7.00	7.00	125.00	30.80
13.50	11.00	125.00	30.80
18.50	9.00	125.00	NoLiq
23.50	5.00	125.00	NoLiq
28.50	16.00	125.00	NoLiq
29.00	6.00	125.00	NoLiq
31.00	10.00	125.00	NoLiq
36.75	30.00	125.00	30.00
38.50	20.00	125.00	30.00
40.00	18.00	125.00	30.00
43.50	8.00	125.00	NoLiq
46.00	27.00	125.00	30.00
48.50	34.00	125.00	30.00

Output Results:
Settlement of Saturated Sands=0.77 in.
Settlement of Unsaturated Sands=1.00 in.
Total Settlement of Saturated and Unsaturated Sands=1.77 in.
Differential Settlement=0.887 to 1.171 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.48	0.45	5.00	0.77	1.00	1.77
1.00	2.48	0.45	5.00	0.77	1.00	1.77
2.00	2.48	0.44	5.00	0.77	1.00	1.77
3.00	2.48	0.44	5.00	0.77	1.00	1.77

4.00	2.48	0.44	5.00	0.77	1.00	1.77
5.00	2.48	0.44	5.00	0.77	1.00	1.77
6.00	2.48	0.44	5.00	0.77	0.99	1.76
7.00	0.25	0.44	5.00	0.77	0.99	1.76
8.00	0.23	0.44	5.00	0.77	0.85	1.63
9.00	0.25	0.44	5.00	0.77	0.62	1.40
10.00	0.24	0.44	5.00	0.77	0.56	1.33
11.00	0.23	0.43	5.00	0.77	0.51	1.28
12.00	0.22	0.43	5.00	0.77	0.45	1.22
13.00	0.22	0.43	5.00	0.77	0.36	1.13
14.00	0.30	0.43	5.00	0.77	0.29	1.06
15.00	0.32	0.43	5.00	0.77	0.24	1.01
16.00	0.31	0.43	5.00	0.77	0.19	0.96
17.00	0.30	0.43	5.00	0.77	0.12	0.90
18.00	0.29	0.43	5.00	0.77	0.05	0.82
19.00	2.00	0.43	5.00	0.77	0.00	0.77
20.00	2.00	0.43	5.00	0.77	0.00	0.77
21.00	2.00	0.43	5.00	0.77	0.00	0.77
22.00	2.00	0.44	5.00	0.77	0.00	0.77
23.00	2.00	0.45	5.00	0.77	0.00	0.77
24.00	2.00	0.46	5.00	0.77	0.00	0.77
25.00	2.00	0.47	5.00	0.77	0.00	0.77
26.00	2.00	0.47	5.00	0.77	0.00	0.77
27.00	2.00	0.48	5.00	0.77	0.00	0.77
28.00	2.00	0.49	5.00	0.77	0.00	0.77
29.00	2.00	0.49	5.00	0.77	0.00	0.77
30.00	2.00	0.50	5.00	0.77	0.00	0.77
31.00	2.00	0.50	5.00	0.77	0.00	0.77
32.00	2.00	0.50	5.00	0.77	0.00	0.77
33.00	2.00	0.50	5.00	0.77	0.00	0.77
34.00	2.00	0.50	5.00	0.77	0.00	0.77
35.00	2.00	0.50	5.00	0.77	0.00	0.77
36.00	2.00	0.50	5.00	0.77	0.00	0.77
37.00	2.40	0.50	4.75	0.77	0.00	0.77
38.00	2.39	0.50	4.74	0.77	0.00	0.77
39.00	0.41	0.50	0.82*	0.72	0.00	0.72
40.00	0.41	0.50	0.81*	0.62	0.00	0.62
41.00	0.34	0.50	0.69*	0.45	0.00	0.45
42.00	0.34	0.50	0.68*	0.28	0.00	0.28
43.00	0.34	0.50	0.67*	0.10	0.00	0.10
44.00	2.00	0.50	5.00	0.00	0.00	0.00
45.00	2.00	0.50	5.00	0.00	0.00	0.00
46.00	2.00	0.50	5.00	0.00	0.00	0.00
47.00	2.33	0.49	4.71	0.00	0.00	0.00
48.00	2.32	0.49	4.72	0.00	0.00	0.00
49.00	2.32	0.49	4.72	0.00	0.00	0.00
50.00	2.31	0.49	4.73	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)
CRRm Cyclic resistance ratio from soils
CSRsf Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S. Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat Settlement from saturated sands
S_dry Settlement from Unsaturated Sands
S_all Total Settlement from Saturated and Unsaturated Sands
NoLiq No-Liquefy Soils

SPT CAL

SPT HAMMER ENERGY MEASUREMENTS

Prepared by;

SPT CAL
5512 Belem Dr
Chino Hills, CA 91709

909-730-2161
bc@sptcal.com

Prepared for;

Baja Exploration
1915 Commercial St.
Escondido, CA 92029

760.743.7678

Date: 09/13/2019

Project Title: Baja Exploration 2019

Project Description: CME 95

Energy Transfer Ratio = 70.6% at 43.7 blows per minute

Testing was performed on September 13, 2019 in Escondido, California

Hammer Energy Measurements performed in accordance to ASTM D4633 using an approved and calibrated SPT Analyzer from Pile Dynamics, Inc.

PRESENTATION OF SPT ANALYZER TEST DATA

1. Introduction

This report presents the results of SPT Hammer Energy Measurements recorded with an SPT Analyzer from Pile Dynamics carried out on September 13, 2019 in Escondido, California.

2. Field Equipment and Procedures

The drill used is a CME 95. It has an attached CME Automatic Hammer. The CME Automatic Hammer uses a 140 lb. weight dropped 30” on to an anvil above the bore hole. The drill rod connects the anvil to a split spoon type soil sampler inside an 8” o.d. hollow stem auger at the designated sample depth. After a seeding blow the sampler is driven 18”. The number of blows required to penetrate the last 12" is referred to as the “N value”, which is related to soil strength.

The first recording was taken at 5' below ground surface and then every 5' to final recording at 25’.

3. Instrumentation

An SPT Analyzer from Pile Dynamics was used to record and the process the data. The raw data was stored directly in the SPT Analyzer computer with subsequent analysis in the office with PDA-W and PDIPlot software. The measurements and analysis were conducted in general accordance with ASTM D4945 and ASTM D6066 test standards.

The SPT Analyzer is fully compliant with the minimum digital sampling frequency requirements of ASTM D4633-05 (50 kHz) and EN ISO 22476-3:2005 (100 kHz), as well as with the low pass filter, (cutoff frequency of 5000 Hz instead of 3000 Hz) requirements of ASTM D4633-05. All equipment and analysis also conform to ASTM D6066.

A 2' instrumented section of AWJ rod, with two sets of accelerometers and strain transducers mounted on opposite sides of the drill rod, was placed below the anvil. It measured strain and acceleration of every hammer blow. The SPT Analyzer then calculates the amount of energy transferred to the rod by force and velocity measurements.

4. Observations

The drill rig motor is diesel fueled. It had an electric throttle control which keeps the rpms stable. The drill and sample equipment looked to be well operated and maintained.

5. Results

Results from the SPT Hammer Energy Measurements are summarized below. It shows the Energy Transfer Ratio (ETR) at each sampling depth. ETR is the ratio of the measured maximum transferred energy to rated energy of the hammer which is the product of the weight of the hammer times the height of the fall. 140 lb x 30” = 4200 lb-in = 0.350 kip-ft.

Energy Transfer Ratio = 70.6% at 43.7 blows per minute

N60=(ETR/60)N

Depth	ETR%	BPM
5	70.1	44.1
10	70.7	43.8
15	71.0	42.9
20	69.9	43.4
25	71.2	44.5
Average	**70.6**	**43.7**

If you have any questions please do not hesitate to call or email.

Thank you,

Brian Serl
Calibration Engineer
SPT CAL
909-730-2161
bc@sptcal.com

MOORE TWINING ASSOCIATES, INC.

Since 1898

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED HOME DEPOT STORE - MISSION VALLEY

1895 CAMINO DEL RIO SOUTH

SAN DIEGO, CALIFORNIA

Project Number: D050R0.01

For:

Home Depot U.S.A., Inc.
4000 West Metropolitan Drive
Orange, CA 92868

January 10, 2020

January 10, 2020 D050R0.01

Home Depot U.S.A., Inc.
4000 West Metropolitan Drive
Orange, CA 92868

Attention: Mr. Bob Burnside

Subject: **Preliminary Geotechnical Engineering Investigation Report**
New Home Depot Store - Mission Valley
1895 Camino Del Rio South
San Diego, California

Dear Mr. Burnside:

We are pleased to submit this preliminary geotechnical engineering investigation report prepared for the Entitlement phase of the project to develop a proposed Home Depot store to be located at 1895 Camino Del Rio South (Mission Valley area) in San Diego, California. This report is considered preliminary since the project details had not been finalized at the time this report was completed. The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations.

Since this report is considered preliminary for Entitlement review, it is recommended that Moore Twining Associates, Inc. (Moore Twining) be provided with updated plans that pertain to the anticipated grading and structure details. Once these details are provided, a design level geotechnical report should be prepared to provide specific recommendations for design and construction.

In addition, it is recommended that Moore Twining be retained to final plans and specifications, as well as to conduct inspection and testing services for the excavation, earthwork, and foundation phases of construction. These services are necessary to determine if the subsurface conditions are consistent with those used in the analyses and formulation of recommendations for this investigation, and if the construction complies with our recommendations. These services are not, however, part of this current contractual agreement.

We appreciate the opportunity to be of service to Home Depot U.S.A., Inc. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

MOORE TWINING ASSOCIATES, INC.

Scott W. Krauter, RGE
Assistant Manager
Geotechnical Engineering Division

REGISTERED PROFESSIONAL ENGINEER
SCOTT KRAUTER
No. C049956
No. GE-2358
Exp. 12/31/20
CIVIL-GEOTECHNICAL
STATE OF CALIFORNIA

EXECUTIVE SUMMARY

Moore Twining Associates, Inc. (Moore Twining) was authorized by Home Depot U.S.A., Inc. to conduct a preliminary geotechnical engineering investigation for the proposed Home Depot store to be located at 1895 Camino Del Rio South (Mission Valley area) in San Diego, California. The subject property comprises a 14.05 acre parcel.

The subject property was being utilized as a Scottish Rite Event Center at the time of our investigation. The existing facility includes an events center building in the east portion of the site that occupies about 70,000 square feet. An existing asphalt concrete parking lot occupies the central and west portions of the site. The proposed Home Depot site/store extends into a portion of the auto dealership parking lot west of the existing Scottish Rite parking lot.

The project will involve the demolition of the existing Scottish Rite Event Center and associated parking lot to construct a new Home Depot store. Current plans indicate the store footprint will occupy about 106,688 square feet with a 17,913 square foot Garden Center. The planned store structure will extend north from the base of the existing cut slope (separated by a 35 foot driveway) across areas currently occupied by the Scottish Rite building and main parking lot. The new store will be served by a new three level parking garage to be located between the proposed store structure and Camino Del Rio South (roadway). The store will include a tool rental center, a lumber canopy and a depressed loading dock. The remainder of the site will generally be covered with asphalt concrete and Portland cement concrete paving.

A total of one hundred-twelve (112) test boring were drilled for this preliminary investigation. From February 25 to March 14, 2019, eighty-eight (88) test borings were drilled in the store building and site areas to depths of between 2 and 51 feet below site grade (BSG). After this initial drilling, a supplemental field investigation for the proposed parking structure was completed on September 17 through 20, 2019 and on December 27, 2019 to drill an additional twenty-four (24) test borings in the area of the proposed parking garage. It should be noted that auger refusal due to cobbles and dense gravels were encountered at depths of 10 feet or less in thirty-nine (39) of the borings drilled.

The near surface soils within the proposed Home Depot store were generally found to be stiff lean clays and loose clayey sands to depths of about 1 to 3 feet. These upper soils were likely disturbed native soils resulting from the extensive cut and fine grading of the existing parking lot areas of the site when developed in the 1950's. These upper loose, or stiff disturbed soils, will not provide uniform support for the proposed floor slabs or foundations. As a part of site preparation, these loose soils should be excavated to expose undisturbed native soils and in order to support the proposed foundations on engineered fill. Below these upper soils, similarly classified sandy clay and clayey sand soils were encountered in hard and dense conditions in the range of 3 to 10 feet below site grade (BSG).

EXECUTIVE SUMMARY (Continued)

The soil conditions in the north portion of the site proposed for the Parking Structure are highly variable and appear to be less consolidated (weaker) compared to the area of proposed for the Home Depot Store. Soils consisting of loose to medium dense silty sands and stiff clays with more dense and hard soil profiles in adjacent borings suggest more variable conditions. Based on the higher compressible soil conditions expected, the parking garage will require deeper over-excavation depending on column loads of the final design or a ground modification program such as Geopiers could be a effective method to densify the upper variable soils to reduce foundation settlements to tolerable levels. Also, the structure could be supported on a continuous mat type foundation to reduce applied soil bearing pressures and to resist higher levels differential settlements expected by the variable conditions.

The on-site clay soils encountered have a medium expansion potential as indicated by expansion index values of 77 and 81. Medium expansive material would cause heave/shrinkage exceeding ½ inch in 50 feet resulting in post construction damage to lightly loaded slabs on grade supported directly on these materials. Therefore, it is recommended to support floor slabs on non-expansive aggregate base and imported non-expansive granular fill; and, extend perimeter foundations below where seasonal moisture fluctuations typically occur.

Variable amounts of fine to coarse gravel and cobbles are present within the lean clay/clayey sand strata encountered at the site. These soils with coarse materials are usually characterized by hard or very dense conditions on the boring logs (N-values greater than 50 blows per foot). These hard and dense conditions and coarse gravel and cobble materials will require more effort to excavate and process than typical soils without coarse materials. Further, oversized materials placed and/or compacted directly below foundations and floor slabs can cause hard points resulting in excessive differential movement and cracking of over-lying footings or slabs on grade. Due to the presence of cobbles and gravel, oversized rock material should be removed by methods such as screening prior to placement and compaction as engineered fill.

A 4.66 acre area of the south portion of the property is occupied by a north facing hillside. Also, an ascending west facing cut slope is also present along the south portion of the east boundary of the site between the adjacent church property above the site. Based on aerial images and the site topographic exhibit provided, the lower portion of the north facing slope, in the southern portion of the site, is a cut slope with graded terraces and a native hillside above. The native hillside extends hundreds of feet above and beyond the subject property line to the south. The total height to the top of the slope, which is located beyond the property line, is estimated to be about 285 feet above the base of the slope, with the upper native slope occupying about 210 vertical feet and the lower graded cut slope occupying about 75 vertical feet of the overall slope.

The lower portion of the existing cut portion of the north-facing slope was observed and evidence of previous erosion, shallow soil slips, remedial erosion control measures, surficial slope repairs, and drainage improvements added after initial construction were noted. The majority of the erosion and soil movement observed was identified in the eastern portion of the north facing slope. In this eastern portion of the slope, the slope was not covered with mature bushes or established native grasses and evidence of significant erosion, shallow sliding of surface materials, failed erosion control measures, and accumulation of sediments were noted. However, the western portion of the north facing cut slope did not exhibit significant erosion issues or evidence of surficial instability. This area of the slope contained better established vegetation.

The existing north facing native slope above the cut slope area was evaluated to identify unfavorable geologic structures as a part of this investigation. No unfavorable geologic structures were identified and this upper native slope has been performing well for quite some time. Therefore, the slope is considered stable and potential instability of the upper native slope is low.

The existing north facing lower cut was inspected above and below the slope. These observations did not identify evidence of scarps, lateral displacement, bulging at the base (retaining wall displacement), or unfavorable geologic structures suggesting that any deep seated instability of the overall slope had occurred. Further, deeper soils encountered in the borings drilled on the exhibited good shear strength characteristics. Given these conditions and the overall 2H to 1V slope across the cut, it was concluded that deep seated slope instability is not a concern since the project does not propose to significantly alter the existing cut slope.

The existing north facing cut slope has an area that has been impacted from past washouts, with exposed cobble deposits, and exposed predominantly granular, low cohesion, soils that have exhibited high erosion and shallow soil slips 1 to 2 feet deep. Also, observations indicate the slope drainage needs improvement. Drainage improvements will reduce, but not eliminate the surficial and erosion issues that have occurred. Thus, some surficial slope movements are anticipated to continue. Considering that the building improvements are planned to be setback at least 35 feet from the slope, impacts to the proposed structures due to shallow slope instability are not anticipated. The current approach by Scottish Rite of maintenance and spot repairs where erosion and slippage has occurred on the cut slopes has been sufficient to maintain function. A similar level of maintenance and repair should be anticipated. In addition, this report recommends that a program of regular inspection of the slopes be implemented to identify conditions that could further degrade shallow slope stability, and to identify areas requiring maintenance and repair/restoration.

An inlet structure which collects runoff from a side canyon area within above and to the south of the site has become blocked in the past, causing runoff to flow around or over the inlet structure. The runoff appears to have drained onto the adjacent terraces and flowed over the north facing cut slopes in the past, contributing to erosion and surficial soil slips within the lower portion of the cut slope. Therefore, it has been concluded that the current drainage inlet structure and the maintenance (i.e., debris removal) are not adequate for the runoff conditions experienced. Thus, to reduce the impacts associated with the blocking of the current inlet structure at the outlet of the side canyon, appropriate debris catchments and inlet structure design should be incorporated into the drainage improvements as a part of construction. The drainage structure and catchments should include redundant systems to reduce the potential for clogging.

EXECUTIVE SUMMARY (Continued)

Also, a variable height cut slope that supports the elevation transitions up to the adjacent church property along the east boundary of the site was observed. In addition, considering that no unfavorable geologic structures were identified and the existing west-facing cut slope has performed

well for quite some time, there is also a low potential for impacts from movement of this slope. Although significant slope movement is not anticipated, it is recommended to provide a minimum setback of at least ½ the slope height from the toe of the slope to the nearest structure.

The results of the liquefaction analyses indicate that some medium dense silty sands encountered in two of the five deeper areas explored at the site below 30 to 40 feet are susceptible to liquefaction in isolated zones. The associated differential seismic settlements were estimated to be ½ inch in the Home Depot store, and ¾ inch within the parking garage.

The results of the R-value tests indicate the near surface soils exhibit poor to good pavement support characteristics as indicated by R-value results ranging from 19 to 22 for most of the clay soils with a result of 63 in some isolated silty sands. Based on the R-values conducted for this investigation, an R-value of 15 was used for design.

Chemical testing of soil samples indicated the soils exhibit a "highly corrosive" to "corrosive" potential for metallic corrosion and a "negligible" potential for sulfate attack on concrete placed in contact with the near surface soils.

This executive summary should not be used for preliminary design and should be reviewed in conjunction with the details included in the attached report.

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED HOME DEPOT STORE - MISSION VALLEY
1895 CAMINO DEL RIO SOUTH
SAN DIEGO, CALIFORNIA

Project Number: D05R0.01

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical engineering investigation for the proposed Home Depot store to be located at 1895 Camino Del Rio South in the Mission Valley area of San Diego, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Home Depot U.S.A., Inc. to perform this investigation. This report was prepared for Entitlement purposes.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, existing site features, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The report appendices contain the drawings and site photographs (Appendix A), the logs of borings (Appendix B), the results of laboratory tests (Appendix C), the results of percolation tests (Appendix D), and the results of liquefaction analysis (Appendix E).

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The purpose of the preliminary investigation was to conduct a field exploration, a laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following:

2.1.1 Evaluation of the near surface soils within the zone of influence of the proposed foundations, exterior slabs-on-grade, and pavements with regard to the Home Depot design criteria;

2.1.2 Conclusions regarding the potential for liquefaction, magnitude of seismic settlement, and recommendations for CBC seismic near source factors and coefficients;

2.1.3 Preliminary geotechnical parameters for use in design of foundations and slabs-on-grade, (e.g., soil bearing capacity and settlement), and development of lateral resistance;

2.1.4 Preliminary recommendations for site preparation including placement, moisture conditioning, and compaction of engineered fill soils;

2.1.5 Assessment of the infiltration characteristics of the soils in the proposed infiltration system location;

2.1.6 Preliminary evaluation of the stability of the existing adjacent slopes;

2.1.7 Recommendations for the design and construction of new asphaltic concrete (AC) and Portland cement concrete (PCC) pavements;

2.1.8 Recommendations for temporary excavations and trench backfill; and

2.1.9 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed project referenced in the Anticipated Construction section of this report. This report does not include recommendations for offsite improvements. This investigation did not include a floodplain investigation, quantitative slope stability analysis, environmental investigation, or environmental audit.

2.2 **Scope:** Our proposal, dated December 17, 2018 outlined the original scope fo services and contract amendments No.1, dated August 23, 2019 and No. 2 dated December 9, 2019, outlined supplemental scopes of our services. It was not the intent of this investigation to fully comply with the Home Depot Design Manual requirements for the number of borings on the site since soil borings could not be conducted within the existing building nor within the existing hillside areas that could not be accessed with exploration equipment. Therefore, the spacing of the soil borings conducted in some areas of the site was not intended to comply with the Home Depot Criteria in all areas. The actions undertaken during the investigation are summarized as follows.

2.2.1 The Home Depot Design Criteria Manual (dated October 17, 2016) was reviewed.

2.2.2 The City of San Diego Guidelines for Geotechnical Reports, (2018) and the City of San Diego, Storm Water Standards (2018) were reviewed

2.2.3 Several versions of site plans for the proposed project were provided for review during the investigation prepared by Lars Andersen & Associates, Inc. The initial field exploration program utilized a site plan (LA-G.2) revision dated February 19, 2019 prepared by Lars Andersen & Associates, Inc. After initiation of the investigation, revised site plans showing the parking structure and updated preliminary building and site improvements was provided by Lars Andersen & Associates, Inc, dated December 12, 2019. This plan is referred to as the site plan in this report.

2.2.4 San Dieguito Engineering, Inc. provided several versions of slope maintenance and improvement plans dated March 26, 2019 through January **, 2020 that were reviewed. Also, an exhibit showing existing topography of the site, prepared by K&S Engineering, Inc. was provided for review and reference to existing site elevations.

2.2.5 Historical aerial photographs of the site and surrounding area, produced by EDR, for the years 1928, 1949, 1953, 1964, 1966, 1970, 1979, and 1985 were reviewed.

2.2.6 Research regarding the existing site and regional geology was conducted, and the following maps and reports were reviewed and utilized during this investigation:

- Geologic Map of the San Diego 30'x60' Quadrangle, California, Regional Geologic Map Series, prepared by the California Geological Survey and compiled by Michael P. Kennedy and Siang S. Tan, dated 2008;

- City of San Diego's Seismic Safety Study, Geologic Hazards and Faults, Grid Title 21, dated April 3, 2008;

- California Division of Mines and Geology (CDMG) Open File report 95-03 by Tan (1995) titled Landslide Identification Map No. 33; Landslide Hazards in the Southern Part of the San Diego Metropolitan Area; and,

- Revised Desktop Geotechnical Geotechnical and Geologic Hazard Evaluation, prepared by The Bodhi Group, dated January 8, 2019.

2.2.7 City of San Diego Building Records were reviewed to identify geotechnical engineering investigation reports prepared for previous developments along the south flank of Mission Valley. These reports are identified in Section 5.1.4 of this report.

2.2.8 Boring permit numbers LMWP-003844 and 004155 were obtained from the County of San Diego for the two phases of subsurface investigation conducted.

2.2.9 Visual site reconnaissance and subsurface exploration were conducted.

2.2.10 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils encountered.

2.2.11 Mr. Bob Burnside (Home Depot), Mr. Scott Mommer (Lars Andersen Associates), Mr. Michael Wolfe (San Dieguito Engineering, Inc.), Mr. Brian James (James Company), and representatives from the Scottish Rite Events

Center were consulted during the investigation. Also, for parking garage information, Mr. Jason Rupp (Architects Orange, LLP) and Mr. Bryan Allred (Seneca Structural Engineering, Inc.) were consulted.

2.2.12 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and engineering properties of the subsurface soils.

2.2.13 This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, and preliminary evaluation, as well as preliminary conclusions and recommendations.

3.0 BACKGROUND INFORMATION

The existing site features, site history, previous studies, and the anticipated construction are summarized in the following subsections.

3.1 Site Description: The subject site comprises a 14.05 acre parcel located at 1895 Camino Del Rio South in the City of San Diego, California (see Drawing Nos. 1 and 2 in Appendix A). For the purpose of this report, project north is considered to be towards Camino Del Rio South, which is about 15 degrees to the west of true north. The site is bordered to the north by Camino Del Rio South, which is a frontage road for Interstate 8 beyond; to the west by an existing auto dealership; to the east by an office building and an ascending slope and church facility beyond; and to the south by an ascending slope and a residential neighborhood beyond that has an elevation about 285 feet higher than the relatively flat portion of the subject site. Descriptions of the slopes on and near the site are provided in Section 5.3 of this report.

The subject property was being utilized as a Scottish Rite Event Center at the time of our investigation. The existing facility includes an events center building in the east portion of the site that occupies about 70,000 square feet. An existing asphalt concrete parking lot occupies the central and west portions of the site. The proposed Home Depot site/store extends into a portion of the existing auto dealership parking lot west of the existing Scottish Rite parking lot.

The existing Event Center building is a single-story structure with ground floor levels that vary in elevation. Observation of the building exterior indicates most of the structure has reinforced concrete walls (tilt-up or prefabricated). However, some portions of the existing building in the south and east portions were noted to have masonry walls possibly associated with additions or remodel of the original structure. Observations of the interior of the building indicate that the floors are concrete slabs-on-grade with an elevated slab about 6 feet higher along the west portion of the building, and a lower level slab in the center and east portions of the building. Steps and elevated doorways to the exterior indicate that the interior floors are as much as 6 feet below the exterior grades along the south side of the structure. Thus, the existing south wall of the building acts as a retaining wall.

The subject site also includes some minor structures and sheds located east of the main Events Center structure between southeast driveway and the ascending slopes. Also, a masonry block retaining wall about 4 feet high was noted at the base of the ascending slopes south and east of the existing structures.

The other developed portions of the site are occupied by asphalt paved parking lots and driveways. Evidence of underground utilities was noted mostly in the area north and east of the existing Scottish Rite building. Other underground utilities, such as electrical for parking lot lighting, were noted. Overhead utility lines were also noted at the time of our field investigation.

A 4.66 acre area of the south portion of the property is occupied by a southerly ascending hillside which ascends well beyond the south property line. The lower portion of the slope within the subject property appears to have been previously graded (cut) and the upper portion of the slope generally appears to be native. Graded cut and fill slopes with terraces are present below the native slope, proximal to the proposed Home Depot store. The native hillside extends hundreds of feet horizontally beyond the subject property line to the south. The total height of the slope to the south of the proposed Home Depot building area is estimated to be about 285 feet, with the upper native slope occupying about 210 vertical feet and the lower graded cut slope occupying about 75 vertical feet of the overall slope.

A south-north trending natural drainage area (side canyon) which receives runoff from areas south of the site including a City Park, and the neighborhood above the site is directed into an inlet structure and drainage pipe in the southeast portion of the property above the developed portion of the site. The 30-inch diameter collection pipe is located in a drainage easement that curves west of the existing events center building and runs north below the existing parking lot to carry the drainage from the side canyon offsite to the north (see Drawing No. 2 in Appendix A of this report).

Also, a west facing hillside is located east of the proposed Home Depot building. More detailed descriptions of this slope, drainage, and features are presented in the Findings Sections of this preliminary report.

3.2 **<u>Site History and Previous Studies</u>**: It is our understanding that the existing site was originally developed as a bowling alley in the 1950's and that the site use was converted to the existing Scottish Rite Event Center in the 1970's.

A historical aerial photograph from 1928 shows most of the site as undeveloped sloped rangeland, with some small scale agricultural activities noted along the north edge of the site. The 1928 image shows the existing natural drainage course (side canyon) with some scattered trees traversing south to north across the eastern quarter of the site. Native bushes and trees provide a relatively dense cover on the slope above the site, while the site area appears covered with grasses. With the exception of the drainage area, the south slope appears as a broad sloped south boundary (bluff) of the San Diego River Valley. North of the site, a two-lane road is present with undeveloped areas on the banks of the San Diego River which is further north.

The next available aerial image from 1949 shows that some grading of the site had begun by cutting into the slope to the south. A 1953 aerial image shows the south slope had been cut with terraces and the cut slope area appears to be exposed with no vegetation cover. Also, it was noted that the properties east and west of the site were still native and had not been graded.

The next available aerial image from 1964 shows cut grading of the south slope had been completed and the bowling alley structure appears to be under construction and nearing completion (parking lot was not paved). Also, the commercial building and church east of the site appear to be under construction. The church development also includes a cut slope at the toe of the native hillside to the south. The image shows a completed Camino Del Rio South roadway and adjacent multi-lane freeway with a shopping center beyond between the freeway and the San Diego River.

The 1966 aerial image shows the bowling alley in use. It was noted that the original main building was smaller than the current facility. The area southeast of the building and below the east slope was occupied by a small parking lot. A 1970 image shows a building addition on the east side of the original building. This configuration of the site is shown in aerial images through 1979. However, between 1979 and 1985, the detached minor structures and sheds were added to the site at the base of the ascending east slope. After 1985, the site appears to have been unchanged.

Additionally, it was reported by Scottish Rite staff that the James Company had been retained over the years to repair portions of the south slope from erosion. At the time of our field investigation, James Company had equipment mobilized onto the slope through a temporary access route from the parking lot. Mr. James reported that some recent repairs of smaller washouts had been completed in the central area of the slope.

Further descriptions of the slope observations, including a description of observed soil slips, erosion etc., are included in Section 5.3 of this report.

No other geotechnical or environmental assessment reports had been provided to Moore Twining at the time of this investigation. If available, these reports should be provided to Moore Twining for review.

3.3 **<u>Anticipated Construction</u>**: Based on review of the site plan for the proposed project, the existing events center building will be demolished and a Home Depot store will be constructed on the site. The current plans indicate the store footprint will occupy about 106,688 square feet with a 17,913 square foot garden center. The planned structure will be located about 35 feet from the toe of the south slope approximately as noted on Drawing No. 2 in Appendix A of this report. A parking lot and parking structure will be constructed between the store and Camino Del Rio north of the site. The Home Depot store will include a tool rental center, a lumber canopy and a depressed loading dock. The remainder of the site will generally be covered with asphalt concrete and Portland cement concrete paving.

It is expected the store building will include concrete tilt-up perimeter walls, a steel frame roof structure supported on isolated interior columns spaced about 50 feet apart. The October 17, 2016 Home Depot Design Criteria Manual indicates maximum column loads of about 76 kips and wall loads of about 4.6 kips per foot for a prototype store. The maximum uniform floor slab load for the slab-on-grade sales floor area will be 325 pounds per square foot. According to the Home Depot Design Criteria, maximum allowable total settlement for floor slabs and foundations shall not exceed 1 inch. The maximum allowable differential settlement for floor slabs and foundations shall not exceed ½ inch in 50 lineal feet. In addition, the maximum total heave of the floor slab and foundations shall not exceed 1 inch and the maximum differential heave of the floor slab and foundations shall not exceed ½ inch.

At this preliminary stage of development, the parking structure is expected to be a three level (two elevated levels plus an at-grade level) post tensioned concrete structure that will occupy about 60,000 square feet in plan area. The details of the parking structure are not known, but we understand one option could include widely spaced interior columns with dead loads as much as 360 kips and live loads of 190 kips supported on shallow spread foundations. However, an alternate structure with more interior columns supported on a continuous mat type foundation was also being considered. The parking structure design engineer reported tolerable settlements of 1½ inches total; and ¾ inch differential for the structure.

The proposed development will include driveways and parking for automobile and truck traffic. Equivalent 18 kip axle loads (EAL) of 50,000 and 220,000 for a design life of 10 years were stated in the Design Criteria Manual for the Home Depot "standard duty" and “heavy duty” pavement sections, respectively.

At the time this preliminary report was issued, the latest version of the grading plan (December 12, 2019.) indicates a finished floor elevation for the Home Depot store and garden center of 52.50 feet AMSL with a finished pad grade of 51.50 feet AMSL is being proposed. Based on the contour elevations in this area, cuts up to 5 feet are anticipated along the south wall of the building with the north wall in less than 1 foot of cut or fill to grade the building pad.

Also, improvements to the existing slope south of the site are planned to improve drainage, provide erosion protection and to improve shallow slope instability. These improvements may include new lined (concrete or asphalt) brow and terrace ditches, debris fences, drainage structures, etc. A new drainage inlet structure is also planned at the outlet of the side canyon to collect runoff. In addition, the reference plans indicate the existing drainage pipe and easement which trends through the center of the site will be abandoned and relocated to extend along the south driveway and to the west of the proposed store.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing programs conducted for this investigation are summarized in the following subsections.

4.1 Field Exploration: The field exploration consisted of a site reconnaissance, drilling test borings, conducting standard penetration tests, soil sampling, and percolation testing.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. A site reconnaissance was conducted by Mr. Scott Krauter (Geotechnical Engineer with Moore Twining) on February 20, and 21, 2019. A site reconnaissance was also conducted by Mr. Ken Clark (Certified Engineering Geologist with Moore Twining) on June 7, 2019. Also, site reconnaissance was conducted by staff geologists and engineers during the drilling operations. The features noted are described in the background information.

During our site reconnaissance, two (2) areas of the existing cut slopes that had exposed native soils (devoid of vegetation) were logged by a Moore Twining staff geologist. The approximate locations of the exposed cut slopes which were logged are noted on Drawing Nos. 2 and 3 in Appendix A. Descriptions of the soils logged and sequence of strata are illustrated on Drawing Nos. 8 and 9 included in Appendix A of this report.

4.1.2 Drilling Test Borings: The number of soil borings drilled in the proposed building area was based on the general requirements of Section 9 of the Home Depot Design Criteria Manual for geotechnical engineering investigations based on the areas which were accessible to exploration equipment at the site. The spacing of the borings drilled for this investigation was generally 40 feet in the proposed Home Depot building and parking structure area; and 80 feet in the parking lot and accessible slope areas.

A total of one hundred-twelve (112) test borings were drilled for this preliminary investigation during two separate phases of the investigation. The initial field investigation was conducted from February 25 to March 14, 2019, and included drilling eighty-eight (88) test borings in the store building and site areas to depths of between 2 and 51 feet below site grade (BSG). At the time of the initial investigation, a parking structure was not planned as part of the development. After the initial drilling, a supplemental field investigation for the proposed parking structure was conducted on September 17 through 20, 2019 and December 27, 2019 to drill an additional twenty-four (24) test borings in the area of the proposed parking structure. It should be noted that auger refusal due to cobbles and dense gravels were encountered in seventy-two (72) of the borings drilled before the intended maximum depth of exploration was achieved.

These test borings were drilled using a CME-75 drill rig equipped with 6⅝-inch outside diameter (O.D.) hollow-stem augers and a Fastre SPT track mounted rig equipped with 6 inch outside diameter hollow stem augers. Also, to penetrate deeper gravel and cobble materials in an attempt to explore to 50 feet BSG in the liquefaction zone, a larger higher capacity Marl Industries Yeti-10 drilling rig was used to extend a supplemental boring at location M-8 below the depth of auger refusal in a previous boring drilled at this location.

The test borings were drilled under the direction of a Moore Twining geotechnical engineer. The soils encountered in the test borings were logged. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil. Soil samples were collected and returned to our laboratory for classification and testing.

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of borings.

Test boring locations were determined by using a measuring wheel with reference to the existing site features. The locations, as shown on Drawing No. 2 in Appendix A, should be considered approximate. Elevations of the test borings were not surveyed as a part of the investigation since surveys were completed prior to completion of the borings. However, spot elevations and topographic data provided by the project civil engineer were interpolated to estimate the boring elevations to approximately one-half (½) foot. In accordance with the boring permits issued by the County of San Diego, the test borings were backfilled with neat cement. The neat cement backfill was capped with cold patch asphalt in the pavements areas. Some settlement should be anticipated at the boring locations.

4.1.3 **<u>Soil Sampling</u>:** Standard penetration tests were conducted in the test borings, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1⅜-inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by driving California modified split barrel ring samplers into the soil using a drill rig mounted 140 pound trip hammer. In addition, some relatively undisturbed soil samples of the soils exposed on the cut slopes were collected for laboratory tests were obtained by driving a split barrel ring sampler into the subgrade soil using a 35 pound hand operated slide hammer. The soil was retained in brass rings, 2.5 inches O.D. and 1-inch in height. The lower 6-inch portion of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory. Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

4.1.4 **<u>Percolation Test Holes and Testing</u>**: Based on the subsurface soil conditions encountered, and our consultation with San Dieguito Engineering, Inc., three (3) percolation tests were installed at depths of about 6 , 10 and 15 feet BSG along the east portion of the site frontage in the northeast portion of the parking lot (referenced boring locations J-8, I-8 and L-8 on Drawing No. 2).

The percolation tests were installed with a PVC pipe in the borings and the bottom of each boring was packed with gravel to stabilize the boreholes. The details of the test hole construction are shown on the percolation test sheets enclosed in Appendix D of this report.

The percolation tests were conducted on March 12 and 18, 2019. Percolation testing was performed in general accordance with Section D.3.3.2 - "Borehole Percolation Tests of the City of San Diego Storm Water Standards, dated October 1, 2018."

The percolation test holes were pre-saturated the day prior to conducting the tests. Percolation testing included adding water to the test holes periodically and measuring the drop in water level over time until a stabilized rate was measured. Measurements of water levels and the time of each reading were recorded during testing. The depth measurements versus time are presented on the percolation test sheets enclosed in Appendix D of this report.

4.2 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and relatively undisturbed samples representative of the subsurface materials.

The results of laboratory tests are summarized in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

5.0 FINDINGS AND RESULTS

The findings and results of the research, field exploration and laboratory testing are summarized in the following subsections.

5.1 Research: Several sources of information were reviewed as a part of this investigation. These sources included published geologic maps and seismic hazard data; historical aerial photographs; the City of San Diego Seismic Safety Study; and historic USGS 7½ minute topographic maps. Also, City of San Diego building records were researched to identify nearby geotechnical and geologic investigations that included information pertaining to the slopes along the south side of Mission Valley.

5.1.1 Past Site Grading: The site was graded over 65 years ago by cutting into the hillside to the south to establish a relatively flat area for the existing development. Prior to grading, aerial photographs indicate the south slope appeared to have relatively consistent grades east and west of the site, except for the side canyon drainage area noted in the east portion of the site.

An aerial photograph shows that the previous site grading generally occurred from at least 1949 through 1953. These same images show that the sloped areas east and west of the site were still native and had not been graded in that time period. The height of the slope east of the site (adjacent the church facility)provide an indication of the amount of material cut to grade the subject site. Topography indicates this east slope is as high as 50 feet at the base of the cut in the south portion of the pad, and reduces to at-grade in the north.

Review of the aerial photographs indicate the existing hillsides on properties to the west of the site were generally graded in a similar manner as the subject site by cutting in the 1950s to 1960s.

5.1.2 Geologic Setting and Site Geology: The site is located within the Peninsular Ranges geomorphic province. The project site is located on the southern edge of Mission Valley, which is a narrow valley cut by the west flowing San Diego River drainage. The San Diego River has cut the Mission Valley through older geologic formations which are described in the following sections. The river is also responsible for fluvial sediments deposited within the valley, including a part of the site. The referenced 1928 aerial photograph shows fluvial deposition north of agricultural fields, within a few hundred feet of the site.

The "Geologic Map of the San Diego 30' x 60' Quadrangle, California," prepared by the California Geological Survey and compiled by Michael P. Kennedy and Siang S. Tan, dated 2005, indicates the south portion of the site (including most of the south slope) is mapped as being underlain by Mission Valley Formation (Middle Eocene), and the northern portion of the site is mapped as underlain by younger colluvial deposits (Holocene and late Pleistocene).

Descriptions of local formations presented in Bulletin 200 - "Geology of the San Diego Metropolitan Area," prepared by Michael P. Kennedy and the California Division of Mines and Geology, dated 1975, indicate the Mission Valley Formation is a marine sandstone unit which is soft and friable with cobble conglomerate tongues comprising up to 30 percent of the section mapped. The formation description also indicates that interbeds and tongues of claystone of brackish water origin locally comprise 20 percent of the section.

Some loose soils deposited on the terraces that have been experiencing erosion appear to be relatively young colluvial soils.

The younger colluvial deposits in the north portion of the site are described as poorly consolidated, poorly sorted, permeable flood-plain deposits of sandy, silty or clay-bearing alluvium.

The Geologic Map of the San Diego 30' x 60' Quadrangle, California also indicates numerous bedding dips in the site region, measured in the Mission Valley Formation, the underlying Stadium Conglomerate, and the overlying San Diego Formation. These bedding dips predominantly range from about 2 to 5 degrees from horizontal in the general site vicinity. The portion of this regional geologic map showing the site location is presented on Drawing No. 4 in Appendix A. The referenced geologic map indicates the existing hillside within the south portion of the site has geologic conditions which are consistent with the existing hillside areas which border the south side of the site, and extending west and east of the site. These adjacent slopes to the west and east border numerous existing developed properties along the south side of Mission Valley.

Also, a site geologic map and cross section showing the Mission Valley and younger colluvial deposit geologic units identified are presented on Drawing Nos. 6 and 7 in Appendix A

5.1.3 Geologic Hazards: The City of San Diego Seismic Safety Study, "Geologic Hazards and Faults", was reviewed. The site is located on Grid Map 21 of the Hazard Map Series. The map shows the northern portion of the site in the area where the parking structure is planned is located within a zone of high potential liquefaction (category 31). However, the Home Depot store area is located outside the liquefaction hazard zone.

Based on Grid Map 21, the ascending slope area in the south portion of the site is located in a zone indicated as "sloping terrain, unfavorable geologic structure, low to moderate risk" (category 53). The map also indicates a concealed segment of a fault is located adjacent to the northeast corner of the site. However, the fault category is described as "Potentially Active, Inactive, Presumed Inactive, or Activity Unknown".

Also, as required by City of San Diego Geotechnical Report guidelines, the potential for tsunamis to impact the site were considered. The California State Department of Conservation published Tsunami Inundation Maps for San Diego County do not include the non-coastal site area. Due to the inland location and elevation of the site, tsunamis are not considered a significant hazard for the project.

5.1.4 Landslide Hazards: The subject site is located at the base of an ascending slope that forms the south flank of Mission Valley. The existing slope extends well beyond the limits of the subject property. Similar slope conditions occur within numerous developed properties to the east and west of the subject site.

Various geologic maps and reports were reviewed for background information with regard to the stability of the geologic materials within the subject slope. The geologic maps (see Drawing Nos. 4 through 7 in Appendix A) indicate the Mission Valley Formation comprises most of the hillside, with only thin sections of San Diego and Pomerado Formation conglomerates within the upper portion of the slope.

A California Division of Mines and Geology (CDMG) Open File report 95-03 by Tan (1995) titled *Landslide Identification Map No. 33; Landslide Hazards in the Southern Part of the San Diego Metropolitan Area*. This mapping indicates the hillside south of the site is "generally susceptible" to landsliding. The mapping identifies a slide area on an east facing slope within the side canyon which is located south of the site as having the "most susceptible" designation. See Drawing No. 5 in Appendix A for an excerpt of this landslide map in the vicinity of the site. However, no landslides are mapped within the subject site.

The City of San Diego Building Records were reviewed to identify geotechnical engineering investigation reports prepared for previous developments along the south flank of Mission Valley with generally similar topographic and geologic conditions as the subject site. Although numerous reports were reviewed, three geotechnical engineering reports were identified that included conclusions regarding the stability of the native and cut slopes along the south side of Mission Valley that possess similar geologic conditions as that of the subject site.

Lennart and Associates conducted a Soils Investigation for the adjacent First Methodist Church of San Diego in 1962. The property borders the subject site to the east, and includes a steep, north facing cut slope on the south side of the site which is mapped as Mission Valley Formation material. The report included an evaluation of proposed cut slopes that were extended into the "steeply sloping upper southerly site area" to accommodate the current church development. The report indicated the materials in this area of the slope were a rock material which was indicated to be "clastic sediment of Tertiary age, mainly sandstone and conglomerate." The description indicates the bedding is nearly level with a slight dip to the south (which is consistent with the geologic map referenced in Section 5.1.1 of this report). The report further states: "*The rock is well indurated, reasonably well cemented, and is resistant to erosion.*" The report further indicates that cut slopes in this rock can be graded to 1H to 1V with some shorter sections of 0.5H to 1V.

Research of more recent geotechnical engineering investigations identified two Professional Service Industries, Inc. geotechnical engineering reports for hotels that were constructed on Hotel Circle South. The sites are located about 2 miles west of the site, at the base of the slope on the south side of Mission Valley. A report for the Marriott Residence Inn California (1865 Hotel Circle South), dated February 28, 2000, and a report for a La Quinta Inn, dated November 30, 1997 were reviewed regarding the stability of the slopes to the south.

The Marriott Residence Inn report states the following with respect to the stability of the slope on the south side of Mission Valley:

> *"A relatively steep natural slope was observed to extend upward from the rear of the property at an approximate gradient of up to 1:1 (horizontal to vertical) to a maximum height of 160 feet. Review of geologic maps (Kennedy and Petersen, 1975) indicate the slope is composed of Stadium Conglomerate within the bottom third of the slope, with the Mission Valley Formation comprising the portion of the exposed slope face extending from the top of the Stadium Conglomerate to the crest of the slope. Both of these formations are generally considered stable with respect to landsliding and even steep slopes. This is due to several features of the slope formational units including: their composition (high percentage of sand and silt as opposed to clay); their moderate to high degree of cementation; their relatively high consolidated and cohesive nature; and their conformable and massive nature. Furthermore, although the Seismic Safety Study for the City of San Diego classifies the materials as possessing unfavorable geologic structure (Risk Category 53), we found the materials to exhibit favorable sub-horizontal structure, which is typically favorable with respect to slope stability."*

The La Quinta Inn report states the following with respect to stability of the native slope at the base of the slope on the south side of Mission Valley:

> "*Significant natural slopes, located on both the project site and along the southern perimeter of the site, were observed to have an approximate inclinations of 1:1 (horizontal to vertical) or steeper. However, it is our opinion that the potential for slope failure is relatively low. This opinion is based upon the sub-horizontal bedding of both the Stadium Conglomerate and Mission Valley Formations, the weakly to moderately cemented nature of these formational units, the conformable contact between theses formational units, and the fact that these formational materials, along with the encountered stiff/dense to hard/very dense slopewash/colluvial materials, are generally considered non-susceptible to slope failures, provided the earthwork recommendations in this report are followed. It should be noted that a detailed deterministic evaluation of the on-site and adjacent slope areas was not included within our scope of services, PSI would be pleased to provide such an evaluation, if required, upon request.*"

5.2 Surface Conditions in Existing Developed Area of Site: As noted in this report, at the time of our field investigation, the site was occupied by an events center building, parking lot, minor outbuildings and sheds, retaining walls, and an ascending slope.

Observations of the existing building were conducted as a part of our site reconnaissance. The observation of the exterior walls did not identify any significant distress beyond minor shrinkage cracking over some doorways, and horizontal movement and construction joints. On the interior, some evidence of distress along a line of VCT tile flooring was noted in the building interior running north-south about half the distance across the lower level of the floor. The distress may have been associated with a control joint in the underlying slab or differential movement of the slab on grade. No evidence of excessive differential movement caused by settlement or heave was noted in the walls, or exterior sidewalks around the building.

The conditions of the existing asphalt concrete pavement at the site varied from fair to good in the low traffic open parking areas to poor in the higher traffic driveway area that runs to the east of the existing building. Block cracking of the pavements was the principal distress type noted in the parking lot areas. Meanwhile, the east driveway was noted to have areas of alligator cracking (suggesting structural failures). This driveway is used for frequent truck deliveries for the events center and a catering business that operates out of the southeast portion of the facility.

The borings in the pavement areas encountered a wide range of thicknesses of asphalt and base materials. The existing thicknesses of the asphalt concrete (AC) encountered at the site were quite variable and ranged from about 3 to 10 inches. A majority of the AC sections were underlain by highly variable thicknesses of aggregate base materials ranging from about 1 inch to 12 inches, with most sections measured between about 2 and 6 inches thick. However, numerous borings did not encounter aggregate base material below the AC section.

The 4 foot tall CMU block retaining wall at the base of the slope along the south boundary of the parking lot and driveway did not indicate any significant distress or evidence of rotation. We understand that washouts have displaced materials over the top of the wall in the past and no significant damage to the wall was noted (or prior repair of the wall reported).

It should be noted that the minor structures present south of the driveway were in active use during the field investigation and Scottish Rite staff requested that these areas used for catering operations not be impacted by this investigation. So the minor structures and any portion of the retaining wall in the east areas were not observed to identify any distress.

5.3 **<u>Existing Slope Conditions</u>**: Various slopes are located within or adjacent to the property. The surface conditions of the existing slopes descending toward the proposed building and pavement areas were observed to assess the performance of the slopes. The existing hillside to the south extends hundreds of feet horizontally beyond the south property line and appears to be about 285 feet in total height to the top of slope. However, the height of the lower portion of the slope from the existing parking lot to the subject property line is about 120 vertical feet. Thus, the majority of the existing slope is outside of the subject property. The lower portion of this hillside (estimated to be about 75 vertical feet) on the subject property was cut (steepened) from previous grading conducted in the 1950s and includes several terraces. The overall average gradient of the lower (cut) portion of the slope is about 2 horizontal (H) to 1 vertical (V). However, the gradient of the intermediate slopes between existing benches is steeper than 2H:1V. Above the cut portion of the slope, the slope appears to be native. The overall average gradient of the upper (native) portion of the slope, based on limited topographic information, is about 1.5H to 1V.

An existing variable height slope also occurs within a portion of the eastern side of the property. However, much of this slope is offsite.

The following subsections describe the different portions of the slopes observed on and adjacent to the site.

Photographs of the overall slope and notable features are included in Appendix A of this report; and a general cross-section of the overall slope and proposed building location is included on Drawing No. 7 in Appendix A.

5.3.1 **<u>Lower Portion of South Cut Slope</u>**: Based on the topographic maps provided by San Dieguito Engineering (referenced in this report), the lower cut portion of the slope within the southern portion of the site is about 75 feet high. The cut slope includes three (3) separate 20 to 30 foot high sections that are separated by two flat terraces which are 15 to 20 feet wide. The slopes between the terraces have variable inclinations with the steepest portions about 1H to 1V in the area west of the outlet of the side canyon drainage and most of the other areas of the slope range from about 1.5H to 1V to 1.75H to 1V. The existing cut slope configuration appears to be generally similar to the steep terraced cut slopes which commonly occur within the lower portion of the slopes on the south flank of Mission Valley between Mission Center Road and Qualcomm Way.

In general, it appears that the lower portion of the slope was cut throughout the 1950's to increase the usable area of the property. However, some limited fill soils were encountered at the east end of the upper terrace (see boring log from location S-1). Considering the prior grading, these fill soils were likely placed in the later 1950's to allow access to the upper terrace from the adjacent church property which was graded after the grading of the subject site. A large surface drain pipe located at the west end of the terraces carries runoff to the toe of the slope at the west boundary of the property.

In general, the soils exposed in the central portion of the upper part of the cut slope were noted to be granular, including an abundance of sub-rounded gravel and cobble materials (see photographs 8 through 11 in Appendix A). Also, a section of fluvial deposits were exposed in a temporary cut made to access the lower terrace from the base of the slope (see photographs 14 and 15 in Appendix A). In general, these granular soils appear to be more prone to erosion than other areas of the slope. It is our understanding cobbles, gravel and sediment that accumulates below the slope from erosion, etc. have been periodically removed for many years (see photographs 2 and 7 in Appendix A).

The lower portion of the cut slope area was observed and evidence of previous rill type erosion, shallow soil slips (less than 1 to 2 feet in depth), remedial erosion control measures and slope repairs, accumulated sediment and drainage improvements added after initial construction were noted. The majority of the erosion and soil movement was noted in the central and western portion of the north facing cut slope as indicated on Drawing No. 3 in Appendix A. In this central portion of the slope, the slope is not covered with mature bushes or established native grasses. However, the eastern half of the cut slope did not exhibit significant erosion features or evidence of surficial instability. This area of the slope generally contained well established vegetation.

Further, a drainage inlet on the upper terrace, not associated with the main drainage features, suggests attempts to remedy past drainage problems. Also, poor drainage was noted within the existing terraces as exhibited by standing water from recent rainfall along many portions of the upper and lower terraces (shown in Photograph No. 9). In general, the central section of the cut slope area appears to be impacted by continued erosion of the more granular, less cohesive soils exposed on these portions of the slope. Larger cobble and coarse gravel materials were noted accumulate at the base of the slopes. This condition restricts the intended drainage to the west, and allows surface runoff to pond on the terraces.

Also, it is our understanding that the inlet of the pipeline collecting runoff from the existing natural drainage in the southeastern portion of the site (at the outlet of the canyon) has been blocked during intense rain storms over the years. Once this inlet is blocked, the flow redirects along the upper terrace and down across the middle section of the slope, resulting in erosion of the slopes and terraces below. Mr. Brian James (James Company) indicated that the middle portion of the south cut slope below the outlet of the side canyon had “washed out” several times in the past. One particular washout was severe enough that three or more feet of the lower terrace to the west of the drainage outlet (in the middle portion of the slope) had eroded into the existing parking lot area below the slope. Mr. James reported that the washout was repaired with a geogrid reinforced fill. In addition, Mr. James (James Construction) reported that repairs of a past washout included filling two erosion features that were about 6 feet wide.

5.3.2 <u>Upper North Facing Native Slope</u>: The native slope above the existing cut portion of the north facing hillside has a natural grade of about 1.5H to 1V, which flattens slightly to a 2H to 1V slope just above the cut portion of the slope. The upper native slope has an elevation at the top of about 340 feet AMSL compared to the elevation of about 130 feet to the top of the lower cut slope. This results in an overall native slope height of about 210 feet above the cut portion of the slope.

The upper about 170 feet of slope height is outside of the subject property. The upper, north facing native slope (above the existing graded cut slope) was observed to be covered with native grasses, dense bushes, and trees (see photographs 4 and 5 in Appendix A). Based on our site observations, we did not identify any significant soil slips or excessive erosion within the native slope which would require repair. The native slopes appeared to be performing well. The dense vegetation growth covering the slope seems to provide adequate resistance to erosion and shallow slope movements.

5.3.3 <u>Upper Drainage Side Slopes</u>: Landslide mapping by Tan (1995) indicates the presence of a slide area on the east facing slope of the upper drainage (side canyon). This area is identified on the map included as Drawing No. 5 and photograph 6 of this east facing slope is included in Appendix A of this report .

Based on our observations, a large amount of cobbles had accumulated near the existing drain pipe inlet and along the flow line at the outlet of the side canyon. At the time of our observations, the drainage area did not contain flowing water. In this area, a chain link (debris) fence had been placed above the existing drain pipe inlet and wing wall structure to prevent cobbles from entering the pipe and clogging the inlet. However, erosion had occurred around the fence, and numerous cobbles were noted between the fence and the pipe inlet. It is expected that the cobbles and sediments will continue to migrate toward the drainage inlet due to sedimentation, and erosion of up-slope areas from the natural drainage area in the future. In addition, the presence of a mapped landslide in the canyon above could contribute a higher potential for sediment transport within the natural drainage.

5.3.4 <u>Eastern Property Boundary Cut Slopes</u>: The subject site also includes a variable height cut slope which is located southeast of the proposed Home Depot building. The adjacent church property is located near the top of the slope. The slope grades in this area are about 1.5H to 1V and the slope varies in height from 40 to 50 feet at the south end of the slope. The northern extension of this slope is offsite to the east. At the time of our site observations, this slope was covered with mature trees with a native grass undergrowth and bushes. No evidence of sliding, soil slips, erosion or washouts was noted in this eastern slope area. Thus, this area of slope appeared to be performing well and contained mature vegetation which has provided effective resistance to surficial instability and erosion. A paved driveway with curbs associated with the adjacent church development is located above the slope. Thus, the slope does not receive any up-slope surface drainage.

5.4 **<u>Soil Profile</u>:** Subsurface exploration was conducted during this investigation in the proposed building, pavement areas and within the lower portion of the hillside area to the south. The following descriptions constitute a general summary of the soil conditions encountered in the test borings drilled for this investigation. Detailed descriptions of the soils encountered at each test boring are presented on the logs of borings in Appendix B. The stratification lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual.

5.4.1 **<u>Home Depot Building Area</u>:** The borings drilled in the area of the proposed Home Depot building were drilled through existing asphalt pavements. Based on the geologic maps, the subsurface soils encountered in the test borings drilled in the south deep cut portion of the site are designated for the Home Depot store are considered to be Mission Valley Formation materials.

The soils encountered generally consisted of sandy lean clays and clayey sands. Field classifications noted that variable amounts of fine to coarse gravel and cobbles are present within the lean clay/clayey sand stratum. These coarse grained gravel and cobble materials typically encountered about 2 to 5 foot thick layers at isolated depths and locations. The larger rock materials often resulted in drilling auger refusal at depths as shallow as about 2 feet in the south portion of the proposed Home Depot building pad. In addition to gravel and cobbles, non-plastic silty sands, and sandy silts with occasional layers of poorly graded sands were encountered in zones only a few inches thick, to layers about 5 to 10 feet thick.

Also, some fill soils were encountered in the boring drilled in the north portion of the auto dealership property (Boring A-1) to a depth of about 4 feet. Fills were not encountered in any adjacent borings, so although the extent of the fill soils is unknown.

5.4.2 **<u>Parking Structure and Parking Lot</u>:** The soils encountered within borings drilled within the northern portion of the site were somewhat similar as the soils encountered in the Home Depot building area. Based on the geologic maps, the subsurface soils encountered in the test borings are likely colluvial deposits possibly underlain by and irregular Mission Valley Formation deposits.

The upper 5 to 10 feet BSG did encounter more granular silty sands, and non-plastic sandy silts compared to the Home Depot pad. Also, deeper layers of cobbles and sands were encountered below the typical sandy lean clays and clayey sands.

Larger rock materials resulting in drilling auger refusal at depths as shallow as about 2 feet were also encountered in the eastern portion of the proposed parking structure (such as borings H-8B, J-7.3 and J-8B). Also, areas of fill soils (identified by buried pavements and construction debris) were encountered near the north boundary of the parking structure and parking lot (Camino Del Rio South frontage) at boring locations A-8, C-8, E-7.6, E-8, F-7B, I-8A K-7.6, M-8A and N-8. Also, although debris was not noted, the lower N-values from Standard Penetration Testing indicate variable thicknesses of fill soils are likely present within the northern portion of the site near the Camino Del Rio South frontage.

Since the parking structure area is located within a zone of high potential liquefaction according to City of San Diego geologic hazard maps, the supplemental field investigation included extending borings to 50 feet BSG to evaluate liquefaction and seismic settlement potential. Initially, two borings were intended to be advanced to 50 feet, however, auger refusal in a gravel/cobble stratum at 30 to 35 feet BSG prevented deeper exploration. After three attempts to penetrate this deep stratum with a CME-75 drill rig failed, a higher torque drill rig was used to extend one boring to deeper depths near the northeast corner of the proposed parking structure. This supplemental boring was advanced through gravel/cobble material to a depth of 45 feet, were auger refusal on larger material (likely boulder size) was encountered.

In these deeper borings, the soil stratum encountered below the upper fine grained sandy lean clays sandy silts, and clayey sands (encountered to about 25 to 30 feet BSG) included predominantly coarse grained granular materials including interbedded layers of gravels, cobbles, poorly graded sands and silty sands to the maximum depths explored, 45 feet BSG.

5.4.3 South (North Facing) Slope: This investigation included drilling soil borings within the lower portion of the ascending south slope. These borings, designated S-1 through S-8 (see Appendix B) were drilled on the accessible upper terrace with surface elevations ranging from 107 to 115 feet. The soils encountered were generally sandy lean clays with some minor fractions of gravel and some cobbles to depths of about 35 to 40 feet BSG. The upper lean clays were interbedded with low to non-plastic sandy silt layers about 5 to 10 feet thick. Also, an approximately five (5) foot thick layer of clayey gravel was encountered at a depth of 10 feet BSG in the middle section of the slope, and some of the borings encountered auger refusal on cobbles at depths ranging from about 3 to 9½ feet BSG.

Below the upper sandy lean clays encountered at the site, a stratum of poorly graded sand was encountered in the deepest boring drilled to a depth of 47½ feet BSG (elevation 60 feet). This stratum was encountered in the bottom of both borings at the east and south slope areas explored beginning at an elevation of about 80 feet. These granular soils were also exposed in a cut slope in the west portion of the slope, just above the retaining wall and parking lot. The soils exposed in the cut were described as poorly graded sands with varied amounts of gravel and cobbles.

Distinct bedding was not noted in the small diameter borings drilled, nor the surface cuts logged for this investigation.

It should be noted that fill soils were encountered at the eastern end of the slope terrace. The fill soils were also sandy lean clays but were mixed with wood debris and were found to extend to depths between about 10 and 15 feet BSG. The other soils encountered on the upper slope terrace were native soils. It is possible that this fill was placed at the east end to allow access to the terrace from the adjacent church parking lot.

Generalized logs of the soils exposed on the lower cut slope are illustrated on Drawing Nos. 8 and 9 in Appendix A of this report.

5.5 **<u>Laboratory Testing</u>:** Laboratory testing of soil samples was conducted to determine selected properties of the soils. The results of laboratory tests are included on the boring logs in Appendix B and on the laboratory test reports included in Appendix C.

The consolidation characteristics of the sandy lean clay and clayey sand soils were determined by seven (7) consolidation tests. The tests measured consolidation of from 5.9 to 11 percent under a load of 16 kips per square foot. The samples tested indicated a slight to moderate collapse (ranging from 0.3 to 2.3 percent) when inundated with water under a load of 2 kips per square foot. However, one sample from the north parking structure location indicated more consolidation (13.2 percent at 16 kips per square foot) and a swell of 1.7 percent when inundated with water under a load of 2 kips per square foot. This indicates different consolidation characteristics of the soils in the north portion of the site in the more recent colluvial material compared to the conditions encountered in the Home Depot building pad area where the soils had likely been subject to overburden from the former slope.

Shear strength tests were conducted on five (5) samples of the various soils encountered using direct shear methods. Sandy lean clay samples indicated angles of internal friction ranging from 18 to 36 degrees with cohesion values ranging from 1,080 to 50 pounds per square foot, respectively. Also, to evaluate the shear strength of engineered fill soils, four (4) samples of clayey sands and sandy lean clays were remolded at 90 percent of the maximum dry density (ASTM D1557) for shear strength testing. The remolded samples indicated angles of internal friction ranging from 19 to 35 degrees with cohesion values ranging from 410 to 190 pounds per square foot, respectively.

The expansion potential of the clay soils was evaluated by expansion index tests. The clay soils within the building pad were found to exhibit a medium expansion potential as indicated by two expansion index results of 77 and 81. Tests conducted on sandy lean clay and clayey sand samples indicated maximum dry densities of 126.5, 126.8, 126.8 and 129.3 pounds per cubic foot with optimum moisture contents of 8.3, 9.9, 10.1 and 10.4 percent.

R-value tests conducted on three sandy lean clay samples indicated R-values of 19, 20 and 22. One R-value test conducted on a clayey sand sample indicated an R-value of 22. One R-value test conducted on a silty sand sample from the south portion of the site indicated an R-value of 63.

5.6 **<u>Groundwater Conditions</u>:** Groundwater was encountered in six (6) of the soil borings. Groundwater was encountered at a depth of about 29½ feet BSG (elevation of about 24½ feet) in boring A-2 drilled in the southwest corner of the site. Groundwater was encountered at a depth of about 30 feet BSG (elevation of about 21 feet) in boring F-6, which was drilled in the middle portion of the north wall of the proposed Home Depot store. In the parking structure, groundwater depths ranged from 25 feet at location G-8 (elevation about 26 feet), 30 feet BSG at location E-7.6 (elevation about 18 feet), and 30 feet at location M-8A drilled in September of 2019 (elevation of 21 feet), and 26 feet at location M-8B drilled in December of 2019 (elevation of 25 feet). Note that 24 hour measurements, as required by Home Depot Guidelines, could not be taken to comply with Scottish Rite (site owners) requirement to backfill borings each day since the site was open to the public.

Based on our review of California Department of Water Resources Control Board Geotracker data, Two sites were identified within ½ mile of the Home Depot project that included groundwater data. Research identified five (5) monitoring wells installed at a fuel station about one-half (½) mile northeast of the site in 2004 indicated groundwater depths ranging from about 27 to 28 feet BSG. These wells are on a property near the San Diego River, which has a similar elevation to the site of about 58 feet AMSL. A second site about one-half (½) mile west of the site in 2003 and 2004 indicated groundwater depths ranging from about 20 to 21 feet BSG. Although the elelvation fo wells was not indicated, this site was an auto dealership south of Camino Del Rio too, so site elelvaitons are likley similar to the project site.

To research historical groundwater levels, groundwater data on the Department of Water Resources Water Well Data Library was reviewed. The nearest well to the site in this database (16S03W13Q004S) has groundwater elevation measurements from 1978 to 1990 and is located about ½ mile north of the site (north of the San Diego River). This well has a surface elevation of about 45 feet which is about 5 to 10 feet lower than the project site. The elevations of groundwater in this well ranged from elevation 33 feet in 1980, elevation 25 feet 1989.

Considering the locations and elevations of researched well data, and the range of groundwater depths encountered during the field investigation, a historic high groundwater of about 20 feet was used for analysis.

It should be recognized that groundwater elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation/measurements may vary from those encountered both during the construction phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

5.7 **<u>Percolation Test Results</u>:** The infiltration rates estimated from the percolation test data are summarized in Table No. 1 below. The field measurements for each percolation test are included in Appendix D.

Table No. 1
Results of Percolation Testing

Location and Depth	Field (Unfactored) Infiltration Rate (Inches per Hour)[1]	Subgrade Soil Type
P-1 at J-8; 6 feet BSG	0.1	Silty Sand
P-2 at I-8; 10 feet BSG	0.5	Clayey Sand
P-3 at L-9; 15 feet BSG	0.4	Silty Sand

Notes:
BSG - Below site grade
1. Includes no factor of safety

The unfactored estimated infiltration rates do not take into account the long term effects of subgrade saturation, silt accumulation, groundwater influence, nor densification as a result of the construction process. Percolation/infiltration rate of the soils will decrease when the soils are saturated and the percolation/infiltration rate is further reduced the longer the soils are saturated. Published studies indicate short term field infiltration rates can significantly overestimate the saturated permeability. In addition, soil bed consolidation, sediment, suspended soils, etc. in the discharge water can result in clogging of the pore spaces in the soil. This clogging effect can also reduce the long term infiltration rate. Numerous other factors, such as variations in soil type and soil density across the entire area of the system, can influence the percolation/infiltration rate, both short and long term.

The percolation test data are included in Appendix D of this report.

6.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for project design and preparation of geotechnical related construction specifications are summarized in the following subsections. The evaluations were based upon the subsurface conditions determined from the investigation, our review of the project site plans, research of available maps and reports, and our understanding of the proposed construction. The conclusions obtained from the results of our evaluations are described in the Conclusions section of this report (Section 7.0).

6.1 Existing Surface Conditions in New Buildings: Due to the existing development, demolition and removal of the existing site improvements will be required as part of site preparation. The existing structure and facility consist of foundations, retaining walls, sub-level slabs-on-grade, utilities and other improvements. All of these features will need to be removed and the resulting excavations properly prepared and backfilled. All existing surface and subsurface structures, such as shallow foundations, retaining walls, floor slabs, utilities, etc., should be removed entirely and not buried in place. Areas with existing improvements should be over-excavated to at least 12 inches below the bottom of the existing improvements to be removed, or to the depth to remove disturbed soils from the demolition activity, whichever is greater. The existing 30 inch storm drain that will be abandoned and relocated should be excavated, completely removed, and backfilled as an engineered fill (not abandoned in-place) as a part of site preparation. The location of this existing storm drain line is shown on Drawing No. 2 in Appendix A.

After excavation and removal, the exposed soils should then be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted as engineered fill. All excavations conducted as part of the demolition should be backfilled with engineered fill. All existing underground utilities and the associated fill soils should be removed and replaced with engineered fill.

6.1.1 Oversize Rock / Soil Processing: Fine to coarse gravel and cobbles were commonly encountered at shallow depths throughout the site. Drilling auger refusal was encountered in seventy-two (72) of the one hundred-twelve (112) borings at depths as shallow as about 2 feet BSG (thirty-nine borings encountered refusal within the upper 10 feet or less). The soils with coarse materials were usually characterized as hard or very dense on the boring logs (N-values greater than 50 blows per foot). These oversized materials are typical of the "cobble conglomerate" deposits described by the regional geologic reports. It was noted that more cobbles were generally encountered in the near surface soils (upper 3 to 5 feet) in the eastern portions of the Home Depot and parking structure buildings. Less cobbles were encountered in the near surface soils in the western portion of the building pads (although cobbles were encountered).

These hard and dense conditions and coarse gravel and cobble materials will require more effort to excavate and process. Further, oversized materials placed and/or compacted directly below foundations and floor slabs can result in hard points resulting in differential movement and cracking. To provide uniform support, the Site Preparation and Earthwork recommendations of this report indicate that if on-site soils are to be used as engineered fill, cobble material should removed by screening prior to placement and compaction as engineered fill. In order for the onsite soils to be used as fill on the site, removal of over-sized rock should be anticipated. Screening should be anticipated due to the presence of cobbles and coarse gravel. In order to reduce export of materials screened from the soils, it may be possible to crush the oversized material on-site to sizes suitable for use in engineered fill.

In order to obtain additional information for use in bidding the screening type requirements for over-size materials in the onsite soils, a supplemental investigation is recommended. The investigation should include subsurface exploration using test pits in order to document the fraction of rock and range of sizes that anticipated to be encountered during grading.

6.1.2 Undocumented Fill: Fill soils were identified in the area of the proposed parking garage (north portion of the site) as indicated by buried pavements and construction debris encountered to depths of about 5 to 6 feet along the east section of the roadway frontage and to a depth of about 10 feet in the northwest corner of the site (boring locations A-8, L-8 and N-8). The other borings along this north lowest portion of the site did not identify debris or buried features (borings C-8, E-7.6, E-8, F-7B, I-8A K-7.6, and M-8A) but did indicate lower N-values in the upper soils, which suggest the potential for undocumented fill placed during past mass grading of the north portion of the site. The undocumented site fill soils along the north side of the site should be identified during site preparation for the parking structure and pavement areas, excavated to expose undisturbed soils, and replaced as engineered fill to final grades.

Also, fill soils were encountered in the boring drilled in the north portion of the auto dealership property (Boring A-1) to a depth of about 4 feet. Since these soils are located in an future pavement area for the rear driveway, the fill should be excavated and compacted as engineered fill as a part of site preparation.

Some fill soils were also encountered at the east end of the upper slope terrace to a depth of about 15 feet BSG. These soils were likely placed so the upper slope terrace and drainage area inlet structure could be accessed from the adjacent property parking lot. Since these fills were found to have relatively high shear strength, slope instability was not noted in this area, and the fills do not support any permanent pavement or structural improvements, these soils can remain in-place.

6.1.3 Wet, Unstable Soils: During the February and March 2019 field investigations, moisture contents as high as 30 percent in some of the sandy lean clays were measured in the soil samples collected within the upper approximately 5 feet BSG. About 10 percent of the samples of clays within the upper approximately 5 feet were found to be 10 to 20 percent above the optimum moisture content. Accordingly, it is anticipated that the some of the clay soils excavated during site grading will need to be aerated, i.e. dried, to meet the moisture conditioning requirements of this report (between at least two (2) percent and five (5) percent above optimum moisture content) and to allow compaction of the wet soils as engineered fill. Due to the high soil moisture contents, these wet soils could be exported from the site, or spread and repeatedly mixed/disced, or chemically treated to dry the soils in order to achieve proper compaction.

In addition, where wet, unstable soil conditions are encountered, methods such as aeration, mixing wet soils with drier soils, chemical treatment, or the use of aggregate base or crushed rock and a geotextile stabilization fabric may be required to achieve a stable condition at the bottom of the excavations and in areas that require subgrade preparation. Thus, the contractor will be required to treat wet, unstable soils to obtain the compaction requirements of this report and establish stable subgrade soil conditions prior to placement of fill.

6.2 Static Settlement and Bearing Capacity of Shallow Foundations: The potential for excessive total and differential static settlements of foundations and slabs-on-grade is a geotechnical concern evaluated for this building site. The increases in effective stress to underlying soils which can occur from new foundations and structures and placement of fill, etc. can cause vertical deformation of the soils, which can result in damage to the overlying structure and improvements. The differential component of the settlement is often the most damaging. In addition, the allowable bearing pressures of the soils supporting the foundations were evaluated for shear and punching type failure of the soils resulting from the imposed foundation loads.

Since the proposed development includes a new Home Depot Store and a parking structure, and considering the different soil conditions and structure types expected, the evaluation of foundation design parameters and site preparation for these two structures are presented in the following separate subsections.

6.2.1 Home Depot Store: Considering the anticipated wall and column loads for the Home Depot building, the consolidation and hydro-collapse characteristics of the soils encountered below the Home Depot Store, conventional shallow building foundations and floor slabs would meet Home Depots criteria for total static settlements. However, conventional footings supported on the variable very dense gravel/cobble materials and the stiff to hard sandy lean clays would be subject to excessive static differential settlements (more than ½ inch in 50 feet).

In order to provide more uniform support of foundations to meet Home Depot differential settlement requirements, over-excavation would need to occur to a depth of about 3 feet below the existing site grade to remove the upper disturbed soils and any undocumented fill soils; and to provide at least 2 feet of engineered fill below all foundations, whichever provides the deepest over-excavation. The allowable soil bearing pressure for spread foundations supported on engineered fill is 2,500 pounds per square foot for dead-plus-live loads. Based on this bearing capacity, the following settlements are anticipated for the foundations and slabs on grade: 1) a total static settlement of ¾ inch and 2) a differential static settlement of ½ inch in 50 linear feet.

6.2.2 Parking Structure: The soil conditions in the north portion of the site proposed for the parking structure are highly variable and appear to be less consolidated (weaker) compared to the area of proposed for the Home Depot Store. Based on the higher compressible soil conditions expected, it is estimated 2 to 3 inches of static settlement could occur under the typical interior column (360 kips dead load and 190 kips live load for 550 kips total) using a recommended allowable bearing capacity of 2,500 psf.

The most direct method to reduce the settlements would be to over-excavate the variable compressible soils and replace these materials as densified engineered fill. Significant over-excavation would be required to place engineered fill below footings to limit static settlements. As an alternative, considering the depth of removal, ground modification such as Geopiers could be a effective method to densify the upper variable soils and reduce foundation settlements to tolerable levels.

Also, the designers for the parking garage have indicated that the garage structure spans could be reduced (add more columns) and the structure could be supported on a continuous mat type foundation. Mat foundations would reduce the applied soil pressures by increasing the bearing area, and typically can resist more differential settlements than similar structures supported on isolated shallow spread foundations.

The costs of the structures and different site preparation recommendations should be evaluated to identify the type of foundation to be used based on the preliminary recommendations provided in this initial report.

6.3 **<u>Expansive Soils</u>:** One of the potential geotechnical hazards evaluated at this site is the expansion potential of the near surface soils. Over time, expansive soils will experience cyclic drying and wetting as the dry and wet seasons pass. Expansive soils experience volumetric changes (shrink/swell) as the moisture content of the clayey soils fluctuate. These shrink/swell cycles can impact foundations and lightly loaded slabs-on-grade when not designed for the anticipated expansive soil pressures. Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). Expansion potential may not manifest itself until months or years after construction. The potential for damage to slabs-on-grade supported on expansive soils can be reduced by placing non-expansive fill underlying the slabs-on-grade and extending the perimeter foundations to depths necessary to establish a moisture cutoff.

Expansion index (swell) testing was performed on samples of the near surface lean clay soils collected from the proposed building pad subgrade at the site. The tests indicated a medium expansion potential, with expansion index values of 77 and 81. Medium expansive material would be expected to cause heave/shrinkage exceeding ½ inch in 50 feet. Thus, it is recommended to support Home Depot floor slab on at least 6 inches of aggregate base underlain by 18 inches of imported non-expansive fill (EI less than 20) for a total depth of 24 inches of non-expansive materials. Foundations can also be damaged by expansive soils, so it is recommended to extend perimeter continuous foundations to at least 30 inches below the lowest adjacent grade, below where seasonal moisture fluctuations typically occur.

6.4 **<u>Seismic Ground Rupture and Design Parameters</u>:** The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The closest active fault is the Rose Canyon Fault Zone, which is located approximately 4 miles west of the site. The City of San Diego Seismic Safety Study indicates a concealed segment of the Texas Street fault is located adjacent to the northwest corner of the site. However, the fault category is described as "potentially active, inactive, presumed inactive, or activity unknown." Accordingly, the potential for ground rupture at the site is considered low.

Seismic coefficients and spectral response acceleration values were developed in accordance with the 2019 California Building Code (CBC). The CBC methodology for determining design ground motion values is based on U.S Geological Survey seismic hazard maps, which incorporate both probabilistic and deterministic seismic ground motion.

A table providing the recommended seismic coefficient and earthquake spectral response acceleration values for the project site is included in the "Seismic Factors" recommendations section of this report. The standard penetration test results indicate a Site Class D based upon N-values between 15 and 50 blows per foot, for the upper 100 feet BSG. These field N-value results indicate the subgrade soils are considered a stiff soil site based on the method included in ASCE 7-16, Section 20.4.2.

A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.617g was determined for the site using the Ground Motion Parameter Calculator from the Structural Engineer's Associates of California (https://seismicmaps.org). A Maximum Considered Earthquake magnitude of 6.89 was determined for the site based on deaggregation analysis (United States Geological Survey deaggregation website (https://earthquake.usgs.gov/hazards/interactive/).

6.5 Liquefaction and Seismic Settlement: Based on Grid Tile 21 of the City of San Diego Seismic Safety Study Geologic Hazards and Faults, dated April 3, 2008, prepared by the City of San Diego Development Services Department, the northern portion of the subject site within the parking structure area is located in a liquefaction hazard zone. The Home Depot building area is not located in the liquefaction hazard area.

Liquefaction and seismic settlements are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing can result. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the common characteristics for liquefaction.

Liquefaction and seismic settlement analyses were conducted based on soil properties revealed by the test borings and the results of laboratory testing. The analyses were conducted for soils encountered in the deeper borings for the Home Depot store (A-2 and F-6). Also, the analysis was conducted for the deeper borings drilled in the parking structure location (G-8 and M-8). The analysis was conducted using the software program LiquefyPro developed by CivilTech. A horizontal ground acceleration of 0.617g, a maximum considered earthquake of 6.89 and a high groundwater depth of 20 feet were used in the analysis. Soil parameters, such as wet unit weight, N-value, fines content, and depth of N-value tests, were input for the soil layers encountered throughout the depths explored (see test boring logs, Appendix B).

The N-values generated were used to determine the cyclic stress ratio needed to initiate liquefaction. For the borings drilled using the CME-75 drill rig, a hammer energy ratio correction of 1.5 was applied to the field N-value results based on the results of equipment specific hammer energy calibrations. The hammer energy ratio correction was based on overall transfer efficiency of 89 percent for the hammer as indicated in the report titled: "Energy Measurement for Dynamic Penetrometers," prepared by GRL Engineers, Inc., dated July 10, 2019 (included in Appendix E of this report). For the boring drilled using the Yeti-10 drill rig, a hammer energy ratio correction of 1.6 was applied to the field N-value results based on the energy transfer data provided by the Marl Industries eSPT system output with an average transfer efficiency of 96 percent for the hammer.

One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils, however, seismic settlements are typically largest where liquefaction occurs (saturated soils).

For the Home Depot store, the results of the liquefaction analyses indicate that a thin layer of medium dense silty sands encountered at a depth of about 40 feet at test boring A-2 would be susceptible to liquefaction (A-2 is located along the east wall of the building). The total estimated seismic settlement was 0.9 inch.

For the parking garage location, the results of the liquefaction analyses indicate that some loose to medium dense poorly graded gravels and sands and silty sands encountered between the depths of 28 to 38 feet near the northeast corner of the area proposed for the parking structure (boring M-8C) are also susceptible to liquefaction. The total seismic settlement was estimated to be about 1.1 inches.

Given the depth and relatively thin layer thickness where liquefaction is expected to occur, it is not expected that the loss of strength associated would impact the ability of the soils to support the proposed foundations or surface improvements. Also, considering the depth and isolated nature of these zones susceptible to seismic settlements, the Home Depot store should be designed for a estimated surface seismic settlements of about ½ inch in 50 feet. Given that more seismic settlement was indicated in the parking garage area, the parking structure should be designed for a differential seismic settlement of ¾ inch in 50 feet.

The liquefaction and seismic settlement analysis output are included in Appendix E of this report.

6.6 **<u>Slope Stability</u>:** The City of San Diego Seismic Safety Study classifies the ascending slope area in the southern portion of the site as: "sloping terrain, unfavorable geologic structure, low to moderate risk" (category 53). Given the low to moderate risk, the focus of the evaluation of slope stability was to conduct a qualitative assessment of slope stability considering the geologic nature of the material, the potential unfavorable geologic structures and past performance of the slopes on and near the site. For the purpose of evaluation, the slope areas are described separately including: 1) the undisturbed native slope that extends above the cut slope beyond the property line; 2) the lower south facing cut slope areas; and 3) the shorter transition slope located east of the proposed building site and adjacent to the church driveway.

6.6.1 **<u>Upper Native Slope</u>**: The existing undisturbed native slope has a height of about 210 feet and a natural grade of about 1.5H to 1V, which flattens slightly to a 2H to 1V slope within the property just above the lower cut portion of the slope. The majority of this slope is located offsite.

Further, since subsurface exploration of the upper native slope was not feasible due to the terrain and offsite conditions, site observations and research were used to develop opinions about the stability of the upper native slope. This upper slope is covered with thick scrub brush type vegetation. No visual evidence of slope instability or disturbed vegetation was noted within this slope during our site reconnaissance and no indications of significant slope movement was noted in the historic photographs reviewed. Thus, the slope appears to be performing well with respect to stability. A similar slope condition occurs with respect to topographic and geologic conditions along the south flank of Mission Valley to the west and east of the site and no mapped landslide features are known on these north facing slopes. The Kennedy and Tan (2005) geologic map (see drawing No. 4 in Appendix A) indicates bedding of the Mission Valley Formation at several nearby locations was found to dip slightly to the south (into the slope) at about 3 to 5 degrees. This indicates that the geologic structure is generally neutral and is not considered unfavorable with regard to slope stability.

As indicated in Section 5.1.4 of this report, previous geotechnical investigations by others noted that the existing hillsides adjacent to other nearby sites along the south flank of Mission Valley (including the property immediately to the east of the subject site) were not considered to have unfavorable geology and the geotechnical reports concluded the slopes were stable.

Considering that no unfavorable geologic structures were identified, the existing native slope has been performing well for quite some time, and the geologic nature of the slope materials, the slope is considered stable and the potential for instability of the upper native slope is low.

However, it is recommended to have a geologist or geotechnical engineer observe the upper native slopes periodically to note any changing conditions that could impact the site.

6.6.2 Lower North Facing Cut Slope: The existing cut portion of the south slope is about 75 feet high, and includes three (3) separate 20 to 30 foot high sections that are separated by two 15 to 20 foot wide terraces. The steepest portion of the slope between terraces has a gradient of about 1H to 1V in the area west of the outlet of the side canyon drainage. However, most of the other areas between terraces have intermediate slopes ranging from about 1.5H to 1V to 1.75H to 1V. The overall grade of the slope from the top to the toe of the cut portion of the slope is about 2H to 1V. Site reconnaissance above and below the slope by the undersigned Certified Engineering Geologist did not identify evidence of scarps, lateral displacement, bulging at the base (retaining wall displacement), or steep bedding dipping towards the slope inclination (unfavorable geologic structure) suggesting concerns with deep seated instability. Further, deeper soils encountered in the borings drilled on the slope exhibited good shear strength characteristics and the existing hillsides within and adjacent to the site have performed well with respect to deep seated stability for quite some time. Thus, it was concluded that deep seated slope instability is not a significant concern since the project does not propose to significantly alter the existing cut slope.

However, areas of the middle and lower sections of the slope have exhibited areas surface erosion and shallow slips (see Drawing No. 3 in Appendix A). The center section of the slope has deposits with abundant cobble fraction, and exposed predominantly granular, low cohesion, soils have exhibited high erosion and shallow soil slips 1 to 2 feet deep. Further, significant slope erosion has reportedly occurred from past washouts caused by blockage of the existing drainage inlet structure above the upper terrace, It was reported that the middle and lower slopes have been repaired over time after washouts and when significant sediments accumulate at the base of the slopes on the terraces (an aerial photograph from 2005 shows this area of the slope without vegetation). Also, inspection of these slopes for this investigation have identified drainage issues. Drainage improvements will reduce, but not eliminate the surficial and erosion issues that have occurred. Thus, some surficial slope movements are anticipated to continue. Considering that the building improvements are planned to be setback at least 35 feet from the slope, significant potential impacts to the structure due to shallow slope instability are not anticipated. The current approach by Scottish Rite of maintenance and spot repairs where erosion and slippage has occurred on the cut slopes has been sufficient to maintain function. A similar level of maintenance and repair should be anticipated. However, in addition, it is recommended that a program of regular inspection of the slopes be implemented to identify conditions that could further degrade shallow slope stability, and to identify areas requiring maintenance and repair/restoration. The Slope Improvement and Maintenance Section (Section 6.6.2.1) of this report details recommendations to address these issues during and after construction.

It should be noted that only a portion of the cut slope has experienced erosion and shallow slips. The west portion of the slope (see area west of section line A-A' on Drawing No. 6 in Appendix A) is covered with native grasses and larger established bushes and trees. No evidence of surficial instability or erosion has been noted in the western portion of the slope. Since this west section of the slope has been performing well for over 65 years, it is recommended to not disturb this area of the slope.

6.6.2.1 Slope Improvement and Maintenance: Since the existing north facing cut slope has experienced drainage and erosion issues, slope improvements to improve drainage and reduce the potential for excessive erosion of the slope are recommended. In addition, to reduce potential impacts from cobble (rock) fall, debris fences should be incorporated at the base of slope on each terrace (per City of San Diego standards). Also, at the base of the slope, a retaining wall with a minimum of three (3) feet of freeboard should be implemented to reduce potential migration of cobbles and sediment onto the pavement areas below the slope. Sediment will need to be regularly cleared from the slope to maintain drainage.

To improve drainage, concrete or asphalt line drainage V-ditches should be provided to intercept surface runoff and drain the flow away from sloped surfaces such that runoff is not allowed to accumulate on the terraces and flow over the tops of lower slopes or retaining walls.

In addition, it is recommended, to implement effective erosion control such as by establishing deep rooted vegetation on the portions of the slopes not covered with established deep rooted bushes and trees to improve resistance to surficial stability and erosion. A regular inspection and maintenance program should be established to monitor drainage and maintain deep rooted vegetative cover.

Regular inspections of slopes, debris fences, inlets, and drainage ditches should be conducted to determine when slope maintenance and sediment removal is required. Thus, it will be critical to ensure access to the slope for sediment removal.

As indicated in this report, the existing inlet structure which currently collects runoff from the side canyon area within the southern portion of the site has become blocked in the past, causing runoff to flow around or over the inlet structure. The runoff appears to have drained onto the adjacent terraces and flowed over the north facing cut slopes in the past, contributing to erosion and surficial soil slips within the lower portion of the cut slope. Therefore, it has been concluded that the current drainage inlet structure and/or the maintenance (i.e., debris removal) are not adequate for the runoff conditions experienced. Thus, to reduce the impacts associated with the blocking of the current inlet structure at the outlet of the side canyon, appropriate debris catchments and inlet structure design and maintenance should be incorporated into the drainage improvements as a part of construction. The drainage structure and catchments should include redundant systems to reduce the potential for clogging. In addition, a detailed maintenance plan should be established to regularly inspect the performance of the inlet structure and remove sediment and debris to maintain functionality and reduce potential impacts to the slope. It should be noted that a landslide has been mapped within the side canyon in the southeast portion of the site and therefore significant debris and sediment would be expected to be transported along the drainage if earth movements from the side canyon occur. Thus, redundant measures to intercept cobbles and sediments that will continue to migrate down the drainage will be critical in the design of the project.

6.6.3 East Transition Cut Slope: The east boundary of the site includes a variable height, west facing cut slope that transitions the site grade to the adjacent church property to the east. The inclination of this slope is mostly about 1.5H to 1V. However, the slope steepens to the north (off site) to about 1.3H to 1V. This slope was observed to be covered with mature trees with established native grass undergrowth and bushes. No evidence of slope instability was noted in this eastern slope area and this slope has not exhibited shallow slips or erosion, such as has occurred in the middle of the north facing cut slope. Historic photographs also did not suggest the occurrence of any significant slope movement for the west facing cut slope. Thus, the slope appears to be performing well for over 65 years.

It should be noted that presently structures are located at the toe of the slope (almost zero setback). The proposed project will include a paved driveway at least 35 feet in width at the toe of the slope. Thus, the setback to the proposed structure will be significantly increased from the existing conditions.

Considering that no unfavorable geologic structures were identified and the existing west facing cut slope has been performing well for quite some time, and relatively dense vegetation is established on the slope, it is recommended to not alter or disturb this east transition slope as a part of the proposed construction operations. Modifying the slope by grading would require a significant length of time to re-establish a similar vegetative cover. Until the vegetation would establish, the slope would be prone to potentially significant erosion issues.

6.7 Asphaltic Concrete (AC) Pavements: Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report. The thicknesses of the asphalt concrete and the underlying aggregate base materials are based upon the amount and type of traffic loads being considered and the Resistance or R-value of the subgrade soils which will support the pavements. The measure of the amount and type of traffic loads are based upon an index of equivalent single axle loads (ESAL) from loading of heavy trucks, i.e., a traffic index (T.I). As a part of the evaluation of the pavement design for this project, samples of the onsite soils anticipated to be representative of the soils which will support pavements were obtained and R-value testing was performed in accordance with ASTM D2844. The R-value test results are summarized in Appendix C of this report.

The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highways Design Manual. The traffic loading data were obtained from the Design Criteria Manual provided by Home Depot U.S.A., Inc. For the proposed Home Depot store, the "standard duty" pavement should be designed for a life of 10 years and an EAL (18 kips) of 50,000 axles. An EAL of 50,000 equates to a traffic index of 6.5. The "heavy duty" pavement was designed for a life of 10 years and an EAL (18 kips) of 220,000 axles. This equates to a traffic index of 7.5. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

The results of the R-value tests indicate the near surface soils exhibit poor to good pavement support characteristics as indicated by R-value results ranging from 19 to 22 for most of the clay soil samples tested. Based on the R-values determined for this investigation, an R-value of 15 was used for design.

6.8 Portland Cement Concrete (PCC) Pavements: Recommendations for Portland cement concrete pavement structural sections are presented in the "Recommendations" section of this report. The PCC pavement sections are based upon the amount and type of traffic loads being considered and the Resistance or R-value of the subgrade soils which will support the pavement. The measure of the amount and type of traffic loads are based upon an index of equivalent axle loads (EAL) from the loading of heavy trucks, i.e, a traffic index (T.I).

As a part of the evaluation of the PCC pavement design for this project, samples of the onsite soils anticipated to be representative of the soils which will support PCC pavements were obtained and R-value testing performed in accordance with ASTM D2844. The R-value test results are summarized in Appendix C of this report.

The EALs for each of the PCC pavement sections were converted to the number of 5-axle trucks per day, one direction, anticipated for the proposed store. The EAL for the "standard duty" pavement section of 50,000 was converted to 14 axles or 6 five-axle trucks per day. The EAL for the "heavy duty" pavement section is 220,000 or 26 five-axle trucks per day. The recommended structural sections were based primarily on the Portland Cement Association "Thickness Design of Highway and Street Pavements."

The PCC pavement sections were designed for a life of 10 years, a load safety factor of 1.1, a single axle weight of 12,000 pounds, and a tandem axle weight of 36,000 pounds. A modulus of subgrade reaction, K-value, for the pavement section, considering a minimum 6-inch layer of aggregate base material (minimum R-value of 78) was used for pavement design.

6.9 **Stormwater Infiltration**: Percolation tests were conducted along the east frontage near Camino Del Rio South as part of this investigation based on the slightly more granular nature of the materials identified in the initial test borings in this area. Percolation tests were conducted at depths of 6, 10, and 15 feet BSG as requested by the project Civil Engineer based on the types of infiltration systems being considered for the project. The soils encountered in the test borings drilled for the percolation tests (P-1, P-2, and P-3) comprised silty sands and clayey sands. The infiltration rates ranged from 0.1 to 0.5 inches per hour. These rates indicate the soils have a limited infiltration capacity and, at these rates, the soils would not meet City of San Diego Storm Water Standards for a full infiltration type system (which requires a minimum factored rate of 0.5 inches per hour).

Considering the results of the infiltration rate tests, and the presence of clays in the area, an average un-factored infiltration rate of 0.1 inches per hour should be used for infiltration system design. At this rate, according to the City of San Diego Storm Water Standards, a partial infiltration type system may be feasible. Minimum factors of safety as required by the City of San Diego Storm Water Standards should be applied.

In order to reduce potential impacts to the proposed structure from expansion of clays or settlements of sands, storm water systems which allow infiltration of water into the soils should be setback at least 30 feet from the structure and building foundations. Storm water systems which allow infiltration that meet these criteria should not adversely impact the structures.

6.10 **Soil Corrosion:** The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The results of soil sample analyses on native clay and silty sand samples indicate minimum resistivity values of 1,801, 2,201, and 4,602 ohms-centimeter (full results included in Appendix C of this report). The National Association of Corrosion Engineers (NACE) provides corrosion severity ratings listed in the Table No. 2 below.

Table No. 2
Soil Resistivity and Corrosion Potential Ratings

Soil Resistivity (ohm cm)	Corrosion Potential Rating
>20,000	Essentially non-corrosive
10,000 - 20,000	Mildly corrosive
5,000 - 10,000	Moderately corrosive
3,000 - 5,000	Corrosive
1,000 - 3,000	Highly corrosive
<1,000	Extremely corrosive

Therefore, the near-surface soils exhibit a “highly corrosive” to “corrosive” potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the “highly corrosive” corrosion potential. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

6.11 **Sulfate Attack of Concrete:** Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil/groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with concrete is to perform testing to determine the sulfates present in the soils. The results of the sulfate analysis of three near surface samples indicated 0.0021, 0.0033, and 0.0042 percent by weight. These test results are then compared with the provisions of ACI 318, section 4.3 to provide guidelines for concrete exposed to sulfate-containing solutions. Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios. The test results are included in Appendix C of this report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

The soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters.

7.0 CONCLUSIONS

Based on the data collected during the field and laboratory investigations, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, we present the following general conclusions.

7.1 The site is considered geotechnically and geologically suitable for the proposed construction with regard to support of the proposed improvements, provided the recommendations contained in this report, and future design level geotechnical investigation reports, are followed. It should be noted that the recommended design consultation and observations during construction by Moore Twining are integral to this conclusion.

7.2 The near surface soils encountered below the existing pavements within the proposed Home Depot store area were generally found to be stiff lean clays or loose clayey sands to depths of about 1 to 3 feet. These upper loose, or stiff disturbed soils will not provide uniform support of proposed settlement sensitive floor slabs or foundations. Thus, as a part of the site preparation, these soils should be excavated to expose undisturbed native soils and to achieve the minimum recommended depth of engineered fill below the foundations before filling the building pad to grade. Below these upper soils, similarly classified sandy clay and clayey sand soils were encountered in hard and dense conditions in the approximately upper 3 to 10 foot BSG.

7.3 The soil conditions below the existing pavements in the northern portion of the site where the parking structure is planned are highly variable and appear to be less consolidated (weaker) compared to the soils in the area of proposed for the Home Depot Store. Soils consisting of loose to medium dense silty sands and stiff clays with more dense and hard soil profiles in adjacent borings suggest more variable conditions. Based on the higher compressible soil conditions expected, the parking garage will require deeper over-excavation depending on column loads of the final design, or a ground modification program such as Geopiers could be a effective method to densify the upper variable soils, and reduce foundation settlements to tolerable levels. As another alternative, the structure could be supported on a continuous mat type foundation to reduce applied soil bearing pressures and to resist higher levels differential settlements expected by the variable conditions. Since the parking structure details were not known at the time this report was prepared, final recommendations for the parking structure are deferred to the design level geotechnical report.

7.4 Variable amounts of fine to coarse gravel and cobbles are present in about 2 to 5 foot thick layers within the lean clay/clayey sand strata. These soils with coarse materials are usually characterized by hard or very dense conditions on the boring logs(N-values greater than 50 blows per foot). In addition, shallow drilling auger refusal (10 feet or less) was encountered in thirty-nine (39) of the one-hundred-twelve (112) borings (refer to Boring Logs in Appendix B). These hard and dense conditions and coarse gravel and cobble materials will require more effort to excavate and process than typical soils without coarse materials. Due to the oversize rock, the cobble material will need to be removed such as by screening prior to placement and compaction as engineered fill.

7.5 The on-site clay soils have a medium expansion potential as indicated by expansion index values of 77 and 81. Medium expansive material would cause heave/shrinkage exceeding ½ inch in 50 feet. Thus, it is recommended to support floor slabs on non-expansive aggregate base and imported non-expansive granular fill; and extend perimeter foundations below where seasonal moisture fluctuations typically occur.

7.6 The existing hillside south of the site is located in City of San Diego geologic hazard category 53, which indicates: "sloping terrain, unfavorable geologic structure, low to moderate risk." The existing north facing native slope was evaluated to identify unfavorable geologic structures as a part of this investigation. No unfavorable geologic structures were identified and this upper native slope has been performing well for quite some time. Geologic mapping referenced herein indicates bedding local to the site is neutral with respect to gross stability. Therefore, the slope is considered stable and potential gross instability of the upper native slope is low.

7.7 Shallow soil slips, erosion, and concentrations of cobble material from the up-slope side canyon drainage has blocked the existing drainage inlet for the side canyon drainage, causing runoff to flow over or around the inlet and flow over the top of areas of the lower slopes. These conditions, along with inadequate drainage of the existing terraces and poor maintenance have caused erosion and surficial slippage of areas of the lower cut slope. Thus, to reduce the impacts associated with the blocking of the current drainage inlet, appropriate debris catchments and inlet structure design should be incorporated into the drainage improvements. It should be noted that a landslide has been mapped within the side canyon in the southeast portion of the site and therefore significant debris and sediment could be transported along the side canyon drainage. Thus, measures to intercept cobbles and sediments that will continue to migrate down the drainage should be incorporated in the project design. Also, a regular inspection and maintenance program including sediment removal, etc. should be implemented to maintain the new drainage facilities.

7.8 The existing north facing lower cut was inspected below the upper native slope. These observations did not identify evidence of scarps, lateral displacement, bulging at the base (retaining wall displacement), or steep bedding dipping towards the slope inclination (unfavorable geologic structure) suggesting that deep seated instability of the overall slope had occurred. Further, deeper soils encountered in the borings drilled on the exhibited good shear strength characteristics. The existing adjacent hillsides have performed well with respect to global stability for quite some time. Given these conditions, the geologic nature of the material, and the average 2H to 1V slope inclination, it was concluded that deep seated slope instability is not a concern since the project does not propose to significantly alter the existing cut slope.

7.9 The existing north facing cut slope has a central/eastern area that has been impacted from past washouts, with abundant exposed cobble deposits, and exposed predominantly granular, low cohesion, soils that have exhibited high erosion and shallow slides 1 to 2 feet deep. Also, observations indicate the slope drainage needs improvement. Drainage improvements will reduce, but not eliminate the surficial and erosion issues that have occurred. Thus, some surficial slope movements are anticipated to continue. Considering that the building improvements are planned to be setback at least 35 feet from the slope, impacts to the proposed structures due to shallow slope instability are not anticipated. The current approach by Scottish Rite of maintenance and spot repairs where erosion and slippage has occurred on the cut slopes has been sufficient to maintain function. A similar level of maintenance and repair should be anticipated. In addition, this report recommends that a program of regular inspection of the slopes be implemented to identify conditions that could further degrade shallow slope stability, and to identify areas requiring maintenance and repair/restoration. Slope improvement and slope maintenance recommendations are provided in this report (Sections 8.3 and 8.4) to address these issues during and after construction.

7.10 The western portion of the existing north facing cut slope has areas with mature trees with established native grass undergrowth and bushes which has performed well with respect to gross and surficial stability for quite some time. Therefore, it is recommended not to alter or disturb this western portion of the north facing cut slope.

7.11 The existing variable height cut slope that supports the elevation transitions up to the adjacent church property was observed to be mostly covered with mature trees with established native grass undergrowth and bushes. Considering that no unfavorable geologic structures were identified, the existing slope has performed well for quite some time, and considering that part of the slope extends offsite, it is recommended not to alter this east transition slope.

7.12 Groundwater depths and elevations varied across the site. Groundwater was encountered at a depth of about 29½ feet BSG (elevation of about 24½ feet) in boring A-2, which was drilled in the southwest corner of the site, while in the north portion of the site (proposed Parking Structure) groundwater was encountered at depths ranging from 25 feet to 30 feet BSG. Considering the site elevations, the researched well data, and the range of groundwater depths encountered during the field investigation, a historic high groundwater of about 20 feet was used for analysis.

7.13 The infiltration rates estimated from the percolation tests ranged from 0.1 to 0.5 inches per hour. These rates indicate the soils have a limited infiltration capacity. Based on the results of percolation tests in the silty sands and clays sands, stormwater infiltration systems should consider an un-factored infiltration rate of 0.1 inches per hour. Thus, the infiltration characteristics of the soils tested are poor. However, at this rate, the site may be feasible for a partial infiltration system as defined by the City of San Diego Storm Water Standards.

7.14 The City of San Diego Seismic Safety Study, "Geologic Hazards and Faults", was reviewed. The site is located on Grid Map 21 of the Hazard Map Series. The map shows the northern portion of the site in the area where the parking structure is planned is located within a zone of high potential liquefaction (category 31). However, the Home Depot store area is located outside the liquefaction hazard zone. The results of the liquefaction analyses indicate the potential for liquefaction to impact the site improvements is low due to the depth of soils susceptible to liquefaction. The associated differential seismic settlements were estimated to be ½ inch across the Home Depot store area and ¾ inch across the planned parking structure area.

7.15 The site is not located in a mapped fault rupture hazard zone. The potential for fault rupture on the site is estimated to be low.

7.16 The analytical results of a soil sample analysis indicate that the near-surface soils exhibit a "highly corrosive" to "corrosive" corrosion potential to buried metal objects.

7.17 Chemical analyses indicated a "negligible" potential for sulfate attack on concrete placed in contact with the near surface soils.

8.0 PRELIMINARY RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, the following recommendations are presented for use in the project design and construction. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered.

This report is considered preliminary since the project site and structural details of all the planned improvements were not determined at this time this report was completed.

Where the requirements of a governing agency, utility agency or pipe manufacturer differ from the recommendations of this report, the more stringent recommendations should be applied to the project.

8.1 General

8.1.1 Updated grading and drainage plans, and foundation plans, when available, should be provided to Moore Twining for review to determine if the following preliminary recommendations need to be updated or revised. Once these details are provided, a design level geotechnical report should be prepared to provide specific recommendations for final design prior to bidding and construction. The recommendations presented in this report could change depending on the extent of proposed grading, etc. Therefore, it is critical that updated improvement plans, when available, be provided to Moore Twining for review.

8.1.2 Preliminary foundation loading information was used as noted in the Anticipated Construction section of this report. Once the initial structural design is completed, the column and wall loading information should be provided to Moore Twining for review to determine if the recommendations for site preparation are suitable for the actual design loads.

8.1.3 In order to obtain additional information for use in contractor's bidding the project, a supplemental investigation is recommended to assess the gradation and range in size of the over-size materials contained in soils within the excavation areas. The investigation should include subsurface exploration using test pits located in areas and through depths of identified cobble layers to note the fraction and range of sizes that could be encountered during grading.

8.1.4 A preconstruction meeting including, as a minimum, the owner, general contractor, specialty ground improvement contractor, earthwork contractor, contractor's land surveyor, foundation and paving subcontractors, and Moore Twining should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling.

8.1.5 The subsurface soils encountered include cobble sized rock material (3 to 12 inches in diameter) and very dense gravel material. These materials were encountered at depths as shallow as about 2 feet BSG in the building pad area. Cobbles were encountered in areas and depths across the building pad area which resulted in drilling auger refusal using a CME-75 drill rig in over half the borings drilled for this investigation. Also, cobbles were noted on the cut slopes to the south, and are common in the Mission Valley Formation which underlies much of the site. Cobble materials (exceeding 6 inches in diameter) should not be used as engineered fill within 36 inches of the final pad grade or for trench backfill. Therefore, earthwork bids will be required to include removal of rock, such as by screening/crushing type operations. Also, it should be expected that additional effort may be required to excavate these layers or dense gravels and cobbles during mass grading and installation of deeper utilities. Further, if the native soils are to be used as engineered fill, screening of the excavated soils should be anticipated to remove oversize materials that will allow testing of the precessed soils for compaction and provide uniform support of foundations and floor slabs. Recommendations for the gradation of onsite soils used as engineered fill are included in the Engineered Fill section of this report.

8.1.6 A demolition plan should be developed to identify the existing surface and subsurface improvements to be removed and those which are to remain.

8.1.7 The Contractor(s) bidding on this project should determine if the information included in the construction documents and this geotechnical engineering investigation report are sufficient for accurate bid purposes. If the data are not sufficient, the Contractor shall notify Home Depot in writing that insufficient data are available to prepare an accurate bid for the project.

8.1.8 Contractors should also be aware that wet soils are anticipated that will likely be significantly above the optimum moisture content required for proper compaction and could require soil drying or chemical treatment for stabilization to achieve the required relative compaction. No change orders will be allowed for wet weather conditions, wet soil, soil instability, etc. including chemical treatment, geotextile fabric, rock, soil import, etc.

8.1.9 Appropriate construction methods and equipment, such as low vibration equipment, should be used adjacent to the existing improvements (such as retaining walls) so as not to damage existing improvements which are to remain.

8.2 Building Slope Setbacks, Site Grading, and Drainage for Building Pads

8.2.1 The proposed Home Depot building should be setback horizontally a minimum of 30 feet from the toe of (or retaining wall constructed at the base) of the north facing cut slope. A retaining wall with a minimum of 3 feet of freeboard, as recommended in Section 8.8 of this report, should be placed at the base of the north facing slopes.

8.2.2 The proposed building should also be setback from the toe (or retaining wall constructed at the base) of the west facing transition cut slope by a horizontal distance of at least ½ the height of the slope. A retaining wall with a minimum freeboard of 3 feet, as recommended in Section 8.9 of this report, should be placed at the base of the west facing slope.

8.2.3 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of five percent for a distance of at least ten feet away from the structures to preclude ponding of water adjacent to foundations. Adjacent exterior grades which are paved should be sloped at least 2 percent away from the foundations.

8.2.4 Landscaping after construction should direct rainfall and irrigation runoff away from the structure and not promote ponding of water adjacent to the structures. Care should be taken to maintain a leak-free sprinkler system.

8.2.5 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with low water requirements are recommended.

8.2.6 Perimeter curbs should be extended to the bottom of the aggregate base section, where irrigated landscape areas meet pavements.

8.2.7 It is recommended that landscape planted areas, etc. not be placed adjacent to the building foundations and/or interior slabs-on-grade. Trees should be setback from proposed structures at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

8.2.8 Rain gutters and roof drains should be provided, and connected directly to the site storm drain system. As an alternative, the roof drains should extend a minimum of 5 feet away from the structures and the resulting runoff directed away from the structures.

8.3 Slope Drainage and Debris Catchments

8.3.1 Erosion control including but not limited to establishment of deep rooted vegetation should be provided on the portions of the existing north facing cut slope not covered with deep rooted vegetation such as the scrub brush and thick growth native grasses. On uncovered slopes, appropriate vegetation cover (or other forms of erosion protection as appropriate) should be placed and established on the slopes to provide initial erosion protection until the deeper rooted vegetation can be established.

8.3.2 Irrigation in the areas of the slope where vegetation is to be established should be of a drip type system without surface runoff. Lines in sloping areas should not be pressurized when not in use. All irrigation lines and sprinklers should be periodically monitored for leaks. All leaks and damage should be repaired promptly.

8.3.3 Drainage terraces should be designed and constructed to allow for access to clean sediment from ditches, debris catchments, repair any damage to concrete ditches, and to allow inspection of slopes as a part of on-going maintenance.

8.3.4 Concrete lined drainage ditches and downdrains should be provided to intercept surface runoff and drain the flow away from sloped surfaces such that runoff is not allowed to accumulate on the terraces and flow over the tops of lower slopes or retaining walls. Longer surface ditches on the existing terraces graded to drain to the west end have a higher potential for blockage, so it is recommended to intercept ditch flow by use of frequent down drains.

8.3.5 As a minimum, debris fences should be provided on each terrace at the base of the intermediate slopes to intercept cobbles and sediment between the toe and the recommended drainage ditches.

8.3.6 A retaining wall should be placed at the bottom of the north facing slope and should incorporate a 6 foot level drainage terrace behind the wall. Also, a retaining wall should be provided at the base of the west facing slope. These retaining walls at the base of the slopes should be designed with at least 3 feet of "freeboard" to reduce the potential for migration of sediment and other debris such as cobbles.

8.3.7 A debris catchment and inlet structure design should be incorporated into the drainage improvements to collect drainage and debris from the outlet of the side canyon. The drainage catchment should include redundant systems to collect sediment, trash, cobbles, etc. to reduce the potential for clogging.

8.4 Slope Inspection and Maintenance

8.4.1 Inspections of slopes, debris collection areas, drain inlets, and drainage ditches should be performed on a regular basis prior to the start of the wet season, and after major storm events. If any accumulated material is identified behind debris fences, at pipe inlets, or energy dissipation features that could block drainage ditches, pipe inlets; equipment should be mobilized to remove materials and maintain all slope features. Failure to remove accumulated debris, repair shallow slides, or repair damaged fences or ditches will likely result in damage to sloped surfaces and possible damage to the pavement and building improvements at the base of the slope.

8.4.2 If future erosion or instability in the form of slides, debris or earth flow, accelerated erosion, or other forms of slope instability occur on native or graded slopes, Moore Twining should be contacted to provide recommendations for repair, and the distressed areas should be repaired as soon as possible under the direction of Moore Twining. If instability is allowed to continue, these types of conditions could be an impact to the improvements.

8.5 Site Preparation

The following recommendations are for preparation of planned building areas in the relatively flat northern portion of the site. These recommendations assume that significant grading of sloped areas (slopes steeper than 5:1) does not occur. If grading plans change, and slopes will be significantly re-graded (beyond the recommended drainage and erosion control recommended), Moore Twining should be contacted to provide modified recommendations for earthwork on slopes.

8.5.1 Existing surface and subsurface improvements (including buildings, foundations, pavements, canopies, light poles etc.) in the areas of new construction should be excavated and removed from the site and all soils disturbed from the demolition and removal of these improvements should be over-excavated to expose undisturbed soils. The existing 30 inch storm drain pipe that will be abandoned and relocated is shown on Drawing No. 2 in Appendix A. Where present, existing utility trench and retaining wall backfill soils should be excavated from within a zone extending from 1 foot below the wall, foundation, or pipe at a 1H to 1V slope to the ground surface. Foundations, walls and utilities lines should be completely removed and disposed of off-site. Excavations to remove existing improvements should extend to at least 12 inches below the bottom of the improvements to be removed or to the depth required to remove all soils disturbed from demolition, whichever is greater. After over-excavation, and prior to backfill, the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted as engineered fill. Any existing deep foundations encountered during the demolition activities should be removed to a depth of at least 5 feet below finished grade and to the depth necessary to allow for installation of the proposed improvements, whichever is deeper.

8.5.2 All surface topsoil, vegetation, trees, roots, organics, surface and subsurface improvements (if any) should be removed from all work areas. The general depth of stripping should be sufficiently deep to remove the root systems and organic top soils. All roots larger than ¼ inch in diameter or any accumulation of organic matter that will result in an organic content more than 3 percent should be removed and not used as engineered fill. The depth of stripping should be reviewed by our firm at the time of construction.

8.5.3 Oversized (cobble) materials (exceeding 3 inches in diameter) should not be used as engineered fill within 36 inches of the final pad grade or for trench backfill. Also, it should be expected that additional effort may be required to excavate these layers or dense gravels and cobbles during mass grading and installation of deeper utilities. Further, if the native soils are to be used as engineered fill, screening of the excavated soils should be anticipated to remove oversize materials that will allow the placement, compaction, and testing of the processed soils and provide uniform support of foundations and floor slabs.

8.5.4 For the Home Depot store, after stripping and removal of the existing surface and subsurface improvements, the proposed building pad area should be over-excavated to meet all of the following criteria:

1) over-excavate to at least 3 feet below the pre-construction site grade and finished subgrade;

2) over-excavate to at least 2 feet below the bottom of the proposed foundations;

3) over-excavate to the depths required to remove all existing surface and subsurface improvements; and

4) over-excavate to the depth required to remove all undocumented fill and all soils disturbed from demolition.

The limits of the over-excavation for the building pad should include the footprint of the entire building, all foundations, vestibules, the building exterior concrete apron, lumber canopy, drive-though area, materials storage areas, exterior walkways, stairs, stoops, loading dock, and a minimum of five (5) feet beyond the edges of these improvements and all the foundations. It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for surveying and measuring to verify the horizontal and vertical extent of over-excavation and compaction. The contractor should verify in writing to the owner and Moore Twining that the horizontal and vertical over-excavation limits were completed in conformance with the recommendations of this report, the project plans, and the specifications (the most stringent applies). This verification should be performed by a licensed surveyor and should include a scaled plan showing the "as-graded" limits (i.e., horizontal and vertical extent) in relation to the proposed pad improvements and the elevations of the bottom of the over-excavation. This verification should be provided prior to placing fill and prior to requesting pad certification from Moore Twining or excavating for foundations. Upon approval of the over-excavation limits (horizontal and vertical) by Moore Twining based on survey data by a licensed surveyor provided by the contractor, the soils exposed at the bottom of the excavation should be should be scarified to a minimum depth of 8 inches, aerated or moisture conditioned to between one (1) and four (4) percent above optimum moisture content, and compacted as engineered fill to achieve a stable condition in accordance with the recommendations of this report.

8.5.5 Since the Parking Garage foundation type and loading was not known at the time this preliminary report was completed, site preparation recommendations are not included in this report. The future Design Level Geotechnical report(s) should include specific recommendations for the site preparation of the north portion of the site designated for the Parking Garage.

8.5.6 Across both the Home Depot and Parking structure areas, some of the clay soils encountered were as much as 10 percent above optimum moisture during our Spring 2019 initial field investigation. At these moisture contents, these soils will need to be aerated, i.e. dried to with 4 percent of optimum, or stabilized to achieve the recommended subgrade compaction. Due to the high soil moisture conditions, it is recommended soil stabilization be included in contractor's bids for the bottom of the over-excavation. For the purpose of preliminary estimates, the contractors should assume that 12 inches of a 1 to ½ inch crushed rock, fully encapsulated in a geotextile filter fabric will be required to stabilize the bottom of the excavation.

8.5.7 The subgrade below the interior Home Depot store concrete slabs-on-grade within the building pad limits should be underlain by 6 inches of aggregate base compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557; placed over 18 inches of imported, non-expansive granular fill compacted as recommended in this report. Note that the slab-on-grade ground floor of the parking structure can be prepared per PCC pavement requirements.

8.5.8 After footing excavations are completed, the moisture content and compaction should be maintained until the reinforcement and concrete are placed.

8.5.9 All undocumented fill soils should be excavated and replaced as engineered fill as part of the site preparation. The boring locations where fill soils were encountered are indicated on Drawing No. 2 in Appendix A. Undocumented fills were identified during this investigation at the following locations and estimated depths:

1) Along Camino Del Rio South roadway frontage to depths of 5 to 6 feet along the east section (various boring locations along line 8) and to a depth of 10 feet in the northwest corner of the site (boring location A-8); and,

2) In the driveway area at the southwest corner of the facility (boring location A-1) to a depth of 4 feet BSG.

It should be noted that due to past grading, larger areas of the frontage along Camino Del Rio South, as well as other areas within the site not identified on the boring logs, may also include undocumented fill soils. The overall extent of the undocumented fill soils will not be known until grading.

Following excavation and removal, the exposed subgrade soils shall be scarified to a minimum depth of 12 inches, moisture conditioned to above optimum moisture content as recommended and compacted to at least 95 percent relative compaction of the maximum dry density as determined by ASTM Test Method D1557 to achieve a stable compacted subgrade.

8.5.10 For pavement areas, exterior slabs outside the building pad preparation limits and areas to receive fill outside the building pad limits, after stripping and removal of existing improvements and undocumented fills, the native subgrade should be prepared by over-excavation to at least 12 inches below the pre-construction subgrade elevation, and to the depth required to remove undocumented fills and soils disturbed during the demolition activity, whichever is greater. As an option to Home Depot, if a higher potential for settlement and maintenance of pavement areas is tolerable to Home Depot, and to limit earthwork costs, the existing fill soils could be left in place. For contractors providing construction estimates or bids on the project, assume that the existing fill soils in the pavement areas will be removed and replaced as engineered fill per Section 8.6 of this report. Optional cost credits should be provided to Home Depot for their consideration to prepare pavement and site areas without over-excavation of the undocumented fill soils.

8.5.11 For retaining walls and miscellaneous lightly loaded foundations for non-building structures, after stripping and removal of existing improvements and undocumented fills, the native subgrade should be prepared by over-excavation to at least 12 inches below the pre-construction site grade, to the depth required to remove undocumented fill, and to 12 inches below the bottom of the foundations, whichever is deeper. Following excavation, the exposed subgrade soils shall be scarified to a minimum depth of 8 inches, moisture conditioned to between one (1) and four (4) percent above optimum moisture content and compacted to at least 95 percent relative compaction of the maximum dry density as determined by ASTM Test Method D1557 to achieve a stable compacted subgrade. The moisture content of the subgrade soils should be maintained until placement of the aggregate base.

8.5.12 Exterior slabs-on-grade outside the building pad limits should be underlain by 6 inches of aggregate base compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557; placed over 12 inches of imported, non-expansive granular fill compacted as recommended in this report. The subgrade soils should be prepared as recommended for the pavement areas in Section 8.5.10 of this report.

8.5.13 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

8.5.14 The moisture content and density of the compacted soils should be maintained until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.

8.5.15 The Contractor should use appropriate equipment, such as low pressure equipment, to achieve the required over-excavation, compaction and subgrade stabilization to prevent rutting and subgrade instability.

8.5.16 Final grading should produce a building pad and prepared subgrade ready to receive the slab-on-grade which is smooth, planar, and resistant to rutting. Both the finished pad (before aggregate base is placed) and the aggregate base section should not depress more than one-half (½) inch under the wheels of a fully loaded concrete truck. If depressions more than one-half (½) inch occur, the contractor shall perform remedial grading to achieve this requirement at no cost to the Owner.

8.5.17 The Contractor should be responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable. The Contractor should be responsible to perform the tests, assessments, etc. to determine the appropriate method of disposal. In addition, the Contractor is responsible for all costs to dispose of these materials in a legal manner.

8.6 Engineered Fill

8.6.1 Interior and exterior concrete slabs on grade within the building pad preparation limits (which includes the building floor slab and all concrete slabs adjacent to the building) should be supported on a minimum of 6 inches of non-recycled aggregate base over 18 inches of non-expansive import fill materials. Exterior concrete slabs-on-grade and PCC paving outside the building pad preparation limits should be supported on a minimum of 6 inches of aggregate base placed over 12 inches of non-expansive import fill materials.

8.6.2 The on-site near surface soils encountered include medium expansive clay materials with areas and depths of high moisture contents and oversized cobble materials. The on-site soils will likely require mechanical screening and or laborers for hand picking to remove over-sized cobble materials and achieve compliance with the requirements of this report for use of the onsite soils as engineered fill. Also, due to expansion characteristics, the onsite soils cannot be used as engineered fill within 24 inches of the bottom of the concrete slabs on grade within the building pad preparation limits, nor within 18 inches of the bottom of exterior slabs and PCC pavement sections which are recommended to be non-expansive materials.

8.6.3 For the building pads and pavement sections, the on-site soils may be used as engineered fill below the recommended non-expansive fill, provided the soils are conditioned/dried to a suitable moisture content, do not contain more than 3 percent organics, and are processed so the soils do not contain particles larger than 3-inches in the top 36 inches of the pad subgrade and not larger than 6 inches for other areas, are processed such that a minimum of 70 percent passes a 3/4 inch sieve, are free of debris and are properly aerated/moisture conditioned to achieve the recommendations of this report. Screening and crushing of the rock fraction may be required to achieve the gradation requirements for reuse of the onsite soils as engineered fill.

8.6.4 Flyash may not be used for treatment of soils on the project.

8.6.5 If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

8.6.6 The compactability of the native soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, it is recommended that they be evaluated by the contractor during preparation of bids and construction of the project.

8.6.7 Import fill soil (if any) should be non-recycled, non-expansive and granular in nature with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	75 - 100
Percent Passing No. 200 Sieve	10 - 40
Expansion Index (ASTM D4829)	Less than 20
Plasticity Index (ASTM D4318)	Less than 15
Organics	Less than 3 percent by weight
Sulfates	< 0.05 percent by weight
Resistivity	> 3,000 ohms-cm
R-value	≥25

Prior to importing fill, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Owner) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. The Contractor shall pay for the environmental testing required to determine compliance with the requirements of this report. This certification shall consist of, as a minimum, recent analytical data specific to the source of the import material including proper chain-of-custody documentation. Moore Twining will sample and test the material after the environmental certification submittal is approved to verify that the proposed material complies with the geotechnical engineering recommendations of this report. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested for the geotechnical properties.

8.6.8 On-site, processed clayey soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between one (1) and four (4) percent above optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of subgrade below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts of fill should not be placed if the previous lift or subgrade is not stable.

8.6.9 On-site non-plastic granular soils or imported granular soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between optimum and three (3) percent above optimum moisture content, and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of subgrade below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts of fill should not be placed if the previous lift or subgrade is not stable.

8.6.10 Utility trenches should be a minimum of 24 inches in width to allow for in-place density testing by traditional (nuclear density test) methods and the backfill should be compacted in accordance with the recommendations for engineered fill.

8.6.11 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at the minimum frequency listed in Table No. 3, below.

Table No. 3
Minimum In-place Density Test Frequency

Area	**Minimum Test Frequency**
Building Pad	1 test per 2,500 square feet per lift

Pavements and Slope Grading	1 test per 5,000 square feet per lift
Utility Pipe and Structure Backfill	1 test per 100 linear feet of trench per compacted lift

8.6.12 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency or pipe manufacturer for use as backfill, or for stabilization of trenches, all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. In addition, periodic slurry cutoffs should be provided along trenches where gravel is placed to reduce potential impacts from groundwater migration through the gravel materials. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Materials such as crushed rock should be placed in thin (less than 8 inches) lifts and each lift should be compacted with a minimum of three (3) passes with a vibratory compactor.

8.6.13 Aggregate base below the building slab should comply with State of California Department of Transportation requirements for a non-recycled Class 2 aggregate base or Crushed Aggregate Base (CAB) from the Standard Specifications for Public Works Construction. The aggregate base used below the building pad should not contain recycled materials. However, a recycled aggregate base may be used for pavement areas outside the building pad, provided that the recycled materials are accepted by the Owner and adequate quality control testing is conducted. Aggregate base should be compacted to a minimum relative compaction of 95 percent. Prior to importing the aggregate base material, the contractor should submit documentation demonstrating that the material meets all the quality requirements (i.e., gradation, R-value, sand equivalent, durability, etc.) for the applicable aggregate base. Also, the Contractor shall test the aggregate base for sulfate content. Documentation should be provided to the Owner, Architect and Moore Twining and reviewed and approved prior to delivery of the aggregate base to the site.

8.7 Foundations

8.7.1 Spread and continuous footings supported on engineered fill soils prepared as recommended in this report may be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads. The weight of the footing and the soil backfill may be ignored in design. The building pad should be prepared in accordance with the recommendations included in the "Site Preparation" section of this report.

8.7.2 Perimeter foundations should extend to a minimum depth of 30 inches below the top of the floor slab. Interior footings should extend to a minimum of 24 inches below the top of the interior floor slab. All footings should have a minimum width of 15 inches, regardless of load.

8.7.3 The foundations should be designed and reinforced for the anticipated settlements and for temperature and shrinkage effects. A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations. For the Home Depot store, structural deign should be based on: 1) a total static settlement and heave of ¾ inch, 2) a differential static settlement of ½ inch in 50 feet, and 3) and a differential seismic settlement of ½ inch in 50 feet.

8.7.4 Parking Garage static settlements will be dependant on the foundation type used and applied loading that have not been determined yet. The future Design Level Geotechnical report should include specific settlement recommendations based on the site preparation of the north portion of the site designated for the Parking Garage. However, based on current analysis, the Parking Structure area of the site could experience post-liquefaction differential seismic settlements of up to ¾ inch in 50 feet.

8.7.5 The foundations should be continuous around the perimeter of the structure to reduce moisture migration beneath the structure. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.

8.7.6 Pylon signs (if any) may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier). An allowable skin friction of 150 pounds per square foot per foot of embedment may be used to resist axial loads. Lateral load resistance may be estimated using the CBC non-constrained design. A value of 150 pounds per square foot per foot of depth may be used.

8.7.7 At the time of pier construction and until the concrete is placed, the shaft excavation should have stable sidewalls and all sloughed soil should be completely removed from the bottom of the excavation. If the drilled hole exhibits instability, it should be cased. Moore Twining should observe the excavation to confirm that the pier was constructed as described above, and the soils encountered are similar to those indicated in this report.

8.7.8 Moore Twining should observe the bottom of foundation excavations prior to the placement of reinforcing steel and utilities. The Contractor shall provide a minimum of 48 hours notice for these observations.

8.8 Seismic Design Factors

The following seismic factors were developed for the site using the Ground Motion Parameter Calculator provided by SEOAC and OSHPD (http://seismicmaps.org), based upon a site latitude of 32.76707 degrees and a site longitude of -117.143846 degrees. The data provided in Table No. 5 are based upon the procedures of Sections 1613.2.1 through 1613.2.4 of the 2019 California Building Code, ASCE 7-16 Chapter 11 and Supplement No. 1. The data in Table No. 5 were not determined based upon a ground motion hazard analysis. The structural engineer should review the values in Table No. 5 and determine whether a ground motion hazard analysis is required for the project considering the seismic design category, structural details, and requirements of ASCE 7-16 (Section 11.4.8 and other applicable sections). If required, Moore Twining should be notified and requested to conduct the additional analysis, develop updated seismic factors for the project, and update the following values.

Table No. 4

Item	CBC Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.617
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration ASCE 7-16 (PGA)	0.561
Spectral Response At Short Period (0.2 Second), Ss	1.244
Spectral Response At 1-Second Period, S_1	0.428
Site Coefficient (based on Spectral Response Short Period), Fa	1.002
Site Coefficient, (based on Spectral Response 1-Second Period) Fv	See Note 1
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	1.247

Item	CBC Value
Maximum considered earthquake spectral response acceleration for 1-second period, S_{M1}	See Note 1
Five percent damped design spectral response acceleration for short period, S_{DS}	0.831

Note 1: Requires ground motion hazard analysis per ASCE Section 21.2 (ASCE 7-16, Section 11.4.8), unless the structural engineer determines that an Exception of Section 11.4.8 of ASCE 7-16 is applicable for the project design.

8.9 Site and Loading Dock Retaining Walls

8.9.1 A retaining wall should be placed at the bottom of the north facing slope and should incorporate a 6 foot level drainage terrace behind the wall. Also, a retaining wall should be provided at the base of the west facing slope. These retaining walls at the base of the slopes should be designed with at least 3 feet of "freeboard" to reduce the potential for migration of sediment and other debris such as cobbles.

8.9.2 The planned retaining walls at the base of the cut slopes may be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot if supported on at least 12-inches of engineered fill (compact the bottoms of footing excavations). However, other on-site lightly loaded retaining wall foundations (i.e., less than 1.5 kips/foot line loading) may be designed using an allowable soil bearing pressure of 1,500 pounds per square foot or less may be supported on shallow footings placed entirely on 6 inches of engineered fill.

8.9.3 Retaining walls should be constructed with imported granular backfill placed within the zone extending from a distance of 1 foot laterally from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. This requirement should be detailed on the construction drawings. Granular backfill will reduce the effects of expansive soil pressures on the wall. Granular wall backfill should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	70 - 100
Percent Passing No. 200 Sieve	10 - 15
Plasticity Index	Less than 5

8.9.4 The import fill material should be tested and approved as recommended under the subsection entitled "Engineered Fill" in the recommendations section of this report.

8.9.5 Retaining walls should be constructed with a drain system including, as a minimum, drain pipes surrounded by at least 1 cubic foot of crushed ¾ inch or ½ inch rock backfill fully encapsulated in Mirafi 140 N, or equivalent. The final selection of filter fabric should be as recommended by the fabric manufacturer for the specific site conditions. Drain pipes should be located near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system did not function properly. Drainage should be directed to the site storm drain system. The drainage system should be designed by the wall designer and detailed on the plans.

8.9.6 For loading dock area retaining walls only, as an alternative to using drain pipes behind the wall to adequately reduce the potential for hydrostatic pressures behind the wall, weep holes may be used, provided that a continuous crushed rock (minimum 1 cubic foot per lineal foot) and filter fabric section is provided directly behind the wall. The weep holes cannot have the potential for clogging. The weep holes should discharge directly to an approved drainage.

8.9.7 The bottom surface area of concrete footings in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.35 can be used for design.

8.9.8 The allowable passive resistance of the onsite soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 275 pounds per cubic foot. The upper 12 inches of subgrade should be neglected in determining the total passive resistance.

8.9.9 The active and at-rest pressures of the wall backfill using onsite soils in a drained condition may be assumed to be equal to the pressures developed by a fluid with a density of 40 and 60 pounds per cubic foot, respectively. These pressures also assume level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

8.9.10 Since a new retaining wall will be constructed at the base of the cut slopes, for 1.5H to 1V sloped backfill the active and at-rest pressures of the engineered fill may be assumed to be equal to fluids with a density of 77 and 90 pounds per cubic foot, respectively. These pressures do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

8.9.11 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

8.9.12 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a backdrain as recommended in this report.

8.9.13 The wall designer should determine if seismic increments are required. If seismic increments are required, Moore Twining should be contacted for recommendations for seismic geotechnical design considerations for the retaining structures.

8.9.14 It is recommended to use lighter hand operated or walk behind compaction equipment in the zone equal to one wall height behind the wall to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure.

8.9.15 If retaining walls are to be finished with dry wall, plaster, decorative stone, etc., or if effervescence is undesirable, waterproofing measures should be applied to walls. Waterproofing systems should be designed by a qualified professional.

8.10 Interior Concrete Slabs-on-Grade

8.10.1 The recommendations provided herein are intended only for the design of concrete slabs on grade within the building pad and their proposed uses, which do not include construction traffic (i.e., cranes, ready mix concrete mixers, and rock trucks, etc.). The building contractor should assess the slab section and determine its adequacy to support any proposed construction loading.

8.10.2 A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed floor slab. Concrete slabs on grade supported on subgrade soils prepared as recommended in this report should be designed for a total settlement and heave of 1 inch total and ½ inch differential over 50 feet.

8.10.3 A modulus of subgrade reaction of 150 psi/inch may be used for design of the interior floor slab when the subgrade preparation is conducted in accordance with the recommendations of this report. This value is based on a 1 foot square plate and should be adjusted for the size effects based on the plan area of the applied loads.

8.10.4 Concrete slabs on grade within the building pad should be supported on a minimum of 6 inches of non-recycled aggregate base placed over 18 inches of imported, non-expansive granular fill over the depth of engineered fill required below the foundations. The minimum thickness of AB is recommended directly below the slabs-on-grade to improve the slab support characteristics and for construction stability purposes.

8.10.5 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

8.10.6 The moisture content of the subgrade below the aggregate base section should be verified to be optimum to 3 percent above optimum moisture content within 48 hours prior to placing the overlying layer.

8.10.7 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarder when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The vapor retarder should overly the compacted aggregate base. It should be noted that placing the PCC slab directly on the vapor barrier will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the aggregate base and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with the current ASTM C 755, Standard

Practice For Selection of Vapor Retarder For Thermal Insulation and conform to the current ASTM E 154 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier selection and installation conform to the current ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R), Addendum, Vapor Retarder Location and current ASTM E 1643, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of the floor covering and floor covering adhesive be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

8.10.8 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with the manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be caulked per manufacturer's recommendations.

8.10.9 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations.

8.10.10 The moisture retarding membrane is not required beneath exposed concrete floors, such as warehouses and garages, provided that moisture intrusions into the structure are permissible for the design life of the structure.

8.10.11 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.52 lb./lb. or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or pavements adjacent to the structure, 4) providing adequate drainage away from the structure, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structure.

8.10.12 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity of the floor slab, etc., at a frequency and method as specified by the flooring manufacturer, adhesive manufacturer, underlayment manufacturer, etc. or as required by the plans and specifications, whichever is most stringent. The tests should be conducted in accordance with the applicable ASTM test methods. The results of vapor transmission tests, pH tests, internal relative humidity tests of the floor slab, ambient building conditions, etc. should be within floor manufacturer's, adhesive manufacturer's and underlayment manufacturer's specifications at the time the floor is placed. It is recommended that the floor, adhesive and underlayment manufacturers and subcontractor review and approve the test data prior to floor covering installation.

8.10.13 To reduce the potential for damaging slabs during construction the following recommendations are presented: 1) use perimeter pour-strips at tilt-wall locations to avoid damage to slab-wall connections; 2) design for a differential slab movement of ½ inch relative to interior columns; 3) provide aggregate base below the slabs, 4) it is expected that erection of concrete tilt-up wall panels and roof steel may require cranes. The loaded track and/or pad pressure of any crane which will operate on slabs or pavements should be evaluated by the contractor prior to loading the slab.

8.10.14 For tilt up construction, a perimeter pour strip between the wall footing and the adjacent interior slab should be incorporated into the project design. After the walls are erected and a majority of the differential movement has occurred, the pour strip should be placed.

8.10.15 Backfill the zone above the top of footings at interior column locations, building perimeters, and below the bottom of slabs with an approved backfill and/or an aggregate base section as recommended herein for the area below interior slabs-on-grade. This procedure should provide more uniform support for the slabs which may reduce the potential for cracking.

8.10.16 If the pad subgrade or the aggregate base will be used as a working surface, the Contractor should determine an adequate aggregate base section thickness for the type and methods of construction proposed for the project. The proposed compacted subgrade can experience instability under construction loading.

8.10.17 Aggregate base shall comply with the requirements for non-recycled Class 2 Aggregate Base in the Caltrans Standard Specifications and should have negligible concentrations of sulfates. Aggregate base shall be compacted to a minimum relative compaction of 95 percent of the maximum dry density determined in accordance with ASTM D1557. The Contractor shall test the aggregate base for sulfate content and provide the results to the Owner, Architect and Moore Twining for approval prior to delivery of the aggregate base to the site.

8.11 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, rather lightly loaded sidewalks, curbs, and planters, etc. outside the building pad.

8.11.1 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with recommendations presented in this report for interior slabs-on-grade. Moore Twining can provide alternative design recommendations for exterior slabs, if requested.

8.11.2 Subgrade soils for exterior slabs should be prepared as recommended in the "Site Preparation" section of this report. Upon completion of the over-excavation and compaction of subgrade soils, the exterior slabs should be supported on 6 inches of aggregate base placed over 12 inches of imported, non-expansive granular fill overlying subgrade soils prepared in accordance with the recommendations provided in the "Site Preparation" section of this report.

8.11.3 The moisture content of the subgrade soils should be verified to be at least optimum moisture content within 48 hours of placement of the slab-on-grade. In addition, the density and stability of the prepared subgrade should be verified prior to placement of the aggregate base. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.11.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend to at least a depth of 6 inches below the bottom of the slabs-on-grade.

8.11.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing concrete walks and finish work over dry or slightly moist subgrade should be avoided. It is recommended that the general contractor notify Moore Twining to conduct in-place moisture and density tests prior to placing concrete flatwork. Written test results indicating passing density and moisture tests should be in the general contractor's possession prior to placing concrete for exterior flatwork.

8.12 Asphaltic Concrete (AC) Pavements

8.12.1 Areas for AC pavement should be prepared in accordance with the recommendations section entitled, "Site Preparation." The upper 12 inches of subgrade beneath the aggregate base should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.12.2 The following pavement sections are based on an R-value of 15, a traffic index of 6.5 for the "Standard Duty Pavements," and a traffic index of 7.5 for the "Heavy Duty Pavements." If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic.

Traffic Index = 6.5 "Standard Duty Pavements"

AC Thickness, inches	AB Thickness, inches (Min. R-value = 78)	Min. Compacted Subgrade, inches
3.5	12	12

Traffic Index = 7.5 "Heavy Duty Pavements"

AC Thickness, inches	AB Thickness, inches (Min. R-value = 78)	Min. Compacted Subgrade, inches
4.0	15	12

AC - Asphaltic Concrete compacted as recommended in Section 8.12.9 of this report
AB - Aggregate Base compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D1557)

8.12.3 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base section. This should reduce the potential for subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

8.12.4 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions.

8.12.5 If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement sections should be re-evaluated for the anticipated traffic.

8.12.6 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

8.12.7 Pavement materials and construction method should conform to Sections 25, 26, and 39 of the State of California Standard Specification Requirements.

8.12.8 It is recommended that the base 2 inch thick course of asphaltic concrete consist of a ¾ inch maximum medium gradation. The top course or wear course should consist of a ½ inch maximum medium gradation.

8.12.9 The asphaltic concrete, including the joint density, should be compacted to a minimum average relative compaction of 93 percent, with no single test value being below a relative compaction of 91 percent and no single test value being above a relative compaction of 97 percent of the referenced laboratory density according to ASTM D2041.

8.12.10 The asphalt concrete should comply with Type "A" asphalt concrete as described in Section 39 of the State of California Standard Specifications. The Contractor shall provide an asphalt concrete mix design prepared and signed by a California registered civil engineer and approved by Moore Twining and Home Depot prior to construction.

8.13 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural sections are presented in the following subsections. The PCC pavement design assumes a minimum modulus of rupture of 500 psi and was based on the Home Depot traffic loading requirements. A qualified design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic.

8.13.1 Areas to receive PCC slabs-on-grade should be prepared in accordance with the recommendations section entitled, “Site Preparation.” After over-excavation and compaction, the upper 12 inches of subgrade beneath the aggregate base should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.13.2 The "standard duty" pavements and light vehicular loaded pavements were designed based on an 18 kip ESAL of 50,000 using a 10 year design. A design k-value of 200 psi/in considering a recommended 6-inch layer of Class 2 aggregate base material (R-value of 78) over 12 inches of imported non-expansive fill, over the native compacted soils.

Pavement Component	Thickness, Inches
Portland Cement Concrete	6.0
Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Imported Granular Fill* (95% Minimum Relative Compaction)	12.0
Compacted Subgrade (95% Minimum Relative Compaction)	12.0

* Imported Non-Expansive Fill per the gradation requirements of Section 8.6.7

8.13.3 The "heavy duty" pavement section was designed based on an 18 kip ESAL of 220,000, a design period of 10 years, and a k-value of 200 psi/in considering a recommended 6-inch layer of Class 2 aggregate base material (R-value of 78) over 12 inches of imported non-expansive fill, over the native compacted soils.

Pavement Component	Thickness, Inches
Portland Cement Concrete	6.5
Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Imported Granular Fill* (95% Minimum Relative Compaction)	12.0
Compacted Subgrade (95% Minimum Relative Compaction)	12.0

* Imported Non-Expansive Fill per the gradation requirements of Section 8.6.7

8.13.4 The PCC pavement should be constructed in accordance with American Concrete Institute requirements, the requirements of the project plans and specifications, whichever is the most stringent. The pavement design engineer should include appropriate construction details and specifications for construction joints, contraction joints, joint filler, concrete specifications, curing methods, etc.

8.13.5 Concrete used for PCC pavements shall possess a minimum flexural strength (modulus of rupture) of 500 pounds per square inch. A minimum compressive strength of 3,500 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, etc. should be provided by the designer of the PCC pavement.

8.13.6 The pavement section thickness design provided above assumes the design and construction will include sufficient load transfer at construction joints. Coated dowels, keyed joints, Diamond Dowels, etc. are recommended for construction joints to transfer loads. The joint details should be specified by the pavement design engineer and provided on the plans.

8.13.7 Contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade soils. The type of joint filler should be specified by the pavement designer. The joint sealer and filler material should be maintained throughout the life of the pavement.

8.13.8 Contraction joints should have a depth of at least one-fourth the slab thickness, e.g., 1.5-inch for a 6-inch slab. Specifications for contraction joint spacing, timing and depth of sawcuts should be included in the plans and specifications.

8.13.9 Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.

8.13.10 Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet by 12 feet for a 6-inch slab thickness. Regardless of slab thickness, joint spacing should not exceed 15 feet.

8.13.11 Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.

8.13.12 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas.

8.13.13 Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

8.13.14 Pavement construction should conform to the State of California Standard Specifications.

8.14 Underground Storm Water Infiltration Systems

8.14.1 In general, due to the potential for expansion related heave, or settlement from the introduction of water and long term saturation, stormwater infiltration systems which concentrate surface or subsurface water below or adjacent to existing slopes or proposed improvements are not recommended. If these types of features are required, sufficient setbacks to existing improvements and slopes should be maintained. Alternatively, specific measures such as deepened curbs, cutoffs, liners, etc. could be incorporated in the designs to reduce the potential for excessive settlement of improvements due to moisture and free-water migration from storm water systems. Where onsite stormwater system features are required for the project by a regulatory agency, these systems should be setback as far as possible from the proposed structures and improvements which are sensitive to settlement. At a minimum, it is recommended that storm water disposal systems be setback at least 30 feet from the proposed building and all foundations. Storm water infiltration systems below pavements should be expected to require added maintenance and pavement repairs due to differential settlement of the pavements.

8.14.2 A variety of soil types were encountered in the east portion of the Camino Del Rio South frontage proposed for the infiltration system. The soils encountered in the test borings drilled for the percolation tests comprised silty sands and clayey sands. The estimated infiltration rates of the materials tested ranged from 0.1 to 0.5 inches per hour. Considering that clays with less infiltration likely exist, stormwater infiltration systems should be designed for an average un-factored infiltration rate of 0.1 inches per hour. At this rate, the site may be feasible for a partial infiltration system as defined by the City of San Diego Storm Water Standards.

8.14.3 Since the percolation tests do not take into account the long term effects of subgrade saturation, silt accumulation, vegetation, and deeper impermeable clay layers underlying the depths tested for percolation, an appropriate safety factor ranging from 3 to 10 is recommended or as required by the permitting agency, whichever is more stringent. The safety factor should be determined by the designer and should account for the consequences of exceeding the system capacity, regulatory agency requirements, uncertainty in the inflow rate calculations, the potential for artificial compaction of the soils (and subsequent reduction in permeability) during installation of the storm water system, the degree of maintenance that can be relied upon, and such factors as reduction in infiltration rate due to siltation.

8.14.4 The Contractor shall schedule Moore Twining to observe the bottom of the excavation for the subsurface storm water infiltration systems to observe the exposed soil conditions at the bottom of the excavation for consistency with the infiltration characteristics of the soils anticipated based on this investigation. Cemented soils encountered in the excavation (if any) should be removed from the bottom of the infiltration areas and replaced with a suitable drainage/filter material specified by the designer. The Contractor shall schedule Moore Twining to observe the removal of the cemented materials and replacement with the suitable drainage/filter material.

8.14.5 The bottom of the excavations for the infiltration systems should be excavated to a neat, undisturbed condition prior to construction of the storm water infiltration system. Equipment shall not be allowed to operate in the excavation and the contractor's installation procedures should be performed so that compaction of the soils at the bottom of the excavation does not occur. The contractor shall use such procedures as necessary to achieve a smooth, undisturbed condition at the bottom of the excavations.

8.14.6 If an open graded material such as crushed rock is required around the storm water pipes, a crushed rock may be used as bedding, haunching and to 12 inches above the pipe, provided these materials are fully encapsulated in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Open-graded rock, such as gravel, should be placed in thin horizontal lifts (6 to 8 inch lift thickness) and compacted with vibratory equipment. A sufficient space should be provided beyond the storm drain pipes to allow for proper placement and compaction of the haunching and initial fill materials. Native on-site soils or import soils, may be used for the final fill from 12 inches above the pipe to final design grades. Where infiltration systems are buried below pavements, a layer of Mirafi 600X should be placed below the aggregate base layer over the top of the entire storm water disposal system and extending a horizontal distance of 10 feet beyond the outside edge of the storm water disposal system.

8.14.7 Our experience with infiltration systems is that they have a limited life span. Thus, regular maintenance should be expected to maximize the useful life of these facilities and future expansion or modification of these systems should be anticipated to maintain functionality.

8.14.8 After installation, the bottom of storm water system areas should be flooded with a head of six (6) inches of water to induce settlement prior to construction of the overlying pavements. The objective is for the soils below the proposed infiltration system to receive sufficient water that is evenly distributed to saturate the entire bottom and sides of these trenches prior to placement of aggregate base. The contractor will be required to conduct the flooding under the observation of Moore Twining. These requirements should be specified on the plans.

8.14.9 The Contractor should be responsible for arranging for the manufacturer of the infiltration system (i.e., prefabricated infiltration chambers, etc.) to certify in writing that the pipes have been installed in accordance with their standards. The Contractor is responsible to have the manufacturer conduct sufficient site visits and have the manufacturer to verify that the pipes were installed in accordance with the minimum requirements of the manufacturer. These requirements should be specified on the plans.

8.14.10 For the remainder of the storm water system that consists of solid pipe (not perforated), the system should be designed to be "watertight." The manufacturer should certify that the pipes proposed for the project are "watertight." If encountered, leaks should be immediately repaired. Leaking storm drain could result in settlements, sloughing, etc. causing damage to surface and subsurface structures, pavements, flatwork, etc. The Contractor shall inspect the stormwater pipes associated with the storm water disposal system using a video camera inspection prior to placement of pavements and after pre-loading to verify that the pipelines are constructed properly and are "watertight." The Contractor shall provide the video on both tape and CD with an audio and written narration by the video inspection firm to the Owner, confirming the watertight conditions prior to placing pavements or slabs in these areas.

8.15 Temporary Slopes and Excavations

8.15.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. The contractor is responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades, classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures.

8.15.2 Temporary excavations should be constructed in accordance with CAL OSHA requirements. However, temporary cut slopes should also not be steeper than 1.5 to 1, horizontal to vertical, and flatter if possible. If excavations cannot meet these criteria, the temporary excavations should be supported by engineered shoring systems.

8.15.3 In no case should non-shored excavations extend below a 1.5H to 1V zone below existing offsite improvements, utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 1.5H to 1V envelope should be shored to support the soils, foundations, and slabs.

8.15.4 Shoring systems (if required) should be designed by an engineer with experience in designing shoring systems and registered in the State of California. Moore Twining should be provided with the shoring plan to assess whether the plan incorporates the recommendations in this geotechnical report.

8.15.5 Surface sheet flow drainage shall be directed away from the tops of all excavations. Positive drainage shall be established and maintained throughout the construction process.

8.15.6 Excavation and shoring stability should be monitored by the Contractor. Slope gradient estimates provided in this report do not relieve the Contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the structure occurs, during or after excavation, the owners and Moore Twining should be notified immediately and the Contractor should take appropriate actions to minimize further damage or injury.

8.15.7 Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 1.5 horizontal to 1 vertical downward from the bottom of building foundations.

8.16 Utility Trenches

8.16.1 The utility trench subgrade should be prepared by excavation of a neat trench without disturbance to the bottom of the trench. If sidewalls are unstable the Contractor shall either slope the excavation to create a stable sidewall or shore the excavation. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 92 percent relative compaction prior to placement of bedding material. The Contractor is responsible for notifying Moore Twining when these conditions occur and arrange for Moore Twining to observe and test these areas prior to placement of pipe bedding. The Contractor shall use such equipment as necessary to achieve a smooth undisturbed native soil surface at the bottom of the trench with no loose material at the bottom of the trench. The Contractor shall either remove all loose soils or compact the loose soils as engineered fill prior to placement of pipe and backfill of the trench.

8.16.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The bottom of the trench should be compacted as engineered fill prior to placement of the pipe bedding. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported, non-expansive materials moisture conditioned to within optimum to three (3) percent above optimum moisture content and compacted to a minimum of 92 percent relative compaction. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.16.3 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand should be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321 listed in Table No. 5, below (minimum manufacturer requirements). As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Table No. 5
Minimum Trench Widths for HDPE Pipe with Sand (Caltrans Sand Bedding) Initial Backfill

Inside Diameter of HDPE Pipe (inches)	**Outside Diameter of HDPE Pipe (inches)**	**Minimum Trench Width (inches) per ASTM D2321**
12	14.2	30
18	21.5	39
24	28.4	48
36	41.4	64
48	55	80
60	67.3	96

8.16.4 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to reduce the potential for migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining.

8.16.5 Utility trench backfill should be compacted in accordance with the recommendations for engineered fill included in Section 8.6.10 of this report. The Contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

8.16.6 On-site soils and approved imported engineered fill may be used as final backfill in trenches.

8.16.7 Jetting of trench backfill is not allowed to compact the backfill soils.

8.16.8 Where utility trenches extend from the exterior to the interior limits of a building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

8.16.9 Storm drains and/or utility lines should be designed to be watertight. If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil heave causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. It is recommended that the pipelines, stormwater, sewer, water, retaining wall drains, etc. be inspected by video inspection prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are watertight. The Contractor shall provide to Home Depot and Moore Twining a copy of video tape and a written description of the pipe condition prepared by the video inspection firm prior to placement of improvements above the utilities. In addition, the Contractor is required to inspect and test the utility lines as required by the pipe manufacturer and governing agencies.

8.16.10 Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 1.5 horizontal to 1 vertical downward from the bottom of building foundations.

8.16.11 The project Civil Engineer should include slurry type cutoff collars along utility trenches at critical locations to prevent the surface water and groundwater from draining along the trench backfill/bedding material. For bidding purposes, the Contractor should assume for the project a minimum of ten (10) 18- inch wide collars with 1.5 cubic yards of 2-sack concrete per collar.

8.17 Corrosion Protection

8.17.1 Based on the National Association of Corrosion Engineers corrosion severity rating listed in Section 6.10 of this report, the analytical results of sample analyses indicate a “highly corrosive” to “corrosive” corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on these conditions. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

8.17.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentration of sulfates determined for the near-surface soils (negligible exposure). According to provisions of ACI 318, section 4.3, the sulfate concentration falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. Therefore, no restrictions are required regarding the type, water-to-cement ratio, or strength of the concrete used for foundation and slabs due to the sulfate content. However, a low water to cement ratio is recommended for slabs on grade as recommended for exposed concrete slabs to reduce shrinkage.

8.17.3 These soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Moore Twining is not a corrosion engineer; thus, cannot provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

9.0 DESIGN CONSULTATION

9.1 Moore Twining should be retained to review those portions of the contract drawings and specifications that pertain to earthwork operations and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not part of this current contractual agreement.

9.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

9.3 If Moore Twining is not retained for the plan review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

10.0 CONSTRUCTION MONITORING

10.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.

10.2 Moore Twining can conduct the necessary observation and field testing to provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing and conclusions will be provided regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

10.3 In the event that the earthwork operations for this project are conducted such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) it is recommended that the exposed subgrade that will receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.

10.4 The construction monitoring is an integral part of this investigation. This phase of the work provides Moore Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

10.5 If Moore Twining is not afforded the opportunity to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

10.6 Upon the completion of work, a final report should be prepared by Moore Twining. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to prepare a final report summarizing the observations during site preparation activities relative to the recommendations of this report. This service is not, however, part of this current contractual agreement.

11.0 NOTIFICATION AND LIMITATIONS

11.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of subsurface variations between borings may not become evident until construction.

11.2 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

11.3 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (over 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

11.4 Changed site conditions, or relocation of proposed structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

11.5 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction section of this report. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in this report is not recommended. The entity or entities that use or cause to use this report or any portion thereof for other structures or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

11.6 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

11.7 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

11.8 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

11.9 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

We appreciate the opportunity to be of service to Home Depot U.S.A., Inc. on this project. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience at (800) 268-7201.

Sincerely,
MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

Scott W. Krauter, RGE
Assistant Manager

Exp. 5-31-21

Kenneth J. Clark, CEG
Engineering Supervisor

Read L. Andersen, RGE
Manager

List of References

California Division of Mines and Geology (CDMG) Open File report 95-03 titled Landslide Identification Map No. 33; Landslide Hazards in the Southern Part of the San Diego Metropolitan Area by Tan (1995).

CDMG "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California (2002)

CDMG Bulletin 200 - "Geology of the San Diego Metropolitan Area," prepared by Michael P. Kennedy and the California Division of Mines and Geology, (1975)

City of San Diego, Guidelines for Geotechnical Reports, (2018)

City of San Diego, Seismic Safety Study, "Geologic Hazards and Faults" (Grid 21 of the Hazard Map Series) dated April 3, 2008.

City of San Diego, Storm Water Standard - prepared by Geosyntec, effective date October 1, 2018

EDR - Aerial Photo Decade Package for the years 1928, 1949, 1953, 1964, 1966, 1970, 1979, and 1985 of the site (1561 Camino Del Rio South)

Home Depot Design Criteria Manual (dated October 17, 2016)

Geologic Map of the San Diego 30'x60' Quadrangle, California, Regional Geologic Map Series, prepared by the California Geological Survey and compiled by Michael P. Kennedy and Siang S. Tan, dated 2008

K&S Engineering, Inc. - Undated exhibit showing existing topography of the Scottish Rite Events Center property

Lars Andersen & Associates - Various Site plans site plan showing existing topography and the proposed building and improvements dated March 26, 2019 through December 12, 2019.

Lennart and Associates - Soils Investigations and Design Recommendations for First Methodist Church of San Diego; Alvarado Freeway and Texas Street; San Diego, California , dated 1962

Professional Service Industries, Inc. - Geotechnical Engineering Services; Proposed Marriott Residence Inn; Hotel Circle South; Sand Diego, California, dated February 28, 2000

Professional Service Industries, Inc. - Geotechnical Engineering Services; Proposed La Quinta Inn; Hotel Circle South; Sand Diego, California, dated November 30, 1997

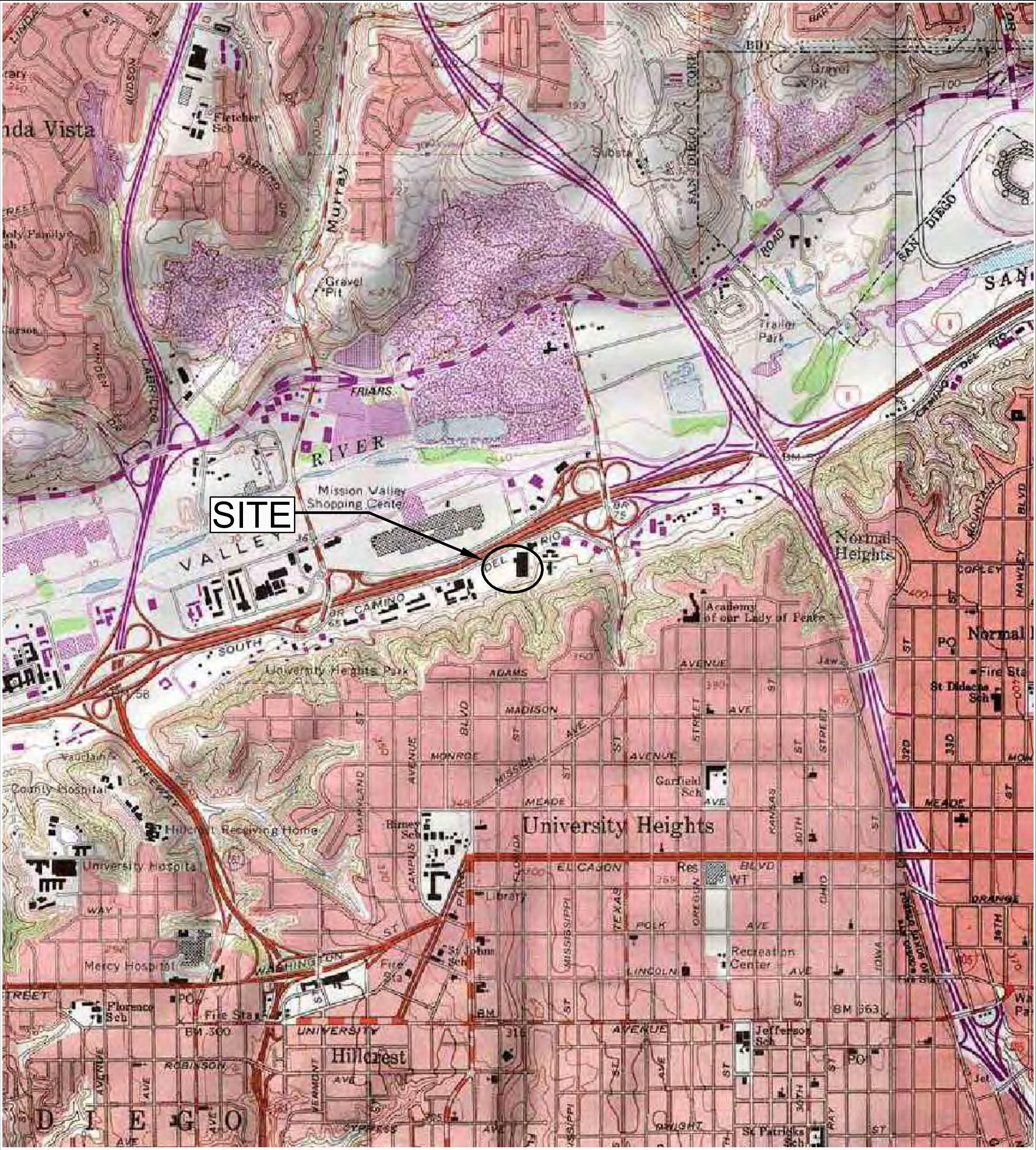
List of References - Continued

San Dieguito Engineering, Inc. - several versions of slope maintenance and improvement plans dated March 26, 2019 through December 2020.

U.S. Geological Survey Professional Paper 851 - Soil Slips, Debris Flows, and Rainstorms in the Santa Monica Mountains and Vicinity, Southern California, prepared by Russell H. Campbell, dated 1975, first printing.

APPENDIX A

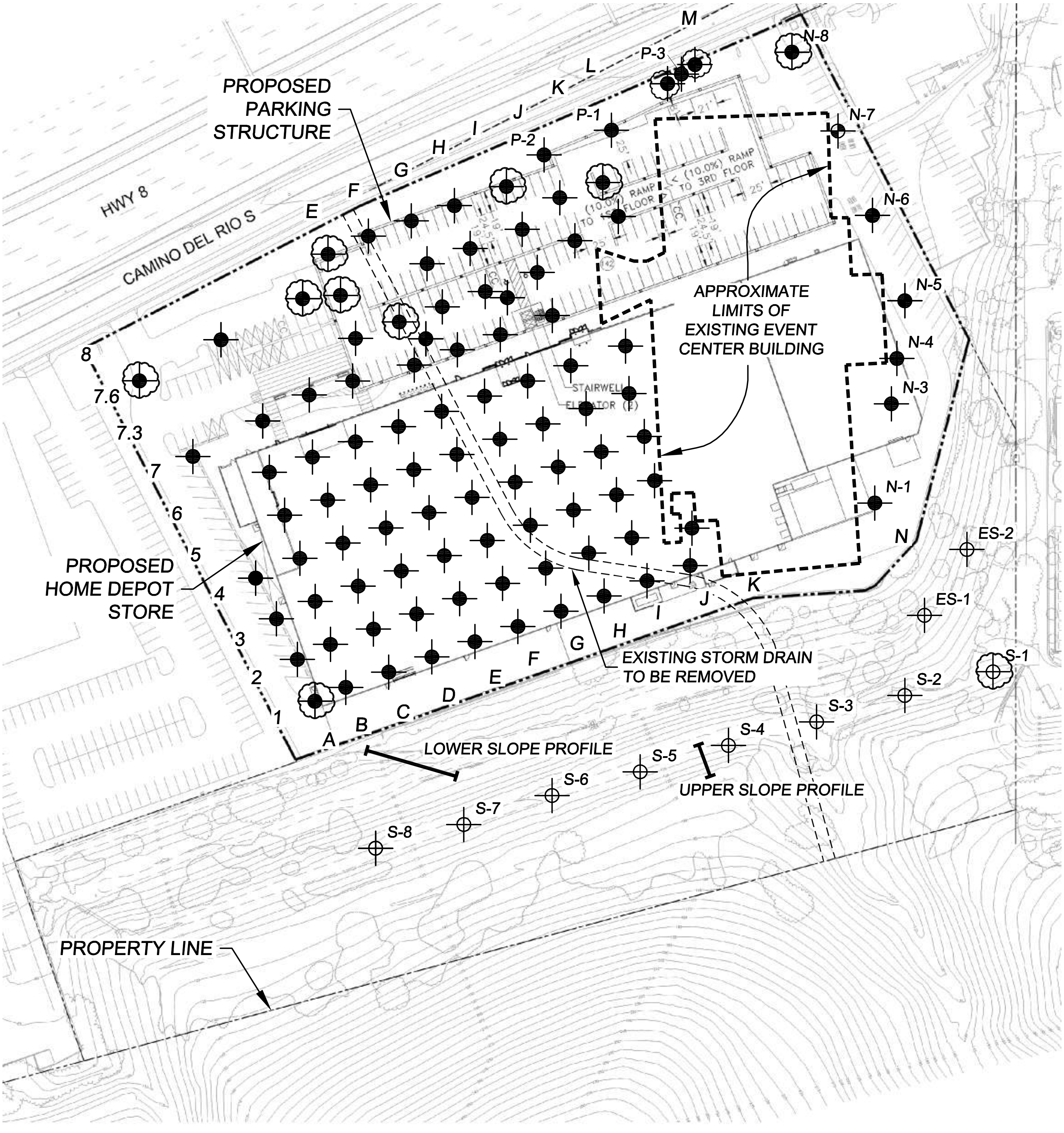
DRAWINGS



SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
LA JOLLA, CALIFORNIA QUADRANGLE 1996

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO.: 050R0-01-01	DATE: 12/20/19	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050R0.01	DRAWING NO. 1	



A-N / 1-8 GRID LINES DESIGNATING BORING NUMBERS

APPROXIMATE TEST BORING LOCATION

APPROXIMATE TEST BORING LOCATION - SLOPE

APPROXIMATE LOCATION OF LOG OF EXPOSED SLOPE MATERIALS (SEE DRAWING NO. 8 & 9)

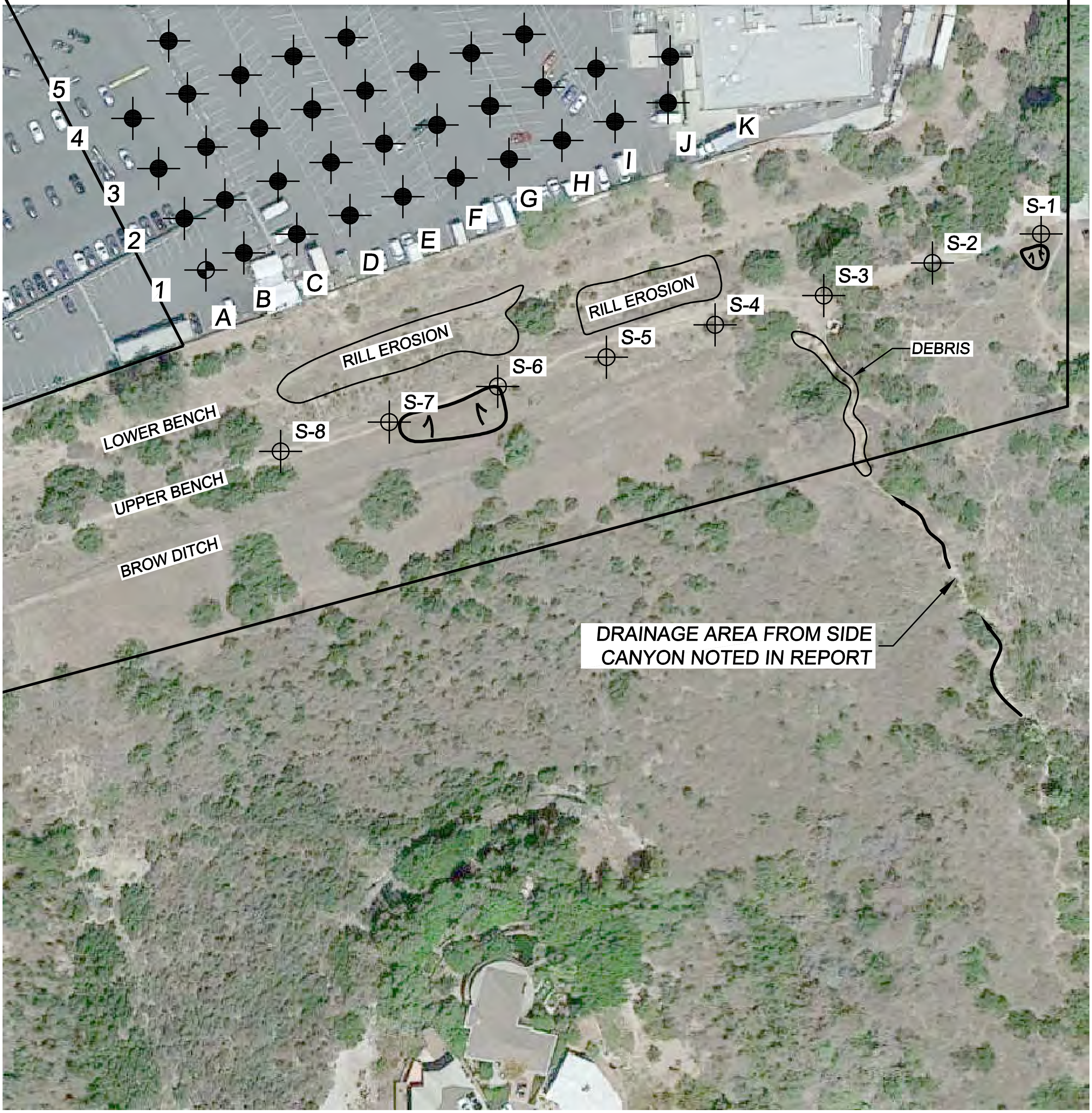
TEST BORING WHERE FILL SOILS WERE ENCOUNTERED

N

0 120

APPROXIMATE SCALE IN FEET

TEST BORING LOCATION MAP PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 050R0-01-02	DATE DRAWN: 01/09/20	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050R0.01	DRAWING NO. 2	



APPROXIMATE AREA OF LARGER, SHALLOW SOIL SLIPS

APPROXIMATE TEST BORING LOCATION - SLOPE

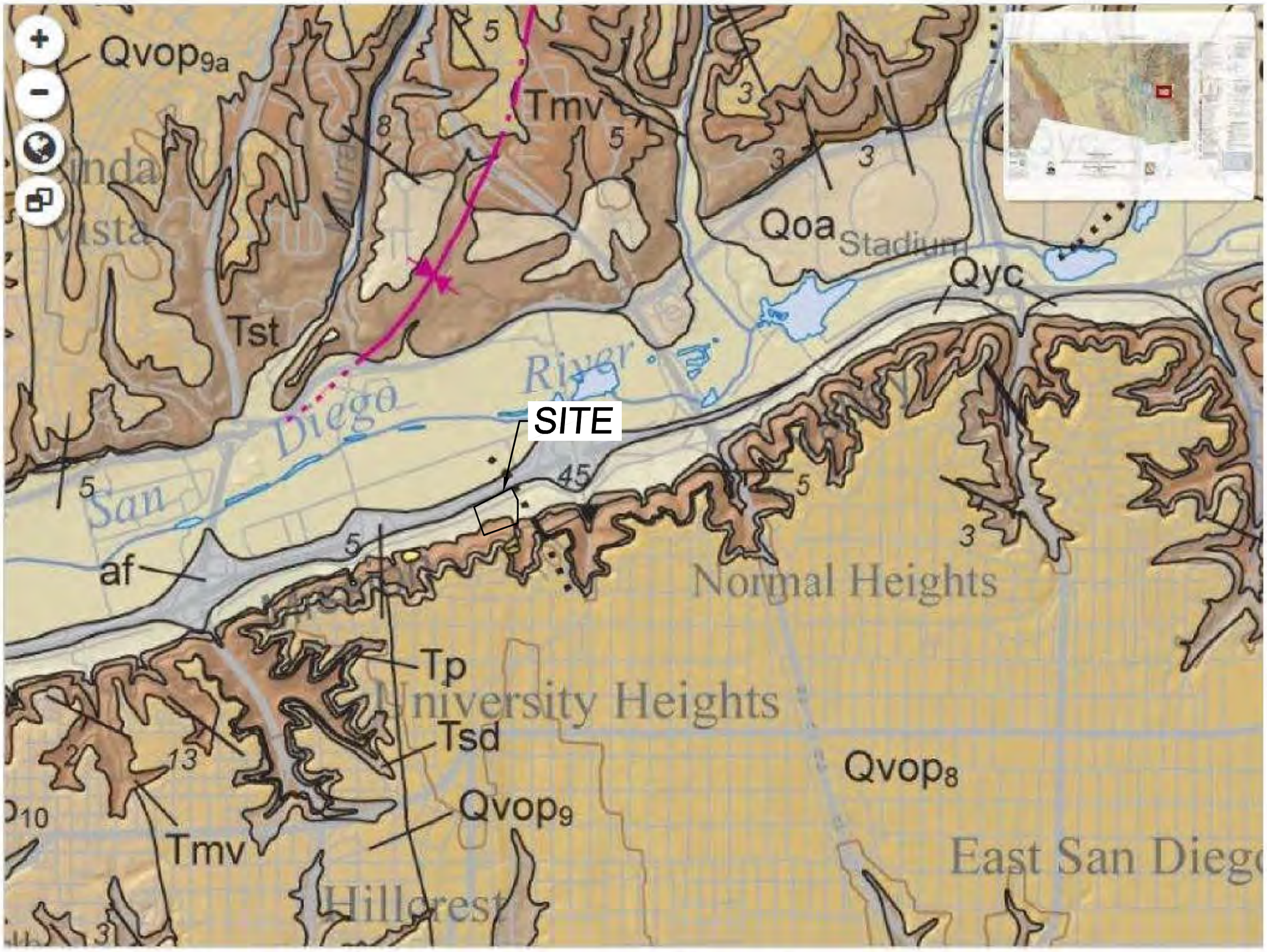
APPROXIMATE TEST BORING LOCATION

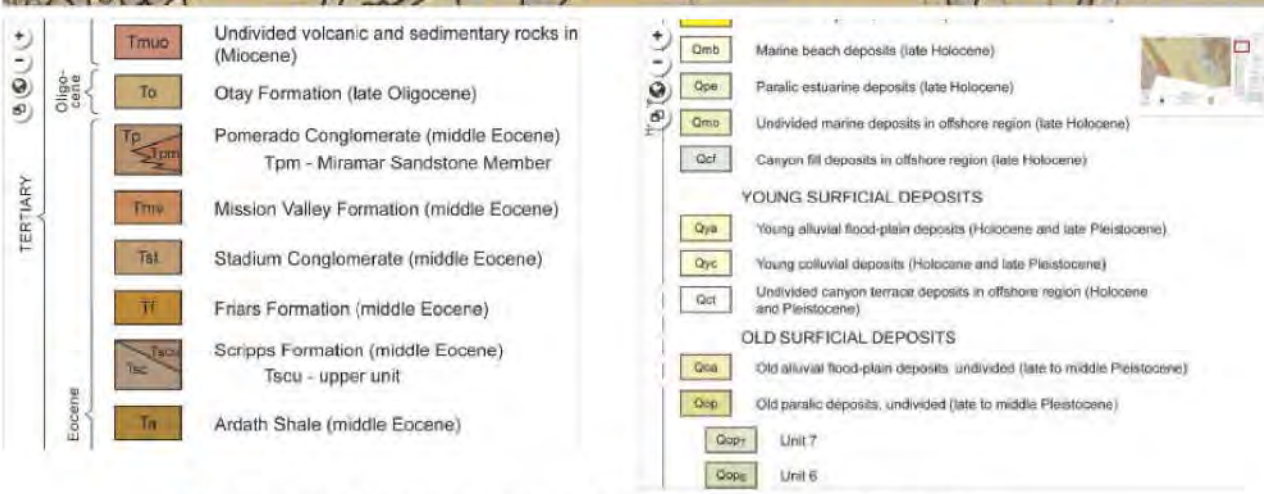
N

0 100

APPROXIMATE SCALE IN FEET

SLOPE OBSERVED CONDITIONS PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 050R0-01-02	DATE DRAWN: 01/09/20	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050R0.01	DRAWING NO. 3	





GEOLOGIC MAP OF THE SAN DIEGO 30' x 60' QUADRANGLE, CALIFORNIA

By

Michael P. Kennedy[1] and Siang S. Tan[1]

2008

Digital preparation by

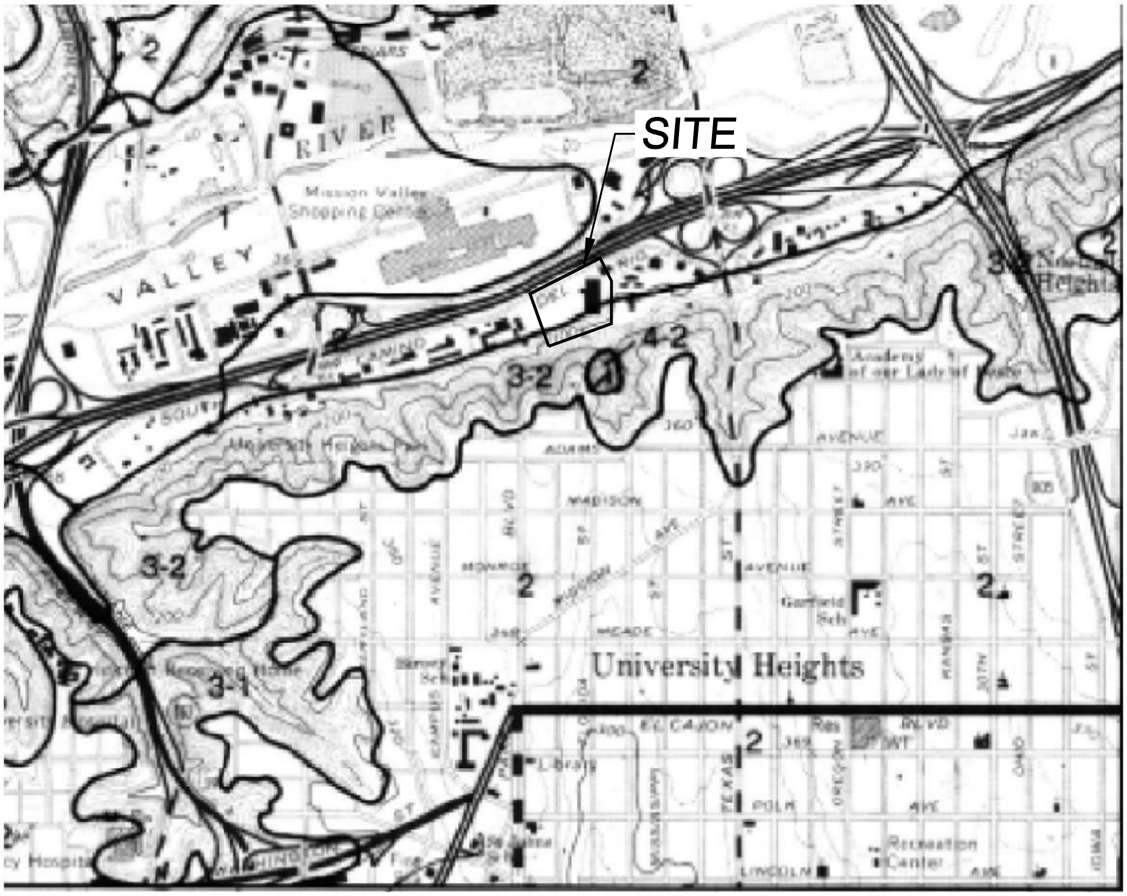
Kelly R. Bovard[2], Anne G. Garcia[2], Diane Burns[2], and Carlos I. Gutierrez[1]

N

0 3500

APPROXIMATE SCALE IN FEET

SITE GEOLOGIC MAP PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 050R0-01-02	DATE DRAWN: 12/20/19	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050R0.01	DRAWING NO. 4	



RELATIVE LANDSLIDE SUSCEPTIBILITY AREAS

1	2	3-1	3-2	4-1	4-2
Least Susceptible	Marginally Susceptible	Generally Susceptible		Most Susceptible	

-------------------------------Increasing landslide susceptibility----------------------------->

N

0 2000

APPROXIMATE SCALE IN FEET

LANDSLIDE HAZARDS IN THE SOUTHERN PART OF THE SAN DIEGO METROPOLITAN AREA, SAN DIEGO COUNTY, CALIFORNIA

by

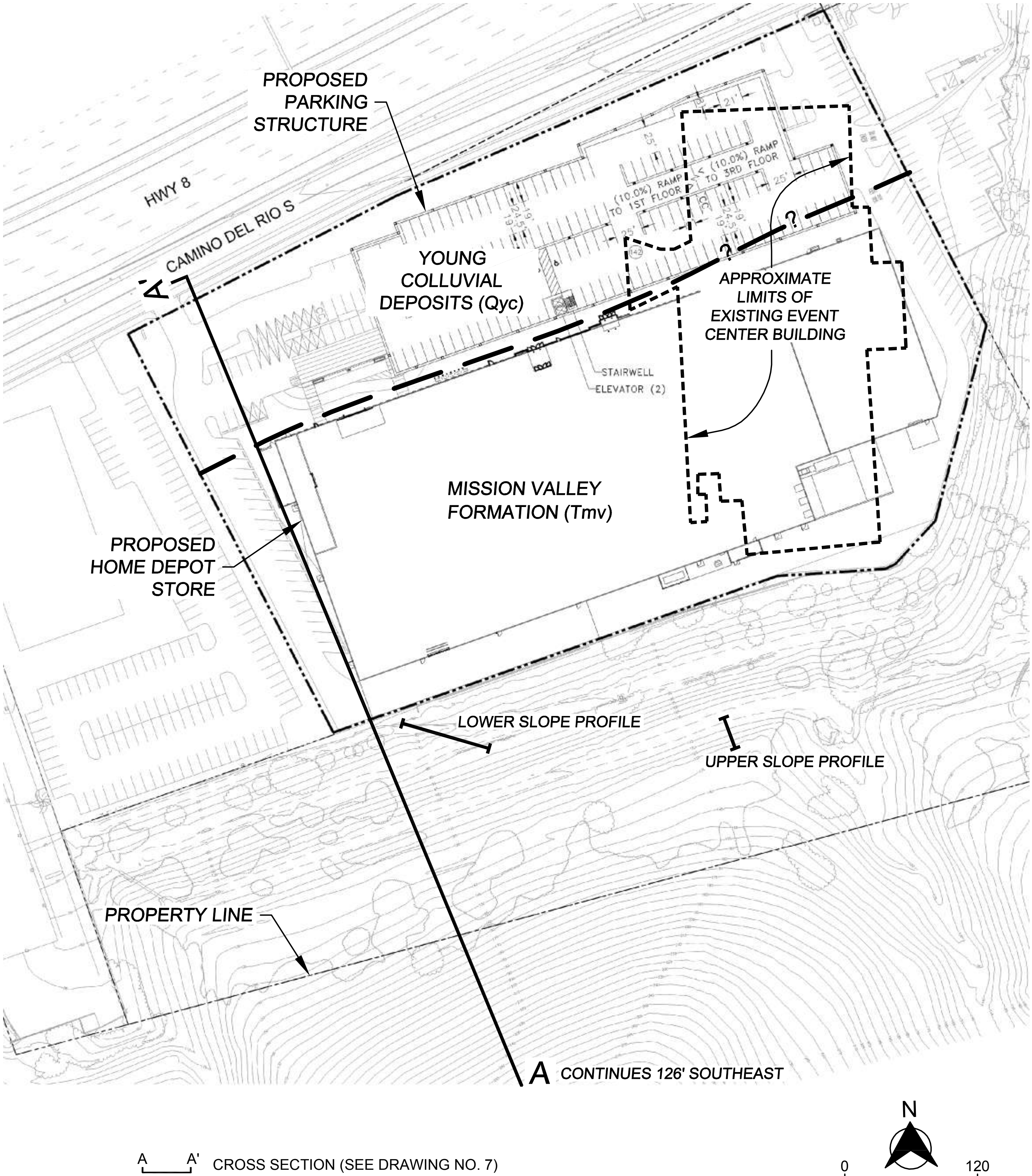
Siang S. Tan
Geologist

1995

RELATIVE LANDSLIDE SUSCEPTIBILITY AND LANDSLIDE DISTRIBUTION MAP

LA JOLLA QUADRANGLE
(PLATE A)

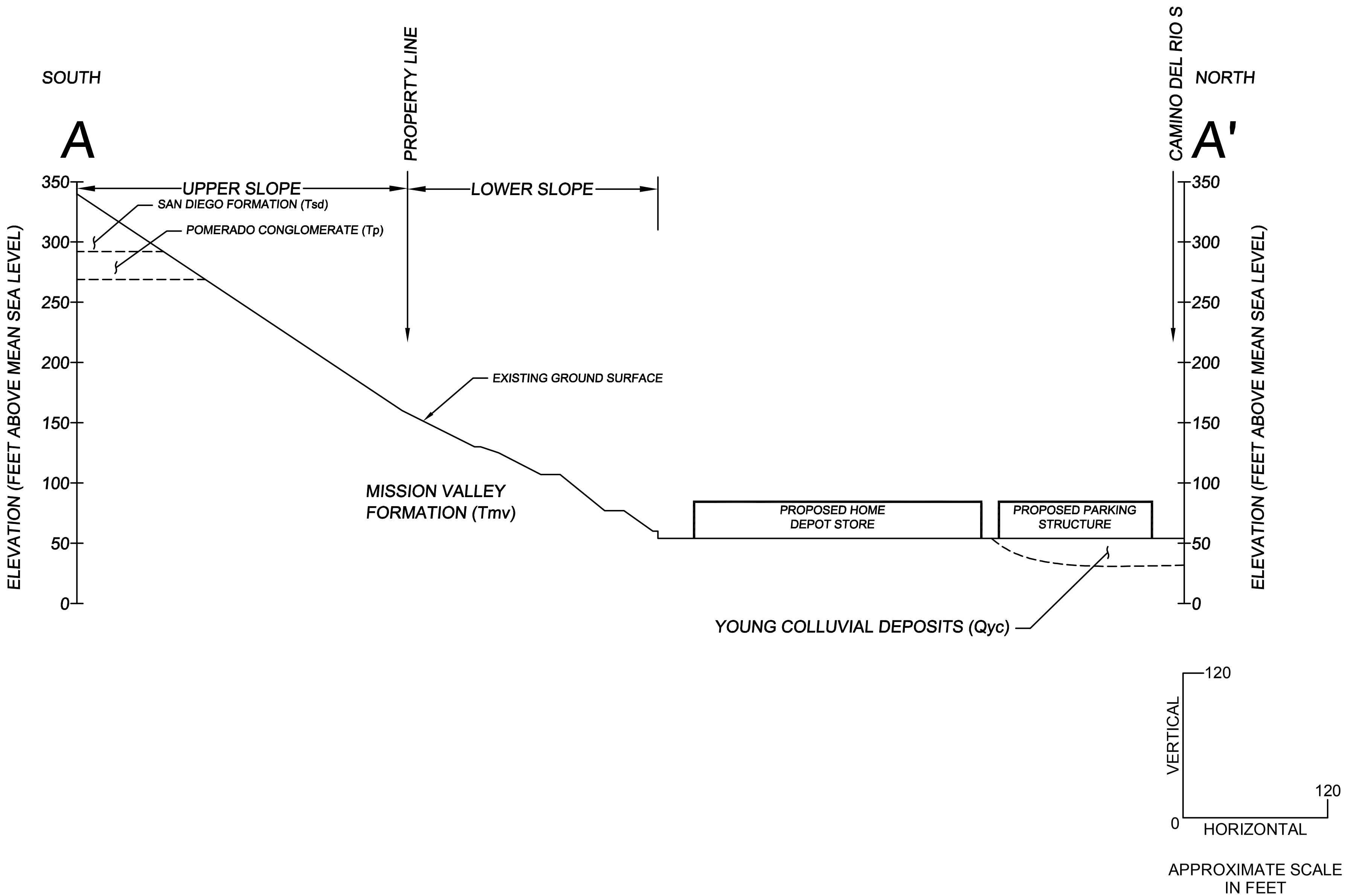
MAPPED LANDSLIDE REFERENCED IN REPORT PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 050R0-01-02	DATE DRAWN: 12/20/19	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050R0.01	DRAWING NO. 5	



A A' CROSS SECTION (SEE DRAWING NO. 7)

APPROXIMATE LOCATION OF LOG OF EXPOSED SLOPE MATERIALS (SEE DRAWING NO. 8 & 9)

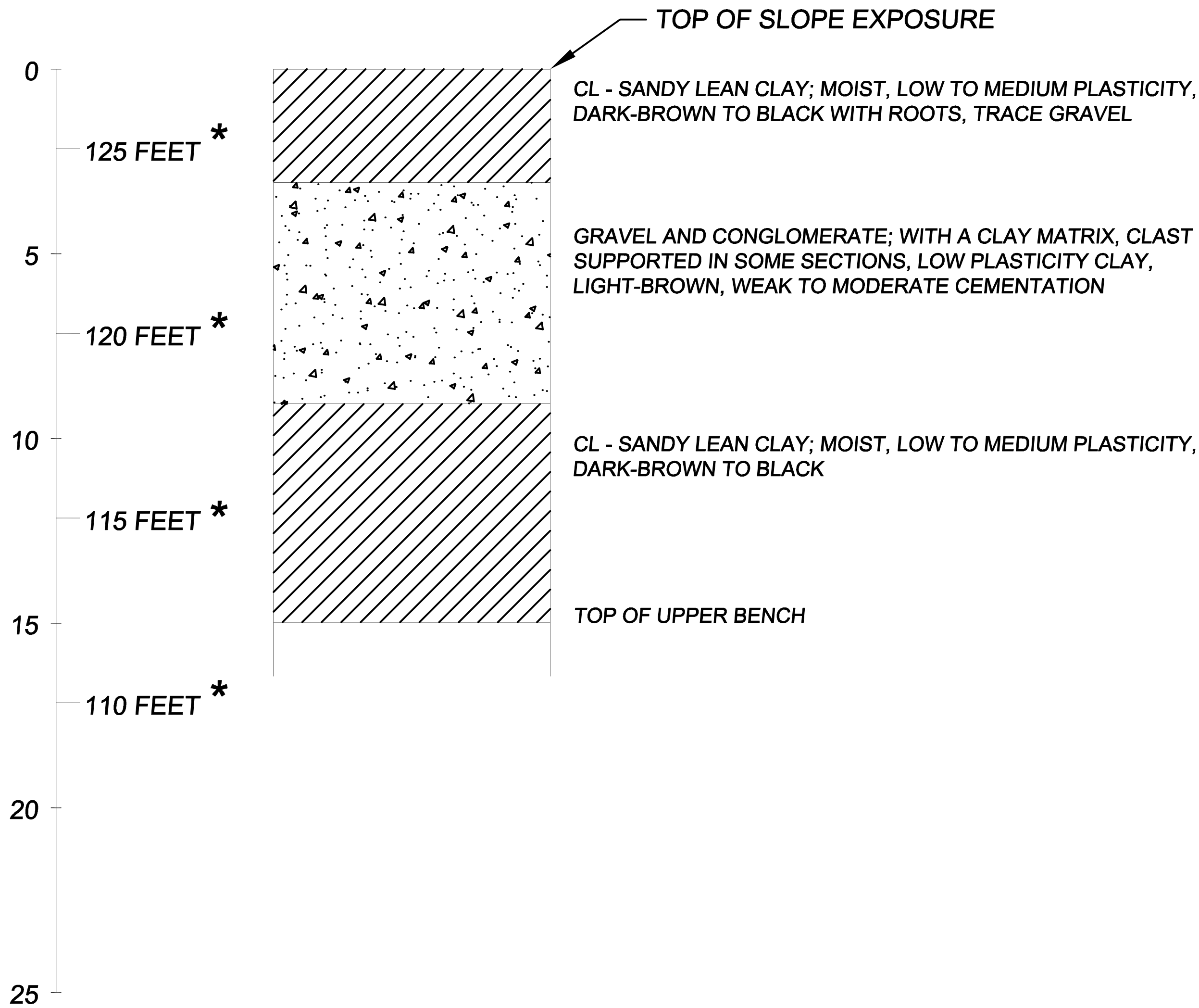
SITE GEOLOGIC MAP PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 050R0-01-02	DATE DRAWN: 01/09/20	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050R0.01	DRAWING NO. 6	



SITE GEOLOGIC CROSS SECTION A-A' PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 050R0-01-02	DATE DRAWN: 01/09/20
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. D050R0.01	DRAWING NO. 7

MOORE TWINING ASSOCIATES, INC.

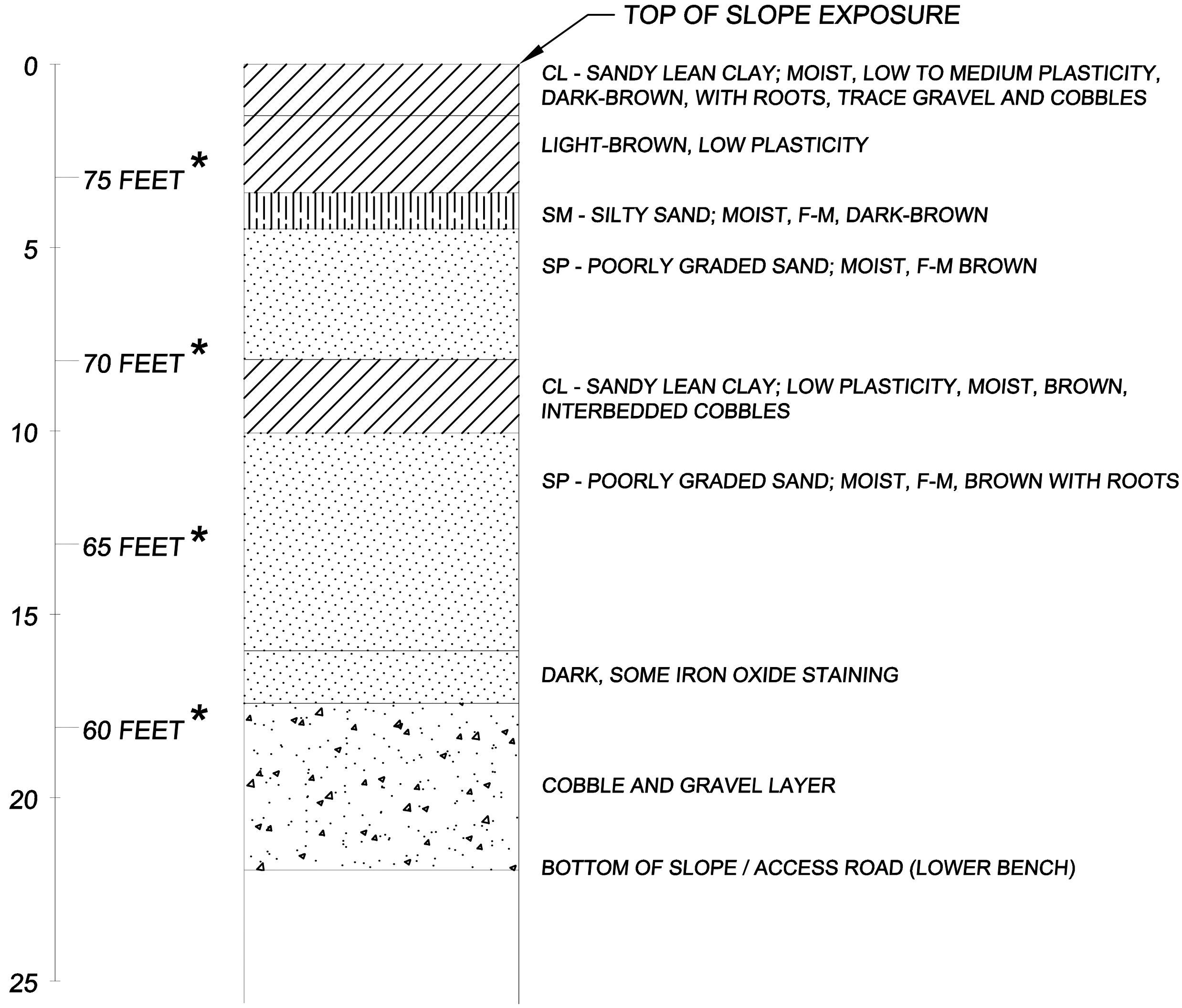
LOG OF "UPPER SLOPE" EXPOSURE



* ESTIMATED ELEVATION IN FEET BASED ON ELEVATION CONTOURS PROVIDED ON SITE PLAN BY SAN DIEGUITO ENGINEERING INC.

UPPER SLOPE SURFACE SOIL STRATIGRAPHY PROFILE PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 050R0-01-02	DATE DRAWN: 01/09/20	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050R0.01	DRAWING NO. 8	

LOG OF "LOWER SLOPE" EXPOSURE



* ESTIMATED ELEVATION IN FEET BASED ON ELEVATION CONTOURS PROVIDED ON SITE PLAN BY SAN DIEGUITO ENGINEERING INC.

LOWER SLOPE SURFACE SOIL STRATIGRAPHY PROFILE (EXPOSED TEMPORARY ROAD CUT) PROPOSED HOME DEPOT STORE 1895 CAMINO DEL RIO SOUTH SAN DIEGO, CALIFORNIA	FILE NO. 050R0-01-02	DATE DRAWN: 01/09/20	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050R0.01	DRAWING NO. 9	

Photograph No.1 – Looking south across Interstate 8 at the site. The upper native and lower benched cut slopes in the background. Also, note drainage canyon in the upper left.

Photograph No. 2 - On the lower cut slope bench looking west. Note recent washout yet to be repaired, and masonry erosion protection features at the base of this section of the middle slope.

Photograph No. 3 - On the upper cut slope bench looking west. Note accumulation of cobble and sediments at the base of the slope.

Photograph No. 4 - Looking west at native slope area above the cut slope (near south property line). Evidence of a brow ditch was noted, but ditch does not show any flow. Note that native grasses are not disturbed, and no evidence of erosion or instability was observed.

Photograph No. 5 - Looking southwest at the native slope area above the cut slope. Note the lower grass covered area and steeper upper potion are covered with undisturbed vegetation. No evidence of erosion, surface sliding, scarps or other features suggesting any recent slope instability are present.

Photograph No. 6 – Looking southwest at east facing drainage canyon slope from the east canyon area. This is the area above the site mapped by Tan (1995) as a slide area. No recent evidence of surface sliding, scarps or other features were noted suggesting recent slope instability.

Photograph No. 7 - Looking to the south at the slope section between upper and lower benches. Note surficial slumps, erosion, exposed soils with cobble material.

Photograph No. 8 - Showing typical a surficial slump observed in east section of upper cut slope (above upper bench)

Photograph No. 9 – Showing standing water on upper bench after recent rain. Bench grades in the area are toward the top of the middle slope section, and drainage blocked from moving west by accumulated sediments at the base of the slope.

Photograph No. 10 - Showing upper cut slope (section west of drainage). Note abundant cobbles, surface erosion and low cohesive soils

Photograph No. 11 - Showing upper cut slope (west section beyond Photograph No. 10 above). Note some surface slumps but less active erosion as noted in section with more granular materials.

Photograph No. 12 - Showing fine grained soils in the upper portion of exposed temporary road cut between base of slope and lower bench.

Photograph No. 13 - closeup of native Sandy Lean Clay Unit.

Photograph No. 14 - Showing granular soils in lower portion of exposed temporary road cut between base of slope and lower bench.

Photograph No. 15 - closeup of native Sand/Gravel/Cobble Unit.

APPENDIX B

LOGS OF BORINGS

This appendix contains the final logs of borings. These logs represent our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these test boring locations. Also, the passage of time may result in changes in the soil conditions at these test boring locations.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: A-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

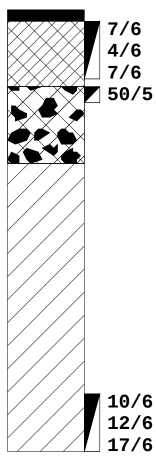
Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/11/2019

Elevation: 55 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 / 0			AC = 3-1/4 inches AB = 1 inch			
	7/6 4/6 7/6	CL	FILL; SANDY LEAN CLAY, medium stiff, damp, low plastic, brown		11	
	50/5	GC	FILL; CLAYEY GRAVEL with Cobbles, very dense coarse gravel and 3 to 5 inch cobbles at 3.5 feet, wire debris in cobbles		>50	
50 / 5		CL	NATIVE: SANDY LEAN CLAY; stiff, damp, low to moderate plasticity, light brown with some fine to medium gravel			
45 / 10	10/6 12/6 17/6				29	
			Bottom of Boring			
40 / 15						
35 / 20						
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: A-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/11/2019

Drill Type: CME-75

Elevation: 54 Feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: 29.4 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	8/6 6/6 7/6	CL	AC = 3 inches AB = 1½ inches SANDY LEAN CLAY; medium stiff, damp, low to moderate plasticity, brown to light brown	RV = 20	13	9
50 / 5	8/6 3/6 7/6		At 5.5 feet, loose Silty Sand layer about 6 inches thick		10	7
	7/6 10/6 14/6		at 7 feet, iron oxide stains, sand content increase in 3 inch thick layers		24	20
45 / 10	10/6 10/6 17/6	SM	SILTY SAND; medium dense, damp, fine grained, gray to gray-brown, some clay	DD= 107.7pcf -200= 33.8% LL = NP	27	7
40 / 15	5/6 6/6 10/6		Color is brown		16	10
35 / 20	4/6 6/6 8/6	CL	SANDY LEAN CLAY; medium stiff, moist, medium plastic, brown trace of fine gravel		14	16
30 / 25	5/6 45/6 16/6		very dense Clayey Sand layer		61	13

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: A-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Logged By: JC

Date: 3/11/2019

Drill Type: CME-75

Elevation: 54 Feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: 29.4 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
24 / 30	11/6 7/6 8/6		very moist, low plasticity, color is brown to light brown some coarse gravel		15	11
19 / 35	8/6 11/6 13/6	SM	SILTY SAND; medium dense, wet, fine grained, brown, trace clay		24	25
14 / 40	2/6 7/6 9/6		some fine gravel	-200 = 15.5%	16	21
9 / 45	25/6 38/6 43/6		Very dense with coarse gravel present		81	18
4 / 50	26/6 50/4				>50	12
			Bottom of Boring			

Elevation scale: 25, 20, 15, 10, 5, 0; Depth scale: 30, 35, 40, 45, 50, 55

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: A-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/12/2019

Elevation: 53 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 4 inches			
	5/6 8/6 9/6	CL	AB = 2 inches		17	6
50	4/6 7/6 12/6		SANDY LEAN CLAY; medium stiff, damp, low to moderate plasticity, brown with fine gravel		19	9
5	2/6 3/6 7/6	SM	SILTY SAND, medium dense, damp, fine to coarse grained, brown with fine gravel and trace clay at 5 feet, more fine grained silt		10	21
45						
10	4/6 7/6 9/6		interbedded with Clayey Sand layers		16	
40						
15	6/6 10/6 12/6		No Clayey Sand layers		22	
35						
20	4/6 7/6 10/6	SC	CLAYEY SAND; medium dense, moist, fine grained, low plastic, brown trace fine gravel		17	
30						
25	4/6 5/6 8/6		Grading to Sandy Lean Clay		17	
			Bottom of Boring			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.
Since 1898

Test Boring: A-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/12/2019

Drill Type: CME-75

Elevation: 52 Feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 4½ inches AB = 1½ inches			
	6/6 7/6 6/6	SC	CLAYEY SAND: medium dense, damp, fine grained, light brown, some fine gravel		13	10
50	7/6 9/6 14/6	SM	SILTY SAND; medium dense, damp, fine grained, light gray, trace of caly		23	4
5	4/6 9/6 14/6	CL	SANDY LEAN CLAY; stiff, moist, low to moderate plasticity, brown interbedded with Silty Sand zones	DD= 99.6 pcf	23	14
45						
10	3/6 9/6 8/6	SC	CLAYEY SAND; medium dense, moist, fine grained, brown		17	
40						
15	4/6 9/6 13/6	SM	SILTY SAND; medium dense, moist, fine grained, light brown to light gray, trace of clay		22	
35						
20	4/6 5/6 7/6		interbedded with Clayey Sand layers		12	
30						
25	4/6 5/6 7/6	CL	SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity, brown to dark brown		12	
25			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: A-7

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

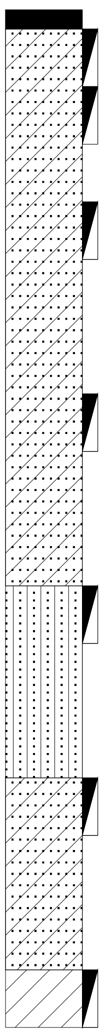
Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/13/2019

Elevation: 48 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	5/6 6/6 7/6	SC	AC = 5-3/4 inches AB = 1 inch CLAYEY SAND with Gravel: medium dense, damp, fine grained sand to fine gravel, brown		13	11
45	4/6 6/6 6/6			no recovery	12	
5	4/6 5/6 5/6		Loose at 5 feet		10	9
40						
10	2/6 3/6 2/6		Loose, color is dark brown		5	
35						
15	2/6 3/6 4/6	SM	SILTY SAND; loose, damp, fine grained, brown with some clay		7	
30						
20	3/6 6/6 8/6	SC	CLAYEY SAND; medium dense, moist, fine grained, brown		14	
25						
25	3/6 7/6 9/6	CL	SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity, brown		16	
20			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: A-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/13/2019

Elevation: 45.5 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
45 / 0			AC = 4-3/4 inches AB = 1-1/2 inches			
	12/6 9/6 7/6	SC	FILL; CLAYEY SAND; medium dense, damp, fine grained, brown, with fine to coarse gravel		16	8
	5/6 6/6 14/6				20	12
40 / 5	5/6 6/6 7/6		wood present, color is brown grading to gray	DD= 105.7 pcf	13	15
35 / 10	2/6 3/6 4/6	SM	SILTY SAND: loose, damp, fine grained, dark gray, trace of clay		7	
			Bottom of Boring			
30 / 15						
25 / 20						
20 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3-11-2019

Elevation: 55 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 / 0	3/6 4/6 7/6	CL	AC = 4 inches AB = None SANDY LEAN CLAY: medium stiff, damp, low to moderate plasticity, light brown to gray, trace coarse gravel		11	14
50 / 5	22/6 17/6 23/6		Hard with fine and coarse gravel		40	5
	9/6 9/6 14/6		Stiff with fine and coarse gravel		23	10
45 / 10	10/6 10/6 18/6		Coarse gravel to fine cobbles	Low Recovery	28	13
40 / 15	15/6 18/6 29/6		trace coarse gravel, brown		47	
	16/6 12/6 16/6		stiff with fine and coarse gravel		28	
35 / 20			Auger refusal at 17 feet BSG, sample extended to 18.5 feet BSG			
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Logged By: JC

Date: 3-12-2019

Drill Type: CME-75

Elevation: 54.5 Feet AMSL

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	7/6 8/6 10/6	SC	AC = 3-3/4 inches AB = 1 inch CLAYEY SAND with Gravel: medium dense, damp, fine grained sand and fine gravel, brown		18	
50 / 5	3/6 6/6 6/6		At 5 feet, interbedded Sandy Lean Clay layer		12	
	23/6 50/4		Coarse grained gravel present, interbedded with Sandy Lean Clay		>50	
45 / 10	7/6 10/6 17/6				27	
		SM	SILTY SAND: medium dense, damp, fine to coarse grained, light brown to gray-brown, trace of clay			
40 / 15	4/6 8/6 12/6	ML	SANDY SILT; stiff, moist, slight plasticity, brown		20	
35 / 20	8/6 5/6 8/6	CL	SANDY LEAN CLAY; medium stiff, moist, low plasticity, brown		13	
30 / 25	34/6 17/6 12/6		Very stiff, increase in plasticity		29	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3-12-2019

Elevation: 54.5 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
25 / 30	28/6 14/6 6/6				20	
			Bottom of Boring			
20 / 35						
15 / 40						
10 / 45						
5 / 50						
0 / 55						

Notes:

Figure Number

Test Boring: B-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3-12-2019

Elevation: 54 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	10/6 8/6 8/6	CL	AC = 4-3/4 inches AB = none		16	
	5/6 7/6 9/6		SANDY LEAN CLAY with Gravel: medium stiff, damp, low to moderate plasticity, brown with fine gravel, interbedded with Silty Sand zones At 2 feet, no Silty SAND and no gravel Hard at 5 feet		16	
50 / 5	11/6 23/6 34/6			sample disturbed	57	12
45 / 10	10/6 8/6 5/6	SM	SILTY SAND; medium dense, damp, fine grained, light gray to gray-brown, trace of clay		13	
40 / 15	14/6 28/6 35/6			No Recovery	63	
35 / 20	6/6 6/6 7/6		Interbedded with Sandy Lean Clay layers		13	
30 / 25	2/6 3/6 7/6	SC	CLAYEY SAND; loose, moist, fine grained, brown, trace of fine gravel		10	
			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3-12-2019

Elevation: 52 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	5/6 8/6 7/6	SC	AC = 4 inches AB = 2 inches		15	
50	9/6 12/6 14/6	SM	CLAYEY SAND; medium dense, damp, fine grained, interbdded non-plastic zones, brown, with trace fine gravel		26	
5 / 45	15/6 23/6 21/6		SILTY SAND; medium dense, damp, fine grained, light gray, trace of clay		44	13
10 / 40	2/6 5/6 6/6	CL	SANDY LEAN CLAY; medium stiff, damp, low to moderate plasticity, gray brown, interbedded with Clayey SAND zones		11	
15 / 35	4/6 9/6 12/6	SC	CLAYEY SAND; medium dense, damp, fine grained, brown, interbedded Silty SAND zones		21	
20 / 30	3/6 7/6 5/6	SM	SILTY SAND; medium dense, damp, fine grained, brown		12	
25 / 25	2/6 4/6 7/6	CL	SANDY LEAN CLAY; medium stiff, damp, low to moderate plasticity, gray brown		11	
			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2-28-2019

Elevation: 51 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0 / 50			AC = 4 inches AB = 3 inches			
		CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, dark brown			
	14/6 15/6 20/6		at 3.5 feet, increase in sand, 3 inch thick clay seam		35	
5 / 45	4/6 7/6 9/6		at 5 feet, decrease in sand		16	
10 / 40	3/6 6/6 10/6	ML	SANDY SILT; very stiff, moist, non-plastic, dark brown		16	
15 / 35	4/6 5/6 8/6		stiff		13	
20 / 30	3/6 7/6 11/6		very stiff, slight plasticity		18	
25 / 25	3/6 5/6 7/6		grading to brown Sandy LEAN CLAY		12	
			Bottom of Boring			

Notes:

Figure Number

Test Boring: B-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/26/2019

Elevation: 50 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0			AC = 5 inches			
	3/6 4/6 3/6	CL	AB = 4½ inches		7	14
	3/6 2/6 3/6		SANDY LEAN CLAY; medium stiff, moist, low plasticity, dark brown, trace gravel		5	20
	3/6 12/6 23/6		at 2.5 2 inch thick clay seam at 4.5, becoming hard, increase in moisture	DD = 98.4 pcf	35	13
45 / 5						
40 / 10	3/6 4/6 12/6	ML	SANDY SILT; very stiff, moist, non-plastic, dark brown		16	10
35 / 15	4/6 4/6 8/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, dark brown		12	25
30 / 20	5/6 7/6 10/6		very stiff, gray-brown, slight increase in sand fraction		17	18
25 / 25	3/6 4/6 8/6		stiff, low to moderate plasticity		12	20
			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: B-7

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/28/2019

Elevation: 49 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 6.5 inches AB = 3.5 inches			
	4/6 5/6 7/6	CL	SAND LEAN CLAY; stiff, moist, low plasticity, brown to dark brown; trace gravel		12	
45 / 5	4/6 3/6 3/6 1/6 1/6 1/6	SM	SILTY SAND; loose, moist, fine to medium grained, dark brown, clay lumps very loose at 5 feet		6 3	
40 / 10	2/6 2/6 3/6	CL	SANDY LEAN CLAY; medium stiff, moist, low plasticity, dark brown		5	
35 / 15			Bottom of Boring			
30 / 20						
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem auger

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/28/2019

Elevation: 47 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/A

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches			
	7/6 6/6 7/6	CL	AB = 2 inches		13	
45	6/6 14/6 8/6		SANDY LEAN CLAY; stiff, moist, low plasticity; brown, trace gravel at 2.5 feet, increase to very stiff		22	
5	8/6 11/6 10/6		color is brown to black		21	
40						
10	5/6 7/6 7/6				14	
35			Bottom of boring			
15						
30						
20						
25						
25						
20						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: C-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

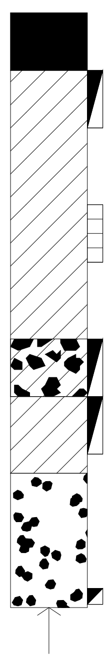
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/25/2019

Elevation: 55 feet AMSL

Depth to Groundwater First Encountered During Drilling: NA



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 / 0			AC = 6 inches AB = 12 inches			
	5/6 5/6 7/6	CL	SANDY LEAN CLAY; medium stiff, moist, moderate plasticity; brown		12	13
50 / 5	8/6 12/6 13/6			DD = 98.9 pcf	25	14
	4/6 17/6 12/6	GC	CLAYEY GRAVEL; medium dense, moist, fine to medium grained, subangular, brown		29	10
45 / 10	6/6 12/6 34/6	CL	SANDY LEAN CLAY w gravel; hard, moist, low plasticty, brown		46	12
		GP	POORLY GRADED GRAVEL; very dense, moist, medium to coarse grained, sub-rounded, difficult drilling			
40 / 15	50/5					
			Auger refusal on dense cobbles			
35 / 20						
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: C-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

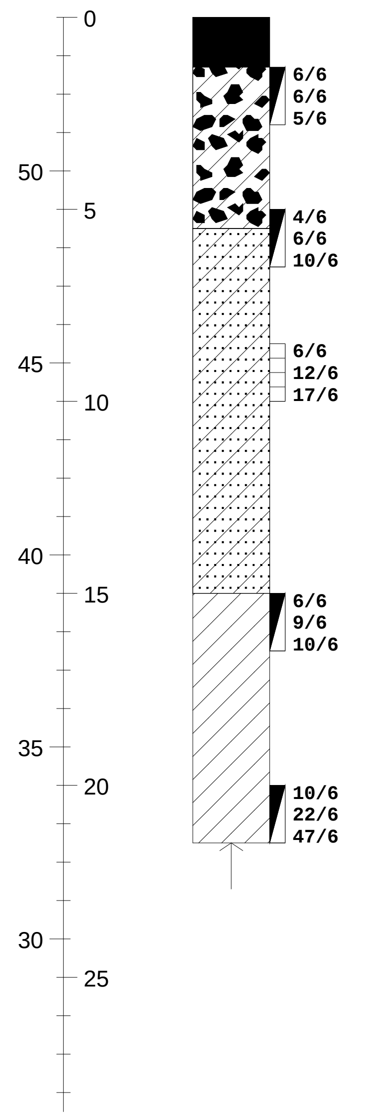
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/25/2019

Elevation: 54 feet AMSL

Depth to Groundwater First Encountered During Drilling: NA



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 6 inches AB = 7½ inches			
	6/6, 6/6, 5/6	GC	CLAYEY GRAVEL; medium dense, damp, fine to medium sub-rounded gravel		11	3
5	4/6, 6/6, 10/6		at 5 feet interbedded clay layer 6 inches thick		16	11
		SC	CLAYEY SAND; dense, moist, fine to medium grained, brown			
10	6/6, 12/6, 17/6			DD= 102.8 pcf	29	17
15	6/6, 9/6, 10/6	CL	SANDY LEAN CLAY; very stiff, moist, low plastic, brown		19	15
20	10/6, 22/6, 47/6		reddish sand grains Grading to Poorly Graded Gravel		69	15
			Auger Refusal on dense gravels and cobbles			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: C-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/25/2019

Elevation: 53 feet AMSL

Depth to Groundwater First Encountered During Drilling: NA

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 8 inches			
			AB = 2 inches			
50	13/6 16/6 13/6	CL	SANDY LEAN CLAY with Gravel; very stiff, moist, low plasticity, brown		29	11
5			Auger Refusal on rounded cobble and dense gravel			

Depth scale: 0, 5, 10, 15, 20, 25; Elevation scale: 50, 45, 40, 35, 30, 25

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: C-3B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: VB

Drilled By: JC

Date: 2/25/2019

Drill Type: CME-75

Elevation: 53 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: NA

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 8 inches			
			AB = 2 inches			
	4/6, 6/6, 8/6	CL	SANDY LEAN CLAY with Gravel; very stiff, moist, low plasticity, brown		14	14
50	7/6, 7/6, 8/6				15	18
5	21/6, 16/6, 15/6			DD= 107.5 pcf	31	16
			Auger Refusal on rounded cobble and dense gravel			

Depth scale: 0, 5, 10, 15, 20, 25; Elevation scale: 50, 45, 40, 35, 30, 25

Notes: *boring drilled 5 feet north of B-3*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: C-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/25/2019

Elevation: 52 feet AMSL

Depth to Groundwater First Encountered During Drilling: NA

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 8 inches			
			AB = 2 inches			
50	3/6 6/6 4/6	CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, brown		10	17
	6/6 8/6 8/6				16	16
5	3/6 7/6 10/6	SC	CLAYEY SAND; medium dense, moist, low plastic, brown		17	11
45						
	9/6 9/6 10/6				19	12
10			Auger Refusal on Cobble Material			
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: C-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/26/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: NA

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 8 inches			
50			AB = none encountered			
	5/6 5/6 9/6	CL	SANDY LEAN CLAY; stiff, moist, moderate plasticity; dark brown		14	13
	6/6 9/6 11/6		at 3.5 feet, color change to light brown		20	25
5 / 45	4/6 9/6 15/6		increase in sand	DD= 103.4 pcf ø = 35° c = 290 psf	24	16
10 / 40	5/6 7/6 11/6				18	26
15 / 35	5/6 9/6 10/6		less plastic, possibly silt		19	19
20 / 30	6/6 7/6 8/6				15	18
25 / 25	8/6 5/6 7/6				12	20
			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: C-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 2/26/2019

Drill Type: CME-75

Elevation: 50 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: NA

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0			AC = 7 inches			
			AB = 4 inches			
		GP	GRAVEL and COBBLES 3 to 6 inches; sub-rounded			
	7/6 11/6 19/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, dark brown with iron oxide staining	DD= 91.8 pcf	30	19
	3/6 4/6 7/6		At 3.5 feet; 3 inch thick clay seam		11	29
45 / 5	5/6 10/6 11/6		At 5 feet, color is light brown, iron oxide staining		21	4
40 / 10	5/6 8/6 8/6	ML	SANDY SILT; very stiff, moist, non-plastic, dark brown		16	24
35 / 15	5/6 8/6 8/6		decrease in sand content		16	24
30 / 20	4/6 4/6 6/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, gray brown		10	17
25 / 25	3/6 5/6 6/6		low to medium plasticity; iron oxide staining		11	21
			Bottom of Boring			

Notes:

Figure Number

Test Boring: C-7

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/28/2019

Elevation: 47 feet AMSL

Depth to Groundwater First Encountered During Drilling: NA

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches			
	4/6 6/6 9/6	SM	AB = 2 inches		15	
45	7/6 11/6 14/6	SP-SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, clay lumps		25	
5	33/6 45/6 35/6		POORLY GRADED SAND with silt; medium dense, moist, fine to medium grained, brown at 5 feet, becoming dense		54	
40						
10	33/6 45/6 10/6	CL	SANDY LEAN CLAY; hard, moist, low plasticity, dark brown, iron oxide stains		55	
35						
15	4/6 3/6 3/6	SM	SILTY SAND; loose, moist, fine to medium grained, dark brown, 3 inch thick clay seam		6	
30						
20	11/6 4/6 11/6	CL	SANDY LEAN CLAY; stiff, moist, low to moderate plasticity, dark brown		15	
25						
25	4/6 6/6 10/6				16	
20			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: C-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/28/2019

Elevation: 45 feet AMSL

Depth to Groundwater First Encountered During Drilling: NA

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
45 / 0			AC = 8 inches			
			AB = 3 inches			
	3/6 5/6 6/6	CL	FILL; SANDY LEAN CLAY; stiff, moist, low plastic, brown to dark brown trace gravel		11	
	4/6 5/6 5/6				10	
40 / 5	2/6 1/6 2/6				3	
35 / 10	3/6 3/6 4/6		medium stiff, color is black		7	
			Bottom of Boring			
30 / 15						
25 / 20						
20 / 25						

Notes:

Figure Number

Test Boring: D-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

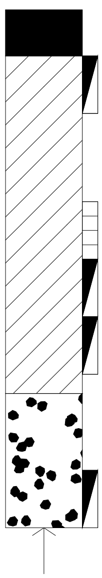
Logged By: JC

Date: 2/25/2019

Elevation: 55.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/DEPTH (feet) scale: Elevation 55, 50, 45, 40, 35, 30; Depth 0, 5, 10, 15, 20, 25



SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		AC = 6 inches AB = 8 inches			
4/6 5/6 8/6	CL	SANDY LEAN CLAY; stiff, moist, low to moderate plasticty, light brown to brown	RV = 22	13	14
9/6 20/6 24/6		hard, increase in sand, color is dark brown	DD= 101.3 pcf	44	12
12/6 19/6 14/6			No sample recovery	33	
8/6 14/6 19/6		color is light brown, coarse gravel present		33	5
	GP	POORLY GRADED GRAVEL with Cobbles and lean clay, hard drilling			
20/6 33/6 43/6		no clay fraction		76	4
		Auger refusal in dense gravels and cobbles			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: D-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 2/25/2019

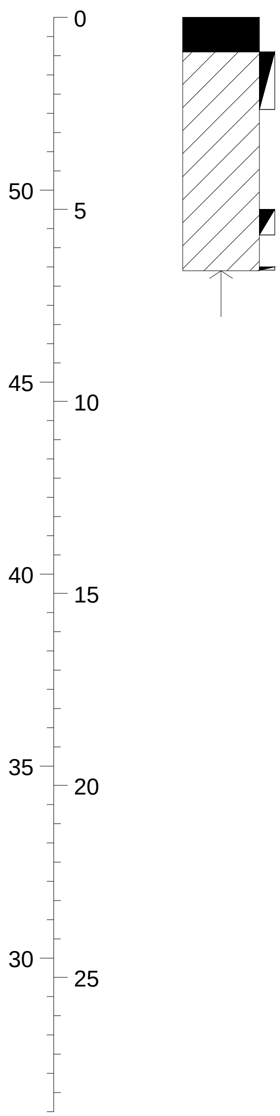
Drill Type: CME-75

Elevation: 54.5 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 6.5 inches			
			AB = 4 inches			
	3/6 5/6 7/6	CL	SANDY LEAN CLAY; stiff, moist, low plastic, light brown, 1 inch thick silty sand seam		12	17
			At 2 feet, coarse gravel and silty sands in drill cuttings			
50 / 5	11/6 50/2		At 5 feet, clay is hard, weakly cemented, coarse gravel		>50	4
	20/0.5				>50	
			Auger and Sampler refusal on dense gravel/cobbles			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: D-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/4/2019

Drill Type: CME-75

Elevation: 53.5 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	5/6 4/6 7/6	CL	AC = 6.5 AB = none SANDY LEAN CLAY; stiff, damp, low plasticity, light brown, some gravel		11	
	12/6 13/6 17/6		at 1.5 feet, hard		30	
50 / 5	40/6 50/2		6 inch thick layer corase grained black silty sand	DD= 114.9 pcf	>50	13
45 / 10	24/6 18/6 14/6		some coarse gravel present		32	
40 / 15	5/6 12/6 13/6		color is light brown to brown		25	
35 / 20	4/6 5/6 7/6		plasticity increase to moderate, less sand		12	
30 / 25	4/6 4/6 8/6		color changing to dark brown with depth		12	
25			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: D-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/4/2019

Elevation: 52.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7½ inches AB = none			
	10/6 6/6 6/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, light brown, some fine and coarse gravel		12	
50	5/6 8/6 13/6		at 2.5 feet, coarse gravel layer		24	
5	10/6 6/6 16/6	SM	SILTY SAND; medium dense, damp, fine to medium grained, brown, with clay lumps and some coarse gravel		22	
45						
10	7/6 14/6 15/6	CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, brown	DD= 99.5 pcf	29	23
40						
15	12/6 15/6 27/7		sand content increase, grading to coarse gravel		42	
35			Auger Refusal on dense gravel / cobbles			
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: D-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/26/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches			
50	4/6 5/6 10/6	CL	AB = 5 inches SANDY LEAN CLAY; stiff, moist, low plasticity, gray at 1 foot, some black organics		16	18
	4/6 5/6 15/6	SM	SILTY SAND, medium dense, moist, medium grained, gray with clay lumps	DD= 98.3 pcf	20	18
5 / 45	14/6 10/6 13/6	CL	SANDY LEAN CLAY; very stiff, moist, moderate plasticity, medium brown		23	8
10 / 40	6/6 7/6 10/6				17	22
15 / 35	3/6 6/6 10/6				16	22
20 / 30	5/6 5/6 7/6		some gravel		12	16
25 / 25	3/6 3/6 4/6		slightly damp		7	20
			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: D-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Logged By: JC

Drill Type: CME-75

Date: 2/25/2019

Auger Type: 6-5/8 inch hollow stem

Elevation: 50 feet AMSL

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0			AC = 6 inches AB = 5 inches			
	13/6 11/6 11/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, light brown with iron oxide staining	DD= 95.7 pcf	22	5
	4/6 5/6 5/6	CL	SANDY LEAN CLAY; stiff, moist, slight to low plasticity, brown, iron oxide staining		10	30
45 / 5	13/6 12/6 13/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, light brown to brown		25	31
		CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, dark brown, iron oxide staining			
40 / 10	4/6 5/6 6/6				11	29
35 / 15	3/6 5/6 11/6		less plasticity		16	20
30 / 20	3/6 5/6 5/6		color is gray-brown		10	18
25 / 25	4/6 5/6 5/6		moderate plasticity		10	22
			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: D-7

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/28/2019

Elevation: 49 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 6 inches			
			AB = 3½ inches			
	8/6 12/6 10/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, trace of clay lumps		22	
45	9/6 14/6 21/6	SP-SM	POORLY GRADED SAND with Silt; dense, very moist, fine to medium grained, brown		35	
5	7/6 5/6 7/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown with iron oxide stains		12	
40 / 10	4/6 9/6 11/6	ML	SANDY SILT; very stiff, moist, non-plastic, dark brown		20	
35 / 15	5/6 10/6 10/6		plasticity increase to slight		20	
30 / 20	5/6 7/6 11/6	CL	SANDY LEAN CLAY; very stiff, moist, low to moderate plasticity, dark gray		18	
25 / 25	4/6 7/6 9/6		color change to dark brown with iron oxide staining		16	
			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: E-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: VB

Drilled By: JC

Date: 2/26/2019

Drill Type: CME-75

Elevation: 55 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 — 0			AC = 10 inches AB = 3 inches			
	2/6 4/6 6/6	CL	SANDY LEAN CLAY; stiff, moist, moderate plasticity; medium brown		10	
50 — 5	2/6 4/6 6/6		increase in sand fracion, also trace of gravel		10	19
	6/6 11/6 14/6			DD= 111.9 pcf	25	14
45 — 10						
	7/6 20/6 16/6 50/1	GC	CLAYEY GRAVEL; dense, moist, fine to coarse grained		36 >50	5
40 — 15			Auger and sampler refusal			
35 — 20						
30 — 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: E-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 2/26/2019

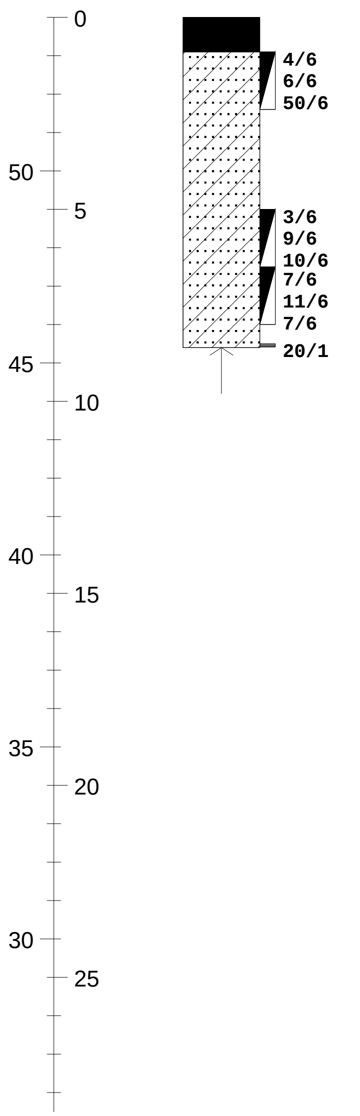
Drill Type: CME-75

Elevation: 54 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches			
			AB = 4 inches			
	4/6 6/6 50/6	SC	CLAYEY SAND; loose, moist, fine to medium grained, olive		56	15
			At 2.5 feet, gravel fraction increase, hard drilling in very dense conditions			
5	3/6 9/6 10/6		At 5 feet, drilling effort reduced, medium dense, color change to light gray with medium gravels		19	16
	7/6 11/6 7/6				18	16
	20/1		sampler refusal		>50	
			Drill refusal on very dense gravel/ cobbles			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: E-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

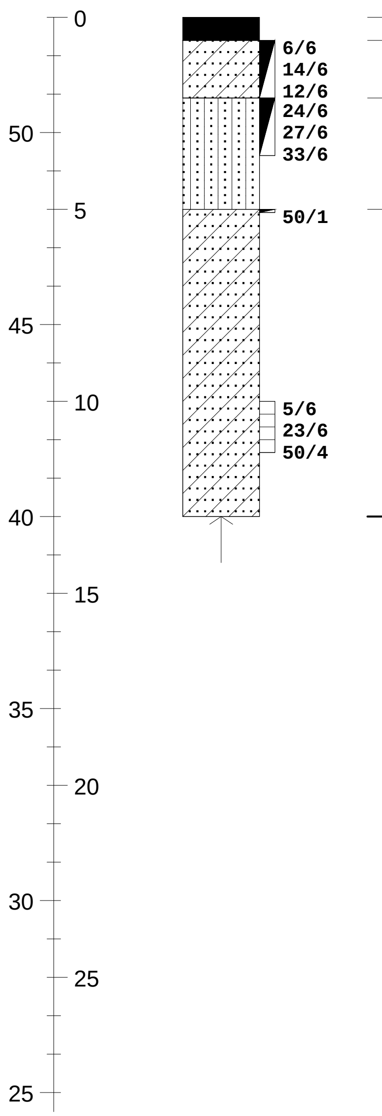
Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/4/2019

Elevation: 53 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
	AC = 7.2 inches AB = None			
SC	CLAYEY SAND; medium dense, damp, fine to coarse grained, light brown, some gravels		26	
SM	SILTY SAND with Gravel; very dense, damp, fine to coarse grained, light brown, some cementation		60	
SC	CLAYEY SAND with Gravel; very dense, fine to coarse grained, brown		>50	
	some fine cobbles in cuttings	DD= 109.2 pcf	>50	17
	Auger Refusal in very dense cobbles/gravels			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: E-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

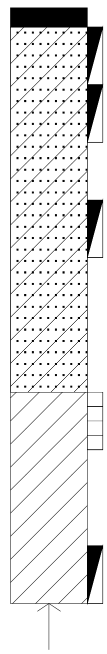
Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/4/2019

Elevation: 52 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5.5 inches AB = None			
	3/6 4/6 5/6	SC	CLAYEY SAND; loose, damp, fine to coarse grained, brown		9	
50	6/6 28/6 50/5		At 2.5 feet, gravel content increase, very dense conditions		>50	
5	8/6 7/6 7/6		interbedded Silty SAND layers, moist, light brown color, some fine gravel		14	
45						
10	3/6 10/6 7/6	CL	SANDY LEAN CLAY; stiff, moist, low plastic, fine sand, brown	DD= 102.4 pcf -200= 68% +4 = 0% LL = 35 PI = 15	17	22
40						
15	9/6 19/6 9/6		grading to Clayey SAND with gravel and cobbles, auger refusal at 14 feet		28	
			Auger refusal in cobble material			
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: E-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/26/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
51 / 0			AC = 6.5 inches			
50 / 1			AB = 6 inches			
	3/6 5/6 5/6	SC	CLAYEY SAND; loose, moist, low plastic, light brown		10	17
	20/6 50/4		at 4 feet becoming very dense	DD= 89.9 pcf	>50	16
46.5 / 4.5			Auger and Sample refusal			

Elevation scale: 50, 45, 40, 35, 30, 25

Depth scale: 0, 5, 10, 15, 20, 25

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: E-5B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: VB

Drilled By: JC

Date: 2/26/2019

Drill Type: CME-75

Elevation: 51 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
51 / 0			AC = 6.5 inches			
50 / 1			AB = 6 inches			
		SC	CLAYEY SAND; loose, moist, low plastic, light brown at 2 feet, drilling resistance difficult			
			Auger Refusal on Cobble material at 2 feet			

Depth scale: 0, 5, 10, 15, 20, 25; Elevation scale: 50, 45, 40, 35, 30, 25

Notes: *boring located 5 feet east of E-5*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: E-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/26/2019

Elevation: 50 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0			AC = 7.5 inches AB = 3 inches			
	5/6 8/6 9/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, light brown to brown, iron oxide staining	RV = 63 -200= 22% +4 = 0% LL = NP DD = 96.6 pcf	17	8
	9/6 31/6 30/6		at 4 feet, very dense conditions, cobbles in cuttings		61	10
45 / 5	7/6 6/6 8/6		at 5 feet, less gravel and cobbles, 2 inch thick clay seam		14	3
40 / 10	3/6 7/6 9/6	CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, gray-brown, iron oxide staining		16	16
35 / 15	10/6 15/6 19/6			No Recovery	34	
			Drilling refusal at 15.5 feet BSG on cobble material			
30 / 20						
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: E-7.3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9/17/2019

Drill Type: CME-75

Elevation: 49 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches AB = 5 inches			
	4/6 2/6 2/6	CL	SANDY LEAN CLAY; soft, moist, low to moderate plasticity, black, trace fine gravel		4	14
45	2/6 3/6 3/6		At 3 feet, medium stiff		6	15
5	4/6 6/6 8/6		at 5 feet, color change to dark brown, trace of coarse gravel	DD = 112.4 pcf	14	15
40 / 10	2/6 3/6 3/6		increase in sand fraction		6	14
35 / 15	25/6 6/6 11/6		coarse gravel and cobble materials present		17	
			Auger Refusal at 14.0 feet, sampler exteded to bottom of boring at 15½ feet.			
30 / 20						
25 / 25						

Notes:

Figure Number

Test Boring: E-7.3B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9/17/2019

Drill Type: CME-75

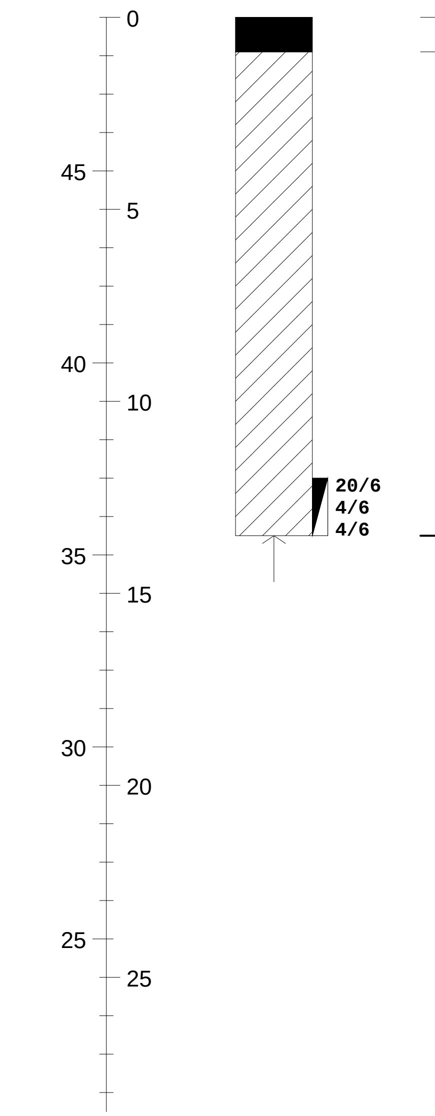
Elevation: 49 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches			
			AB = 5 inches			
		CL	SANDY LEAN CLAY; soft to medium stiff, moist, low to moderate plasticity, black, trace fine gravel			
12	20/6 4/6 4/6		cobble and dense gravel encountered		8	
			Auger Refusal at 12.0 feet, sampler exteded to bottom of boring at 13½ feet.			



Notes: *Moved boring 3 feet east of E-7.3*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: E-7.6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/17/2019

Elevation: 48 feet AMSL

Depth to Groundwater First Encountered During Drilling: 30

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 3 inches AB = 5½ inches			
	4/6 4/6 3/6	SC	FILL; CLAYEY SAND; loose, moist, fine sand to coarse gravel, intebedded Sandy Lean CLay and Silty Sand layers, brown and dark brown		7	8
45	3/6 4/6 5/6	CL	NATIVE; SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity, dark brown, trace fine gravel	DD = 106.9 pcf	9	19
5	2/6 2/6 2/6		at 5 feet, soft with increase in sand fraction		4	18
40						
10	2/6 3/6 3/6		at 10 feet, medium stiff, color is black		6	15
35						
15	6/6 6/6 5/6			no recovery	11	
30						
20	3/6 5/6 7/6		color is gray-brown, low plastic		12	18
25						
25	4/6 8/6 10/6	ML	SANDY SILT; very stiff, moist, non-plastic, dark brown		18	18
20						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: E-7.6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9/17/2019

Drill Type: CME-75

Elevation: 48 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: 30

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	7/6 15/6 23/6	CL	SANDY LEAN CLAY; hard, wet, low plasticity, brown	low recovery	38	26
15						
35	11/6 50/2	SM	SILTY SAND, very dense, wet, fine to medium grained, brown		>50	23
			Auger refusal on cobble material, bottom of boring at 37 feet BSG			
10						
40						
5						
45						
0						
50						
-5						
55						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: E-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9/17/2019

Drill Type: CME-75

Elevation: 47 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 3½ inches			
	7/6 5/6 5/6	SC	AB = 6 inches	Sand = 62.9% -#200 = 37.1% c = 200 PSF ø = 32° LL = 27 PI = 12	10	10
45			FILL; CLAYEY SAND, loose, fine to medium grained, interbedded Silty Sand and Clayey Sand layers, brown to dark brown, trace fine gravel.			
5			Auger Refusal on cobble material			

Elevation	Depth
	0
45	
	5
40	
	10
35	
	15
30	
	20
25	
	25
20	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: E-8B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9/17/2019

Drill Type: CME-75

Elevation: 47 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 3½ inches			
			AB = 6 inches			
45		SC	FILL; CLAYEY SAND, loose, fine to medium grained, interbedded Silty Sand and Clayey Sand layers, brown to dark brown, trace fine gravel			
5						
40			Auger refusal on large cobble, Bottom of Boring			
10						
35						
15						
30						
20						
25						
25						
20						

Notes: *boring moved 3 feet east of E-8*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: F-1

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Logged By: VB

Drilled By: JC

Date: 2/26/2019

Drill Type: CME-75

Elevation: 55 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 / 0			AC = 6½ inches			
			AB = 5 inches			
	7/6 9/6 5/6	CL	SANDY LEAN CLAY; stiff, moist, moderate plasticity; brown		14	
50 / 5			Auger refusal on dense gravel/ cobbles			
45 / 10						
40 / 15						
35 / 20						
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: F-1A

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

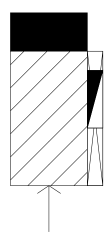
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/26/2019

Elevation: 55 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 — 0			AC = 6½ inches AB = 5 inches			
		CL	SANDY LEAN CLAY; stiff, moist, moderate plasticity, brown, some gravel	EI = 77	12	16
50 — 5			Auger Refusal on dense gravels/ cobbles			
45 — 10						
40 — 15						
35 — 20						
30 — 25						

Notes: *5 feet north of F-1*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: F-2

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/26/2019

Elevation: 54 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches			
			AB = 4 inches			
	12/6 18/6 10/6	SC	CLAYEY SAND; medium dense, moist, fine to medium grained, olive, with iron oxide staining and trace of subangular gravel		28	14
50 / 5	13/6 50/4			No Recovery	>50	
	13/6 14/6 20/6		Dense, light brown to brown, trace gravel		34	8
			auger refusal at 7 foot depth			
45 / 10			Bottom of Boring due to auger refusal at 7 feet on cobbles			
40 / 15						
35 / 20						
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: F-3

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/7/2019

Elevation: 53 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches AB = 2 inches			
50	12/6 13/6 9/6 7/6 11/6 13/6	SC	CLAYEY SAND; medium dense, damp, fine to coarse grained, light brown, some fine gravel at 2 feet moisture increase		22 24	
5 45	6/6 10/6 6/6	CL	SANDY LEAN CLAY with Gravel; medium stiff, damp, low to medium plasticity, brown, fine to coarse gravel present		16	
10	5/6 50/4		sand and gravel content increasing		>50	
			Auger and Sampler refusal on cobbles			
40 15 35 20 30 25 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: F-4

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/7/2019

Elevation: 53 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5-1/4 inches AB = 2 inches			
	14/6 16/6 23/6	SC	CLAYEY SAND with Gravel; dense, slightly damp, fine sand to coarse grained gravel, brown to yellow brown , sub-angular 1½ inch gravel		39	
50	19/6 20/6 26/6				46	
5	11/6 15/6 43/6		at 3 feet, 6 inch thick cemented layer	No Recovery	58	
45						
10	7/6 5/6 11/6		soils are medium dense, color is light brown		16	
40	11/6 14/6 10/6		Auger refusal on cobble at 13 feet, sampler extended below encountered increased coarse gravel		24	
15			Bottom of Boring due to Auger Refusal at 13 feet BSG			
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: F-5

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

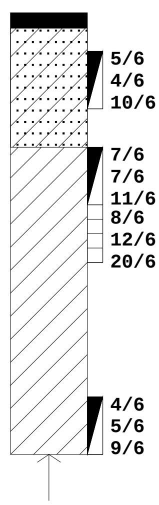
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/26/2019

Elevation: 52 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC = 5 inches			
		AB = None			
	SC	CLAYEY SAND; medium dense, moist, fine to coarse grained, gray with some gravel		14	13
	CL	SANDY LEAN CLAY; very stiff, moist, gray, with some gravel		18	13
5			DD= 111.7 pcf	32	14
10		Cobbles prevent drilling below 10 feet		14	15
		Bottom of Boring due to auger refusal on cobbles at 10 feet			

Elevation scale: 50, 45, 40, 35, 30, 25; Depth scale: 0, 5, 10, 15, 20, 25

Notes:

Figure Number

Test Boring: F-6

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/1/2019

Drill Type: CME-75

Elevation: 51 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: 30 feet

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches AB = 3 inches			
50	2/6 4/6 5/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, gray-brown some iron oxide staining		9	
	35/6 22/6 19/6		at 3 feet, very stiff, color is brown to black		42	
5 / 45	12/6 10/6 10/6			No Recovery	20	
10 / 40	3/6 7/6 7/6		stiff, color is brown, 2 inch thick sandy silt seam		14	
15 / 35	9/6 25/6 20/6	SM	SILTY SAND; dense, moist, fine to medium grained, light brown to brown, some coarse sand, weakley cemented		45	
20 / 30	3/6 7/6 9/6	CL	SANDY LEAN CLAY; very stiff, moist, low to moderate plasticity, light gray-brown		16	
25 / 25	6/6 10/6 12/6		color is dark brown		22	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: F-6

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Drilled By: JC

Logged By: JC

Date: 3/1/2019

Drill Type: CME-75

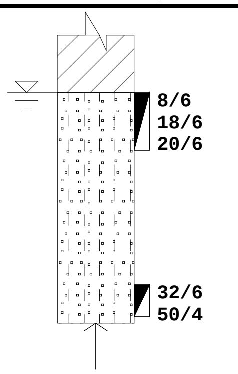
Elevation: 51 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: 30 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30 / 20	8/6 18/6 20/6	SP-SM	POORLY GRADED SAND with Silt; dense, wet, fine to medium grained, brown		38	
35 / 15	32/6 50/4		Very dense		>50	
			Drilling and Sampler refusal			
40 / 10						
45 / 5						
50 / 0						
55 / -5						



Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.
Since 1898

Test Boring: F-7A

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/28/2019

Elevation: 48 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches			
	4/6 11/6 10/6		AB = 5 inches		21	
45	10/6 11/6 14/6	CL	SANDY LEAN CLAY; very stiff, moist, low to moderate plasticity, dark brown, trace of gravel at 2.5 feet, color change to brown-black		25	
5	4/6 9/6 11/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown		20	
40						
10	3/6 7/6 8/6		increse in silt content, slight increase in moisture		15	
			Bottom of Boring			
35						
15						
30						
20						
25						
25						
20						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: F-7B

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

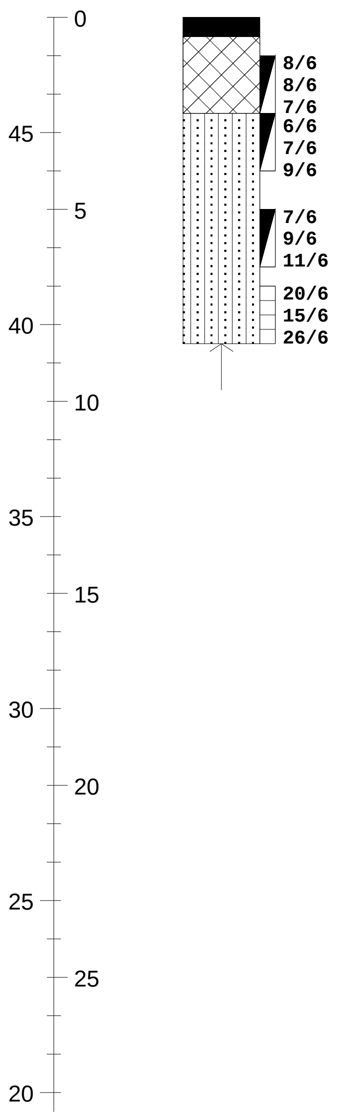
Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/19/2019

Elevation: 48 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
	AC = 5½ inches			
	AB = 2 inhces			
CL	FILL; SANDY LEAN CLAY; stiff, moist, non to moderate plasticity, interbedded Lean Clay and Sandy Silt layers, brown and red-brown, fine to corase angular gravel (broken rock fragments),		15	8
SM	NATIVE; SILTY SAND; medium dense, moist, fine to medium grained, brown		16	10
			20	
		no recovery	40	
	Auger refusal at 7 feet on cobble materials, sampler extended to bottom of boring at 8½ feet BSG			

Notes:

Figure Number

Test Boring: F-7.3

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 9/19/2019

Drill Type: CME-75

Elevation: 47 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 6½ inches			
45	8/6 5/6 7/6	SC	AB = 2 inhces 4 to 5 inch cobble under AB CLAYEY SAND; medium dense, moist, fine to medium grained, dark brown		12	10
	16/6 26/6 21/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, trace of clay clumps	DD = 101.5 pcf	47	8
5	9/6 7/6 7/6	ML	SANDY SILT; stiff, moist, non-plastic, dark brown with iron oxide staining		14	22
40						
10	6/6 7/6 8/6		at 10 feet decrease in sand fraction		15	17
35						
15	6/6 6/6 5/6		color is gray, grading to Sandy Lean Clay		11	
30		CL	SANDY LEAN CLAY, stiff, moist, low to moderate plasticity, gray			
20	4/6 6/6 8/6				14	
25						
25	9/6 13/6 15/6	ML	SANDY SILT; very stiff, moist, non-plastic, dark brown		28	
20			Bottom of Boring			

Notes:

Figure Number

Test Boring: F-8A

Project: Home Depot Store - Mission Valley - San Diego, Ca
Project Number: D050R0.01
Drilled By: JC
Drill Type: CME-75
Auger Type: 6-5/8 inch hollow stem
Hammer Type: 140 pound auto trip
Logged By: JC
Date: 2/28/2019
Elevation: 46 feet AMSL
Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
46 / 0			AC = 7 inches			
45	3/6 6/6 9/6	ML	AB = 2 inches SANDY SILT; stiff, moist, non-plastic, dark brown		15	
	7/6 8/6 9/6		at 2.5 feet, very stiff, slight increase in sand		17	
41 / 5	7/6 9/6 10/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown		19	
36 / 10	3/6 5/6 8/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, dark brown		13	
			Bottom of Boring			

Elevation scale: 45, 40, 35, 30, 25, 20; Depth scale: 0, 5, 10, 15, 20, 25

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: F-8B

Project: Home Depot Store - Mission Valley - San Diego, Ca

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 9/19/2019

Drill Type: CME-75

Elevation: 46 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
46 / 0			AC = 5 inches			
			AB = 2 inches			
45 / 1	2/6 2/6 2/6	CL	SANDY LEAN CLAY; soft, moist, low to moderate plasticity, dark brown, trace of fine gravel		4	15
	3/6 3/6 4/6				7	17
41 / 5	3/6 3/6 4/6			DD = 105.8 pcf	7	14
	6/6 6/6 9/6				15	
36 / 10			Auger refusal at 8 feet, sampler penetration to bottom of boring at 9½ feet			

Elevation scale: 45, 40, 35, 30, 25, 20; Depth scale: 0, 5, 10, 15, 20, 25

Notes: *Boring was drilled about 20 feet south of F-8A completed in February 2019*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: G-1

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

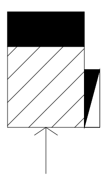
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/26/2019

Elevation: 55 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 — 0	 3/6 10/6 37/6	CL	AC = 6 inches AB = 5 inches SANDY LEAN CLAY w/ gravel; stiff, moderate plasticity, brown at 2.5 feet; grading to dense gravel Auger Refusal on dense gravel and cobbles		47	13
50 — 5						
45 — 10						
40 — 15						
35 — 20						
30 — 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: G-2

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Logged By: JC

Date: 3/7/2019

Drill Type: CME-75

Elevation: 54 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	7/6 6/6 8/6	CL	AC = 5-3/4 inches AB = 1 inch SANDY LEAN CLAY; stiff, damp, low ro moderate plasticity, light brown to yellow brown, trace gravel		14	
50 / 5	8/6 12/6 14/6	SC	CLAYEY SAND with Gravel; medium dense, damp, fine grained sand and coarse gravel, light brown	Low Recovery	26	
	50/5				>50	
			at 7 feet, grading to Clayey Gravel with Cobbles			
45 / 10			Auger Refusal on cobbles			
40 / 15						
35 / 20						
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: G-3

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/5/2019

Drill Type: CME-75

Elevation: 53 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches			
			AB = None			
50	6/6 7/6 13/6 50/3	CL	SANDY LEAN CLAY; very stiff damp, low plastic, brown to light brown, some coarse subangular gravel at 2 feet, less plastic, with very dense Silty Sand with gravel layer		20 >50	
5	10/6 10/6 6/6		at 5 feet color is light brown		16	
45						
10	19/6 37/6 50/3		hard with coarse gravel (2 inch)	DD= 114.1 pcf	>50	13
40			Auger refusal on gravel/cobbles			
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: G-4

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Logged By: JC

Date: 3/5/2019

Drill Type: CME-75

Elevation: 52 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 6½ inches AB = None			
50	9/6 7/6 9/6	CL	SANDY LEAN CLAY; very stiff, damp, low plasticity, light brown		16	
	10/6 13/6 12/6				25	
5	5/6 7/6 17/6		some fine gravel present	DD= 109.5 pcf	24	18
45						
			Auger refusal on gravel/cobble			
10						
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: G-5

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

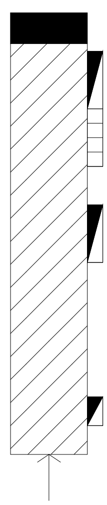
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/26/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
51 / 0			AC = 10 inches			
50 / 1	5/6 8/6 10/6	CL	AB = None SANDY LEAN CLAY; very stiff, moist, moderate plasticity, gray		18	15
	12/6 18/6 17/6			DD= 111.5 pcf -200= 52% +4= 0% LL= 36 PI= 13	35	13
46 / 5	3/6 7/6 9/6		low plasicity		16	13
41 / 10	24/6 50/3		gravel fraction increasing		>50	8
			Auger Refusal			

Depth scale (feet): 0, 5, 10, 15, 20, 25. Elevation scale: 50, 45, 40, 35, 30, 25.

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: G-6

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

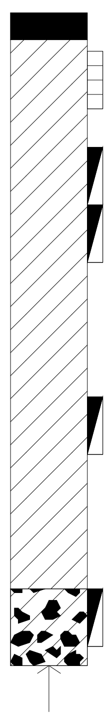
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/27/2019

Elevation: 50 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 — 0			AC = 9 inches AB = None			
	3/6 6/6 7/6	CL	SANDY LEAN CLAY; stiff, moist, moderately plastic, gray	DD= 104.0 pcf -200= 52% LL = 36 PI = 13	13	18
	5/6 6/6 7/6				13	11
45 — 5	6/6 7/6 9/6				16	20
40 — 10	16/6 17/6 7/6				24	15
35 — 15	9/6 10/6 13/6	GC	CLAYEY GRAVEL; medium dense, moist, medium to coarse grained, subrounded		23	8
			Auger refusal on dense gravel/ cobble			
30 — 20						
25 — 25						

Notes:

Figure Number

![MOORE TWINING ASSOCIATES, INC. Since 1898]

Test Boring: G-7

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

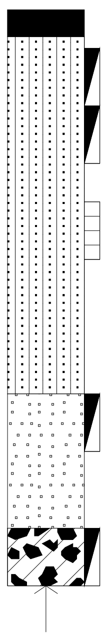
Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/19/2019

Elevation: 49 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 6 inches			
	6/6 8/6 12/6	SM	AB = 2½ inches SILTY SAND; medium dense, moist, fine to medium grained, brown		20	5
45	11/6 15/6 20/6		at 2½ feet, dense		35	
5	14/6 20/6 31/6		at 5 feet, decrease in silt fraction	DD= 107.6 psf	51	5
40 / 10	10/6 15/6 17/6	SP	POORLY GRADED SAND; dense, moist, fine to coarse, brown		32	
35	4/6 5/6 7/6	GC	SANDY LEAN CLAY with Gravel and Cobble; stiff, moist, low to moderate plasticity, brown with oxoide staining		12	
15			Auger refusal at 13½ feet, sampler extended to bottom of boring at 15 feet BSG.			
30 / 20						
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: G-7.3

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9/19/2019

Drill Type: CME-75

Elevation: 48 feet

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5½ inches			
	7/6 7/6 7/6	SM	AB = 2 inches		14	8
			SITLY SAND; medium dense, moist, fine to medium grained, brown			
45	7/6 5/6 4/6	CL	SANDY LEAN CLAY; stiff, moist, low to moderate plasticity, brown		9	13
5 / 40	35/6 20/6 21/6	SM	SILTY SAND; dense, moist, fine to medium grained, brown	DD = 98.3 pcf	41	5
10 / 35	4/6 8/6 14/6	ML	SANDY SILT; very stiff, moist, non-plastic, dark brown		22	14
15 / 30	8/6 5/6 8/6	CL	SANDY LEAN CLAY; stiff, moist, low to moderate plasticity, brown		13	
20 / 25	3/6 5/6 7/6		decrease in sand fraction		12	
			Auger refusal on cobble at 21 feet, sampler extended to 21½ feet			
25 / 20						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: G-7.6

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9/20/2019

Drill Type: CME-75

Elevation: 47 feet

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5-1/4 inches AB = 2 inches			
45	8/6 5/6 8/6	ML	SANDY LEAN CLAY; stiff, moist, low plasticity, brown	Bulk Sample Sand = 31.6% -#200 = 68.4% LL = 30 PI = 12 Remold Shear ø = 19° c = 330 PSF DD = 107.6 pcf	13	
	8/6 10/6 12/6				22	18
5	7/6 17/6 12/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, trace gravel		38	8
40						
10	4/6 6/6 8/6	ML	SANDY SILT; stiff, moist, slight plasticity, brown,		14	23
35						
15	7/6 8/6 9/6				17	
30			cobbles encountered, hard drilling			
20	9/6 4/6 7/6	CL	SANDY LEAN CLAY with gravel and cobble, stiff, moist, low to medium plasticity, brown		11	
25			Auger refusal at 19 feet, sampler extended to bottom of boring at 20½ feet			
25						
20						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: G-8

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/20/2019

Elevation: 46 feet

Depth to Groundwater First Encountered During Drilling: 25

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5 inches			
45	3/6 4/6 4/6	ML	AB = 2 inches SANDY SILT; medium stiff, moist, slight plasticty, dark brown with iron oxide staining		8	21
	8/6 8/6 13/6			No Recovery	21	
5 / 40	8/6 7/6 6/6	CL	SANDY LEAN CLAY; stiff, moist, low to moderate plasticity, dark brown		13	
10 / 35	2/6 5/6 6/6		plasticty is low		11	27
15 / 30	4/6 6/6 9/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, 1 inch thick sandy lean clay lense		15	
20 / 25	5/6 7/6 9/6	CL	SANDY LEAN CLAY; very stiff, moist, low to moderate plasticity, brown at 22 feet, hard drilling in cobbles		16	
25 / 20	8/6 11/6 10/6	SM	SILTY SAND, medium dense, wet, fine to medium grained, dark brown		21	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: G-8

Project: Home Depot Store- Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/20/2019

Elevation: 46 feet

Depth to Groundwater First Encountered During Drilling: 25

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
15 / 30	10/6 24/6 32/6	GP-GM	POORLY GRADED GRAVEL with Sand and Silt; very dense, moist, wet, fine sand to medium gravel, dark brown		56	
			Auger refusal on dense cobble/ gravel at 32 feet			

Depth scale (feet): 30, 35, 40, 45, 50, 55

Elevation scale: 15, 10, 5, 0, -5, -10

Notes:

Figure Number

Test Boring: H-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/5/2019

Elevation: 55.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 / 0	2/6 3/6 4/6	CL	AC = 7 inches AB = None SANDY LEAN CLAY; medium stiff, moist, low plastic, some gravel		7	
50 / 5	50/5	GC	CLAYEY GRAVEL; very dense, moist, medium to coarse grained, subrounded		>50	
	3/6 5/6 23/6	CL	SANDY LEAN CLAY; very stiff, moist, low to moderate plasticity, dark brown, trace angular gravel		28	
45 / 10	23/6 32/6 50/1	SC	CLAYEY SAND with Gravel; very dense, moist, medium sand to coarse gravel, dark brown	Low Recovery DD= 90.2 pcf	>50	2
40 / 15	28/6 20/6 17/6			No Recovery	37	
35 / 20	15/6 29/6 32/6		less fine grained material, Grading to Poorly Graded Sand with clay and gravel		61	
			Auger refusal on very dense gravel/cobble			
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: H-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/7/2019

Drill Type: CME-75

Elevation: 54 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
	5/6 8/6 20/6	CL	SANDY LEAN CLAY; very stiff, damp, low to moderate plasticity, brown, with trace of fine to coarse gravel		28	
50 / 5	30/6 50/5	SC	CLAYEY SAND with Gravel; very dense, damp, fine sand to coarse grained gravel, brown At 5 feet difficult drilling At 7 feet more fine gravel		>50	
	50/1					
45 / 10	11/6 11/6 13/6		Grading to SANDY LEAN CLAY, Auger refusal at 9 feet on cobble		24	
			Bottom of Boring due to auger refusal at 9 feet.			
40 / 15						
35 / 20						
30 / 25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: H-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 2/27/2019

Drill Type: CME-75

Elevation: 53 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches AB = 4½ inches			
	14/6 11/6 6/6	CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, brown, trace gravel		17	10
50	5/6 15/6 19/6		at 2.5 feet, hard, iron oxide staining		34	14
5	12/6 35/6 25/6		at 4 feet, increase in sand, decrease in plasticity	DD= 114.1 pcf	60	10
45						
10	6/6 7/6 10/6		very stiff, color is gray-brown		17	15
40						
15	5/6 8/6 8/6		low to moderate plasticity		16	17
35						
20	10/6 7/6 7/6		iron oxide staining		14	14
30						
25	3/6 4/6 6/6		less sand, color is dark brown-gray			24
			Bottom of Boring			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: H-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/5/2019

Elevation: 52 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches			
			AB = 2 inches			
	6/6 6/6 27/6	CL	SANDY LEAN CLAY; stiff, moist, low plastic, brown, coarse gravel in layer		33	
50	24/6 31/6 20/6	SC	CLAYEY SAND with Gravel; very dense, moist, medium sand to coarse gravel, brown		51	
5			Auger Refusal on cobbles (three locations)			
45						
10						
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: H-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

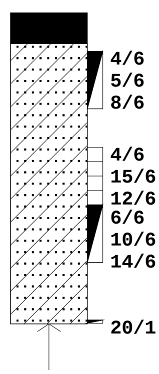
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/27/2019

Elevation: 51.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 9 inches			
50	4/6, 5/6, 8/6	SC	AB = None	EI = 81	13	13
	4/6, 15/6, 12/6		CLAYEY SAND; medium dense, moist, fine to medium grained, brown	DD= 110.5 pcf	27	13
5 / 45	6/6, 10/6, 14/6		At 5 feet increase in clay fraction, some gravel			
	20/1		Auger and Sampler refusal			

Elevation/depth scale: depth 0, 5, 10, 15, 20, 25; elevation 50, 45, 40, 35, 30, 25

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: H-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: VB

Drilled By: JC

Date: 2/27/2019

Drill Type: CME-75

Elevation: 50.5 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0			AC = 8 inches AB = None			
	3/6 4/6 6/6	CL	SANDY LEAN CLAY; medium stiff, moist, moderate plasticity, gray	LL = 36 PI = 12	10	15
	6/6 11/6 15/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, light gray, with clay lumps		26	
45 / 5	8/6 14/6 13/6			DD= 103.7 pcf -200 = 37.8	27	12
		CL	SANDY LEAN CLAY: very stiff, moist, moderate plasticity, gray			
40 / 10	7/6 12/6 17/6				29	10
35 / 15	4/6 8/6 50/1		gravel at 16 feet		>58	15
			Auger and sampler refusal on gravel/cobble			
30 / 20						
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: H-7A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/27/2019

Elevation: 49 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches			
			AB = 5 inches			
	6/6 9/6 6/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, light brown		15	8
45	4/6 3/6 4/6		At 3.5 feet becoming loose		7	12
5	4/6 4/6 7/6		At 6 feet becoming medium dense		11	13
40						
10	8/6 8/6 9/6		At 11 feet, grading to Sandy Lean Clay		17	20
			Bottom of Boring			
35 / 15						
30 / 20						
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: H-7B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/19/2019

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5.5 inches			
	8/6 9/6 10/6	SM	AB = 2.5 inches SILTY SAND; medium dense, moist, fine to medium grained, brown		19	6
	8/6 22/6 33/6		at 3 feet, dense with decrease in silt fraction	DD = 102.4 pcf	55	8
5	12/6 13/6 18/6				31	5
10	50/5	CL	Soils grading to SANDY LEAN CLAY low plasticity, dark-brown, iron oxide staining		>50	24
			Auger and Sampler refusal at 10.5 feet on cobble.			
15						
20						
25						

Notes: *Boring drilled 20 feet west of boring H-7A completed in February 2019*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: H-7.3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9/19/2019

Drill Type: CME-75

Elevation: N/A

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5.5 inches			
	8/6 10/6 8/6	SM	AB = 2.5 inches SILTY SAND; medium dense, moist, fine to medium grained, brown		18	4
	12/6 24/6 26/6			DD = 105.2 pcf	50	4
5	9/6 11/6 13/6				24	6
10	4/6 7/6 11/6	ML	SANDY SILT; very stiff, moist, non-plastic, dark-brown		18	16
15	7/6 6/6 5/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, brown		11	
20	3/6 6/6 9/6		Low to medium, trace broken cobble fragments in sampler		15	
25	4/6 8/6 12/6		Very stiff, increase in sand fraction, dark-brown, iron oxide staining		20	
			Bottom of boring at 26.5 feet BSG			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: H-7.6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/19/2019

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5.5 inches			
	7/6 9/6 6/6	ML	AB = 2.5 inches		15	11
	6/6 7/6 13/6		SANDY SILT; stiff, moist, non-plastic, brown Iron oxide staining		20	12
5	7/6 11/6 12/6		Auger refusal at 4 feet, sampler extended to 5.5 feet to bottom of boring		23	13
			Bottom of boring at 5.5			
10						
15						
20						
25						

Notes: *This log represents the second boring attempt at this location. The intial attempt encountered auger refusal in cobbles at 0.6 feet BSG just below the pavement section*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: H-8A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/27/2019

Elevation: 47.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 8 inches			
			AB = 3 inches			
45	5/6 5/6 7/6	CL	SANDY LEAN CLAY; moist, stiff, moderate plasticity, brown	RV = 19 -200 = 55% +4 = 0%	12	17
	4/6 6/6 10/6				16	23
5	9/6 10/6 11/6				21	13
40						
10	7/6 6/6 5/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown		11	16
35			Bottom of Boring			
15						
30						
20						
25						
25						
20						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: H-8B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/19/2019

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	10/6 7/6 7/6	AC	AC = 5.0 inches		14	23
		SM	AB = 2.5 inches			
			SILTY SAND; medium dense, moist, fine to medium grained, dark-brown, 3 inch thick seam of silt			
5			Auger refusal 2.0 feet			
10						
15						
20						
25						

Notes: *Boring drilled about 20 feet north of B-8A completed in February 2019*

Figure Number

Test Boring: H-8C

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9/19/2019

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5.0 inches			
		SM	AB = 2.5 inches			
	5/6 7/6 12/6		SILTY SAND; moist, fine to medium grained, brown Medium dense	DD = 105.3 pcf	19	9
5	3/6 4/6 7/6		Increase in silt fraction		11	12
10	7/6 11/6 12/6	ML	Auger refusal at 9.0 feet, sampler extended to 11.5 feet, at 9 feet grading to SANDY SILT; very stiff, moist, slight plasticity, brown		23	27
			Bottom of boring at 11.5 feet BSG			
15						
20						
25						

Notes: *second attempt to drill boring about 5 feet west of H-8B*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: I-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Logged By: JC

Date: 3/5/2019

Drill Type: CME-75

Elevation: 56 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
56 / 0			AC = 6-1/4 inch			
	6/6 12/6 12/6	SC	AB = none		24	9
			CLAYEY SAND; medium dense, damp, interbedded non-plastic zones (SM), fine to coarse grained, light brown, 2 inch subrounded gravel			
51 / 5	20/6 44/6 26/6			No Recovery	70	
49 / 7	4/6 6/6 9/6	CL	SANDY LEAN CLAY with Gravel; very stiff, moist, low to moderate plasticity, dark brown to brown, with fine to coarse grained gravel		15	
46 / 10	38/6 38/6 50/6	SC	CLAYEY SAND with Gravel; very dense, damp, fine sand to coarse gravel (up to 3-1/2 inches)	DD= 104.1 pcf	88	13
44 / 12			Auger Refusal on cobbles and dense gravel			

Elevation scale: 55, 50, 45, 40, 35, 30. Depth scale: 0, 5, 10, 15, 20, 25.

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: I-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/7/2019

Drill Type: CME-75

Elevation: 55.5 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 / 0	10/6 13/6 23/6	SC	AC = 6½ inches AB = None CLAYEY SAND with Gravel; dense, damp, fine to coarse grained, light brown, some Silty Sand zones		36	
50 / 5	3/6 6/6 10/6		at 5 feet medium dense, less gravel		16	
	8/6 12/6 13/6		clay fraction increased		25	
45 / 10	19/6 13/6 23/6		interbeded with Sandy Lean Clay layers,		36	
40 / 15	25/6 22/6 23/6	SM	SILTY SAND; dense, damp, fine to coarse grained, light brown to yellow brown, trace clay and fine gravel		45	
			Auger Refusal			
35 / 20						
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: I-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 2/27/2019

Drill Type: CME-75

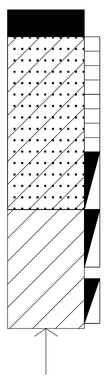
Elevation: 54.5 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches			
	13/6 15/6 17/6	SC	AB = 3½ inches	No Recovery	32	
	10/6 13/6 33/6		CLAYEY SAND; medium dense, moist, fine to medium grained, olive, trace subangular to subrounded gravel	DD= 110.9 pcf	46	12
50 / 5	17/6 19/6 27/6		at 3.7 feet, increase in sand		46	8
	4/6 7/6 7/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, dark brown		14	17
	19/6 18/6 50/4		gravel content increasing		>68	6
45 / 10			Auger and Sampler Refusal on dense gravel/cobbles			
40 / 15						
35 / 20						
30 / 25						



Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: I-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 2/27/2019

Drill Type: CME-75

Elevation: 53 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7.5 inches			
	3/6 4/6 6/6	CL	AB = 4 inches SANDY LEAN CLAY; stiff, moist, low to moderate plasticity, dark brown, trace of gravel		10	15
50	8/6 9/6 50/4		at 3 becomming hard, difficult drilling		>50	14
5			Auger Refusal			
45						
10						
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: I-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: VB

Drilled By: JC

Date: 2/27/2019

Drill Type: CME-75

Elevation: 52 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 9 inches			
	2/6 6/6 10/6	CL	AB = none		16	14
50			SANDY LEAN CLAY; very stiff, moist, moderate plasticity; brown			
	15/6 13/6 15/6			DD= 118.9 pcf	28	9
5	10/6 11/6 21/6		Grading to Clayey Gravel with Cobbles	No recovery	32	
45			Auger Refusal on Cobbles			
10						
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: I-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

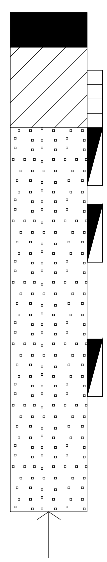
Hammer Type: 140 pound auto trip

Logged By: VB

Date: 2/27/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 3½ inches			
			AB = 7 inches			
	6/6 10/6 11/6	CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, light gray	DD= 106.2 pcf	21	14
	8/6 10/6 11/6	SP	POORLY GRADED SAND with Gravel and Cobble; medium dense, moist, medium grained sand to fine cobble		21	8
5	8/6 10/6 16/6				26	
	24/6 25/6 23/6		Very dense		48	9
			Auger Refusal on dense gravel and cobble			

Depth scale: 0, 5, 10, 15, 20, 25 feet; Elevation scale: 50, 45, 40, 35, 30, 25

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: I-7 A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9-19-19

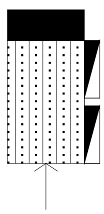
Drill Type: CME-75

Elevation: 50 feet

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 — 0		AC	AC = 6 inches			
		SM	AB = 3 inches			
	13/6 13/6 13/6		SILTY SAND; medium dense, moist, fine to medium grained, brown, trace clay		26	4
	14/6 16/6 17/6				33	6
			Auger refusal at 1.0 foot, samplers extended to 4 feet to bottom of boring.			
45 — 5			Bottom of boring at 4.0 feet			
40 — 10						
35 — 15						
30 — 20						
25 — 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: I-7 B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9-19-19

Elevation: 50 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0		AC	AC = 6.0 inches			
		SM	AB = 3.0 inches SILTY SAND; moist, fine to medium, brown			
45 / 5	13/6 29/6 42/6		Very dense	DD = 109.1 pcf	71	5
40 / 10	7/6 5/6 8/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, dark-brown, with iron oxide staining	no sample recovery	13	
35 / 15	5/6 7/6 6/6		Gray-brown		13	
30 / 20	3/6 9/6 9/6	SC	CLAYEY SAND; medium dense, moist, fine to medium grained, dark-brown, iron oxide staining		18	
25 / 25	4/6 6/6 8/6	Cl	SANDY LEAN CLAY; stiff, moist, low to medium, gray		14	
			Bottom of boring			

Notes: *Boring moved 5 feet west of location I-7A*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: I-7.3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9-19-19

Drill Type: CME-75

Elevation: 49.5 feet

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater
First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5.5 inches			
	5/6 5/6 5/6	CL	AB = 2.0 inches SANDY LEAN CLAY; stiff, moist, low to medium plasticity, dark-brown, trace fine gravel		10	26
	8/6 11/6 12/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown		23	8
45 / 5	14/6 14/6 12/6				26	
	5/6 18/6 22/6		Light-brown, trace gravel	DD = 104.2 pcf	40	6
40 / 10						
35 / 15	7/6 10/6 10/6	CL	Auger Refusal at 14 feet, sampler extended to 15.5 feet BSG, soils grading to SANDY LEAN CLAY; moist, low to medium plasticity, gray		20	
			Bottom of boring			
30 / 20						
25 / 25						

Notes: *This log represents the second boring attempt at this location. A previous boring encountered auger refusal in cobbles at 0.6 feet BSG just below the pavement section.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: I-7.6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Logged By: JC

Date: 9-19-19

Drill Type: CME-75

Elevation: 49 feet

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
49 / 0		AC	AC = 6.5 inches			
	8/6 10/6 12/6	SM	AB = 2.0. inches SILTY SAND; medium dense, moist, fine to medium grained, brown		22	3
	9/6 12/6 16/6	ML	SILT; very stiff, moist, non-plastic, dark-brown, iron oxide staining, iron oxide staining, trace clay	DD = 102.1 pcf	28	25
45 / 5	7/6 9/6 11/6		at 4.5 feet, sand fraction increase, color is brown		20	7
40 / 10	3/6 5/6 8/6		Stiff, dark-brown		13	20
35 / 15	3/6 8/6 11/6		Very stiff, increase in moisture		19	
30 / 20	8/6 9/6 10/6	CL	SANDY LEAN CLAY; very stiff, moist, low to medium plasticity, brown		19	
25 / 25	3/6 6/6 9/6		Stiff, iron oxide staining		15	
			Bottom of boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: I-8A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 2/27/2019

Elevation: 47.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 6.5 inches			
			AB = 4 inches			
	10/6 11/6 12/6	CL	FILL; SANDY LEAN CLAY; very stiff, moist, low to medium plasticity, dark brown, 2 inch thick silt seam		23	22
45	2/6 2/6 3/6		at 2.3 feet, medium stiff,		5	33
	4/6 5/6 7/6		at 3.9 feet, 1 foot thick Silty Sand Layer		12	14
5 / 40						
10	4/6 5/6 7/6	SC	NATIVE; CLAYEY SAND; medium dense, moist, fine to coarse grained, brown less plastic		12	23
35			Bottom of Boring			
15 / 30 / 20 / 25 / 25 / 20						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: I-8B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9-19-19

Elevation: 47.5 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 6 inches			
	3/6 3/6 4/6	CL	AB = 2 inches		7	27
45	2/6 2/6 3/6		SANDY LEAN CLAY; medium stiff, moist, low to medium plasticity, dark-brown		5	41
5	7/6 17/6 24/6	ML	SANDY SILT; very stiff, moist, non-plastic, brown	DD = 104.2 pcf	41	11
40						
10	4/6 6/6 9/6		Stiff		15	18
35						
15	4/6 7/6 11/6		Very stiff		19	
30						
20	6/6 5/6 8/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, gray-brown, iron oxide staining		13	
25						
25	4/6 8/6 10/6		Very stiff, dark-brown		18	
			Bottom of boring			
20						

Notes: *Boring drilled about 10 feet north of I-8B*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: J-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 3/5/2019

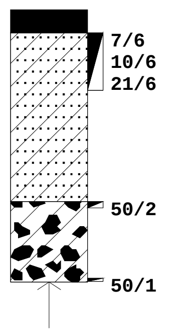
Drill Type: CME-75

Elevation: 56 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 auto trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0 / 55	7/6, 10/6, 21/6	SC	AC = 8 inches AB = None CLAYEY SAND with Gravel; dense, damp, fine grained sand up to 1½ inch diameter gravel, dark brown		31	10
5 / 50	50/2	GC	CLAYEY GRAVEL with Cobbles; very dense	No Recovery	>50	
	50/1		Auger Refusal on dense cobbles/ gravel	No Recovery	>50	

Depth scale (feet): 0, 5, 10, 15, 20, 25

Elevation scale: 55, 50, 45, 40, 35, 30

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: J-1A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 3/5/2019

Drill Type: CME-75

Elevation: 56 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 / 0	11/6 17/6 13/6	SC	AC = 8 inches AB = None CLAYEY SAND; medium dense, damp, fine grained, brown At 2 feet, Gravel and Cobble encountered Auger Refusal in Cobble/Gravel		30	10

Elevation scale: 55, 50, 45, 40, 35, 30; Depth scale: 0, 5, 10, 15, 20, 25

Notes: *3 feet west of J-1*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: J-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 2/27/2019

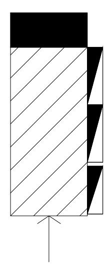
Drill Type: CME-75

Elevation: 55 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 auto trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
55 — 0			AC = 7 inches			
			AB = 3 inches			
	8/6 13/6	CL	SANDY LEAN CLAY with Gravel; very stiff, low plasticity, brown at 1.5 feet, hard drilling likely more cobble/gravel		22	12
	9/6 7/6 17/6		At 2.4 hard, color is light brown to brown		36	10
	19/6 10/6 9/6 50/3		at 4 feet, iron oxide staining Auger refusal on cobbles/gravel at 4.2 feet		>50	11
50 — 5						
			Sampler Refusal at 5.2 feet BSG			
45 — 10						
40 — 15						
35 — 20						
30 — 25						

Notes:

Figure Number

Test Boring: J-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Logged By: JC

Date: 2/27/2019

Drill Type: CME-75

Elevation: 54.5 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7.5 inches			
	7/6 34/6 50/1		AB = 4 inches		>50	4
		CL	SANDY LEAN CLAY with Gravel, hard, moist, low plasticity, dark brown			
50 / 5			Auger and sampler refusal on cobble/gravel			
45 / 10						
40 / 15						
35 / 20						
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: J-3A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 auto trip

Logged By: JC

Date: 2/27/2019

Elevation: 54.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7.5 inches			
			AB = 4 inches			
		GC	CLAYEY GRAVEL with Cobble; 7 inch diameter Cobble at 1 foot			
			Auger refusal on Cobble			
50 / 5						
45 / 10						
40 / 15						
35 / 20						
30 / 25						

Notes: *Boring drilled 5 feet north of J-3*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: J-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/6/2019

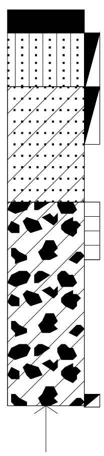
Drill Type: CME-75

Elevation: 53.5 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 auto trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches AB = None			
	33/6 10/6 17/6	SM	SILTY SAND with Gravel; medium dense, damp, fine sand to medium sub-angualr gravel, light brown		27	
50	13/6 14/6 11/6	SC	CLAYEY SAND with Gravel medium dense, damp, fine sand to medium gravel		25	
5	13/6 9/6 13/6	GC	CLAYEY GRAVEL with Sand; medium dense, moist, fine sub-rounded grains, low plastic, light brown	DD= 108.6 pcf -200 = 36% +4 = 33% LL = 38 PI = 18	22	13
45						
10	50/4		at 10 feet, less clay and sand; Cobble fraction increasing		>50	
			Auger and Sampler refusal on very dense cobbles/gravel			
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: J-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 auto trip

Logged By: VB

Date: 2/27/2019

Elevation: 53 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5 inches			
	3/6 3/6 2/6	CL	AB = 5 inches		5	15
50	45/6 45/6 25/6		SANDY LEAN CLAY; medium stiff, moist, moderate plasticity; gray At 2.5 hard; increased drilling resistance	DD= 113.1 pcf	70	14
5	50/3		Auger and Sampler refusal in gravel/cobbles		>50	10
45						
10						
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: J-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 auto trip

Logged By: VB

Date: 2/27/2019

Elevation: 52.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
52.5 / 0			AC = 3 inches AB = 4 inches			
	11/6 7/6 11/6	CL	SANDY LEAN CLAY with Gravel; stiff, moist, moderate plasticity, brown		18	11
	4/6 4/6 4/6		At 3 feet, increase in sand		8	19
5	5/6 8/6 15/6		at 5 feet, less sand, more fine gravel		23	14
	17/6 50/5	GP	POORLY GRADED GRAVEL with Sand; very dense, damp, medium grained, subangular		>50	3
			Auger and Sampler Refusal in very dense gravel/cobble			

Elevation scale: 50, 45, 40, 35, 30, 25; Depth scale: 0, 5, 10, 15, 20, 25

Notes:

Figure Number

Test Boring: J-7

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

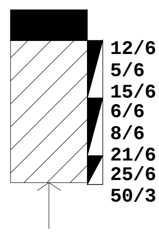
Hammer Type: 140 auto trip

Logged By: JC

Date: 2/27/2019

Elevation: 50.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0			AC = 4.5 inches			
		CL	AB = 5 inches		20	12
	12/6 5/6 15/6		SANDY LEAN CLAY; very stiff, moist, fine to medium grained, trace gravel	Minimal recovery	29	13
	6/6 8/6 21/6		at 1.5 feet, hard drilling, cobble in drill cuttings	No recovery	>50	
45 / 5	25/6 50/3		at 2.3 feet, grading to Gravel			
			Auger and Sampler Refusal on gravel/cobble			
40 / 10						
35 / 15						
30 / 20						
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: J-7.3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 9-18-19

Elevation: 50 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 — 0		AC	AC = 5.5 inches			
	7/6 8/6 9/6	CL	AB = 3.0 inches		17	10
			SANDY LEAN CLAY; very stiff, moist, low plasticity, dark-brown			
			Auger refusal at 2 feet BSG, Sample extended to Bottom of boring at 2½ feet			
45 — 5						
40 — 10						
35 — 15						
30 — 20						
25 — 25						

Notes:

Figure Number

Test Boring: J-7.3B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Logged By: JC

Date: 9-18-19

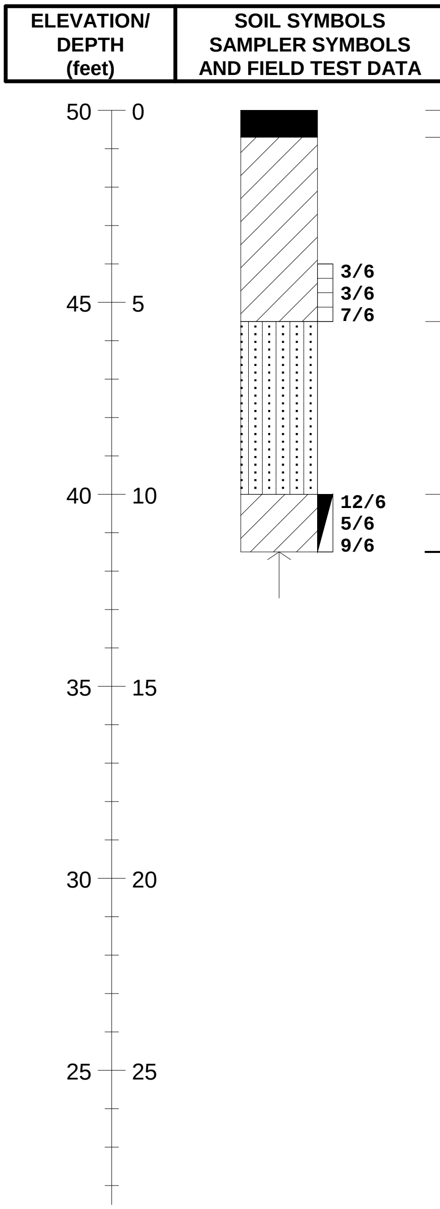
Drill Type: CME-75

Elevation: 50 feet

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
AC	AC = 5.5 inches			
CL	AB = 3.0 inches SANDY LEAN CLAY; medium stiff, moist, low to medium plasticity, dark-brown	DD = 78.6 pcf	10	
SM	SILTY SAND; medium dense, moist, fine to medium grained, brown			
CL	SANDY LEAN CLAY; stiff, moist, low plasticity, brown Auger refusal at 11.0 feet, sampler extended to 11.5 feet BSG		14	22
	Bottom of boring			

Notes: *Boring drilled 5 feet from boring J-7.3A location*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: J-7.6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 auto trip

Logged By: JC

Date: 09/18/19

Elevation: 50 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0		AC	AC = 6 inches			
	6/6 5/6 4/6	ML	AB = 3 inches SANDY SILT; stiff, moist, non-plastic, dark-brown		9	19
	5/6 4/6 4/6		Medium stiff, increase in moisture		8	28
45 / 5	4/6 8/6 12/6	CL	SANDY LEAN CLAY; stiff, moist, low to medium plasticity, dark-brown	DD = 98.1 pcf	20	28
40 / 10	3/6 5/6 7/6		Increase in sand fraction		12	25
35 / 15	4/6 5/6 9/6		SILTY SAND layer 6 inches thick at 15.5 feet, lean clay has increase sand fraction		14	
	5/6 6/6 6/6		Stiff, increase in sand fraction		12	
30 / 20						
	3/6 6/6 7/6		Low to medium plasticity		13	
25 / 25			Bottom of boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: J-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 2/27/2019

Drill Type: CME-75

Elevation: 49.5 feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 6.5 inches			
	6/6 8/6 8/6	SC	AB = 4½ inches CLAYEY SANDY; medium dense, moist, fine to medium grained, low plastic, dark brown to black; 4 inch cobble in cuttings	RV = 22 -200 = 43% LL = 42 PI = 22	16	11
	6/6 4/6 6/6	SM	SILTY SAND; loose, moist, fine to medium grained, brown, 3 inch thick clay seam		10	10
45 / 5	7/6 9/6 11/6		at 4 feet medium dense		20	5
	7/6 9/6 11/6		at 7.5 feet, hard drilling		20	13
40 / 10			Bottom of Boring			
35 / 15						
30 / 20						
25 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: J-8B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 09/18/19

Drill Type: CME-75

Elevation: 49.5 feet

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 6.0 inches			
	10/6 8/6 8/6	ML	AB = 3.5 inchs SANDY SILT; very stiff, moist, non-plastic, brown Auger refusal at 2 feet BSG, sampler extended to 2.5 feet		16	8
45 / 5			Bottom of Boring			
40 / 10						
35 / 15						
30 / 20						
25 / 25						

Notes: *Boring drilled 10 feet north of boring J-8 drilled in Feburary 2019. This boring is the second attempt at this locaiton after refusal on cobbles occurred at 0.7 feet BSG in the initial boring.*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: K-7.6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem augers

Hammer Type: 140 LB Auto Trip

Logged By: JC

Date: 09-18-19

Elevation: 53 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5 inches			
		CL	AB = 3.5 inches			
	15/6 9/6 9/6		FILL; SANDY LEAN CLAY; very stiff, moist, low to medium plasticity, brown, trace coarse gravel		18	10
50	6/6 7/6 8/6	CL	SANDY LEAN CLAY; stiff, low plasticity, trace fine gravel		15	17
	50/2		Hard drilling on gravels and cobbles	No Recovery	>50	
5			Auger refusal at 4 feet BSG, sampler extended below auger refusal			
			Bottom of boring			
45						
10						
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: K-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem augers

Hammer Type: 140 LB Auto Trip

Logged By: JC

Date: 09-18-19

Elevation: 51 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
51 / 0		AC	AC = 5.8 inches			
		GM	AB = 3.0 inches			
50 / 1	8/6 10/6 12/6	ML	FILL; 4.5" X 3" cobble and coarse gravel under aggregate base		22	8
	11/6 18/6 22/6		SANDY SILT; very stiff, moist, non-plastic, brown	DD = 89.4 pcf	40	4
45 / 6	5/6 6/6 7/6	SC	CLAYEY SAND; medium dense, moist, fine to medium grained, brown		13	10
	25/6 36/6 22/6	CL	SANDY LEAN CLAY; hard, moist, low plasticity, dark-brown	No Recovery	58	
35 / 16	4/6 6/6 8/6		Stiff, grading to non-plastic sandy silt		14	
30 / 21	4/6 5/6 6/6		increase to low to medium plasticity		11	
25 / 26	3/6 5/6 8/6		Iron oxide staining		13	
			Bottom of boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: L-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

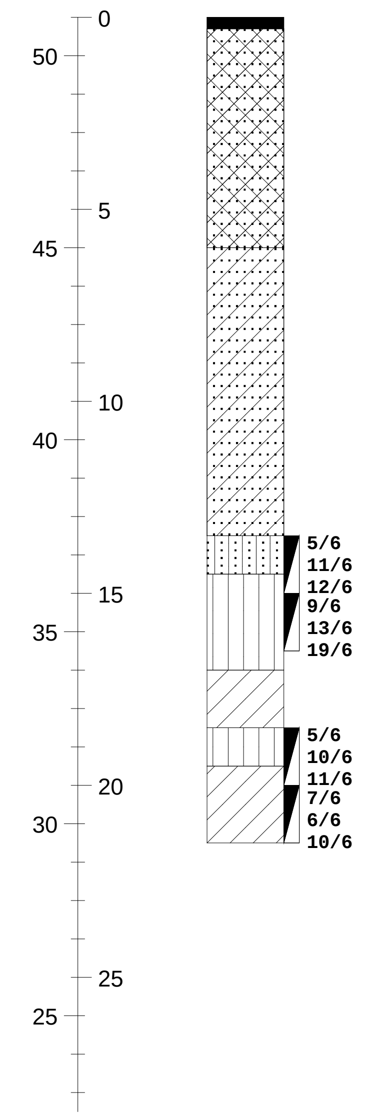
Logged By: JC

Date: 3/6/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet) — SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
SC	FILL; CLAYEY SAND with Gravel, very dense, moist, fine to coarse grained, brown at 3 feet color change to light brown			
SC	NATIVE: CLAYEY SAND, moist, fine grained, brown			
SM	SILTY SAND; medium dense, moist, fine to medium grained, brown		23	
ML	SANDY SILT, very stiff, moist, non-plastic, dark brown at 15 feet, 1 inch thick clay seam		32	
CL	SANDY LEAN CLAY with Gravel and Cobbles; low to medium plasticity, dark brown			
ML	SANDY SILT; very stiff, moist, slight plasticity, dark brown		21	
CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, brown to dark brown		16	
	Bottom of Boring			

Notes: *deeper exploration boring drilled 8 feet south and 22 feet west of M-8A*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: M-8A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/4/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
51 / 0			AC = 4½ inches AB = None			
50 / 1	5/6 13/6 31/6	SC	FILL; CLAYEY SAND with Gravel, very dense, moist, fine to coarse grained, brown		44	
2	9/6 13/6 19/6		At 2 feet, less clay fraction, fine gravel, some slightly cemented soils at 3 feet		32	
46 / 5	2/6 5/6 11/6		At 5 feet, more clay, no gravel, sand is fine grained	DD= 108.1 pcf	16	14
45 / 6		SC	At 6 feet, 3 inch thick aged asphalt section NATIVE: CLAYEY SAND; medium dense, moist, fine grained, brown			
41 / 10	5/6 7/6 10/6				17	
11.5			Bottom of Boring			

Depth scale (feet): 0, 5, 10, 15, 20, 25; Elevation scale: 50, 45, 40, 35, 30, 25

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: M-8B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JS

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem augers

Hammer Type: 140 LB Auto Trip

Logged By: JC

Date: 9-18-19

Elevation: 51 Feet

Depth to Groundwater First Encountered During Drilling: 30 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC = 5.5 inches			
50	22/6 35/6 40/5	SM	AB = 4.5 inches SILTY SAND; very dense, moist, tan-brown, with fine gravel		75	20
	50/5	SC	CLAYEY SAND; very dense, moist, fine to medium grained, reddish-brown to brown, trace fine to coarse gravel		>50	13
5	7/6 7/6 4/6			No Recovery	11	
45						
10	5/6 8/6 13/6			No Recovery	21	
40						
15	3/6 9/6 12/6	ML	SANDY SILT; very stiff, moist, non-plastic, brown		21	19
35						
20	4/6 6/6 8/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, olive-brown		14	18
30						
25	4/6 6/6 8/6		Low to medium plasticity, dark-brown		14	19
25						

Notes: * - *flowing sands may have distubed sample*

Figure Number

Test Boring: M-8B

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JS

Date: 9-18-19

Drill Type: CME-75

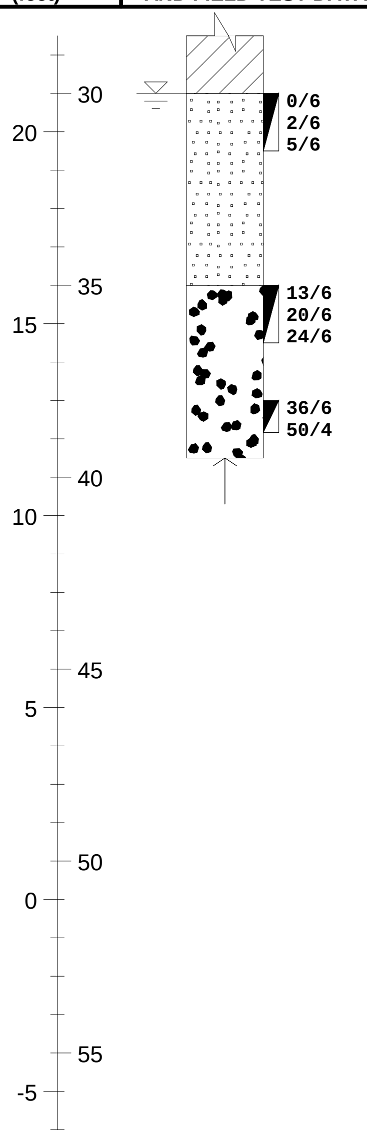
Elevation: 51 Feet

Auger Type: 6-5/8 inch hollow stem augers

Depth to Groundwater First Encountered During Drilling: 30 feet

Hammer Type: 140 LB Auto Trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	0/6 2/6 5/6	SP	POORLY GRADED SAND; loose, wet, fine to coarse grained, brown	+4 = 3.7% Sand = 92.1% -#200 = 4.2%	7*	23
35	13/6 20/6 24/6	GP	POORLY GRADED GRAVEL WITH SAND; dense, wet, fine to coarse grained, dark-brown		44	
	36/6 50/4		Auger refusal at 38 feet BSG, sampler extended to 39.5 feet BSG		>50	
			Bottom of boring			



Notes: ** - flowing sands may have distubed sample*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: M-8C

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: MG - Pacific Drilling

Drill Type: Marl Yeti M-10

Auger Type: 6-5/8 inch hollow stem augers

Hammer Type: 140 LB Auto Trip

Logged By: SWK

Date: 12-27/2019

Elevation: 51 Feet

Depth to Groundwater First Encountered During Drilling: 26 feet

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	AC= 6 inches			
50		SM	AB = 3 inches			
			SILTY SAND w gravel; medium dense, moist, fine to coarse grained, light gray tan			
5 / 45	11/6 5/6 9/6		at 5 feet, no gravel, fine sand, slight plasticty, grading to Sandy Silt		15	
		ML	SANDY SILT; stiff, moist, low non-plastic, gray-tan			
10 / 40	19/6 5/6 9/6				14	
			some scattered gravel			
15 / 35	7/6 11/6 7/6		interbedded plastic zones		18	
		CL	SANDY LEAN CLAY; stiff, moist, low plastic, fine to medium grained, gray			
20 / 30	24/6 13/6 6/6				18	
25	3/6 13/6 16/6			No rings 10% recovery	29	
25	19/6 11/6 11/6		Stiff clay, higher plasticity, less sand		22	
					37	

Notes: *Drilled 10 feet east of M-8B*

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: M-8C

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: MG - Pacific Drilling

Drill Type: Marl Yeti M-10

Auger Type: 6-5/8 inch hollow stem augers

Hammer Type: 140 LB Auto Trip

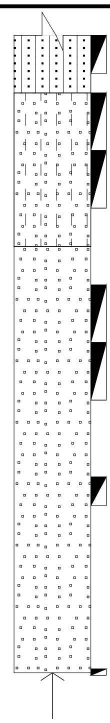
Logged By: SWK

Date: 12-27/2019

Elevation: 51 Feet

Depth to Groundwater First Encountered During Drilling: 26 feet

ELEVATION/ DEPTH (feet): depth 30 / elevation 20; 35 / 15; 40 / 10; 45 / 5; 50 / 0; 55 / -5



SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
28/6 17/6 20/6	SM	grading to silty sand			
49/6 22/6 17/6	SP-SM	SILTY SAND; dense, wet, fine to emdium grained, w/ clay lenses, gray		35	
8/6 6/6 12/6		POORLY GRADED SAND with silt and Gravel; dense to medium dense, wet, fine sand to coarse grained gravel, tan-gray		18	
4/6 5/6 5/6	SP	POORLY GRADED SAND w Gravel and Cobble; matrix is medium dense, wet, hard drilling, coarse gravel and/or cobble material mixed with sand		10	
5/6 6/6 9/6				15	
49/6 50/3				>50	
		Less gravel/cobble			
		at 43 feet, very hard drilling			
50/2		at 44 feet, near refusal 1 foot of advancement in 1/2 hour shattered rock in sampler		>50	
		Auger and sampler refusal on boulder sized material			

Notes: *Drilled 10 feet east of M-8B*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: N-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/6/2019

Drill Type: CME-75

Elevation: 54.5 Feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5-3/4 inches AB = none			
	23/6 50/4	GW-GC	WELL GRADED GRAVEL with Sand and Clay; very dense, damp, fine sand to coarse gravel, light brown		>50	
	34/6 17/6 10/6	SC	CLAYEY SAND with Gravel; very dense, damp, fine sand to coarse gravel At 5 feet, cobbles present		27	
50 / 5	50/5			No Recovery	>50	
			Auger and sampler refusal on cobble			
45 / 10						
40 / 15						
35 / 20						
30 / 25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: N-3A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Logged By: JC

Date: 3/6/2019

Drill Type: CME-75

Elevation: 54.5 Feet AMSL

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5½ inches AB = none			
	8/6, 7/6	CL			30	13
	23/6, 18/6, 44/6, 33/6	SC	SANDY LEAN CLAY with Gravel; very stiff, moist, medium plasticity, light brown, with 1 inch gravel		77	9
50 / 5	30/6, 50/3		CLAYEY SAND with Gravel; very dense, damp, fine to coarse grained, light brown At 5 feet, color change to brown	DD= 90.6 pcf	>50	7
45 / 10	2/6, 5/6, 6/6	CL	SANDY LEAN CLAY with Gravel; medium stiff, moist, medium plasticity, brown, gravel is fine grained		11	
	13/6, 9/6, 13/6		at 13 feet, stiff, coarse gravel fraction increasing		22	
40 / 15	6/6, 8/6, 11/6	SM	SILTY SAND; medium dense, moist, fine grained, light brown		19	
35 / 20	4/6, 4/6, 7/6	SC	CLAYEY SAND; medium dense, moist, fine to medium grained, brown to light brown, trace fine gravel		11	
30 / 25	7/6, 16/6, 13/6				29	
			Bottom of Boring			

Notes: *second boring was located 5 feet east of N-3*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: N-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/5/2019

Elevation: 53.5 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 7 inches AB = None			
	14/6 20/6 6/6	SC	CLAYEY SAND with Gravel; medium dense, fine grained sand to coarse gravel; light brown	Low recovery	26	
	4/6 5/6 6/6				11	
50	11		at 2 feet, fine gravel and increase in coarse grained sand			
5	10/6 12/6 22/6		at 5 feet, gravel is coarse and sub-angular	DD= 103.2 pcf	34	16
45						
10	50/0		hard drilling, likley cobble	No Recovery	>50	
			Auger refusal on cobbles			
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: N-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/5/2019

Elevation: 53 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 5-3/4 inches AB = None			
	2/6 3/6 5/6	SC	CLAYEY SAND; loose, moist, fine grained, light brown with tree roots		8	
50	3/6 5/6 6/6		at 2 feet, tree roots are finer, soils are medium dense		11	
5	5/6 15/6 50/5		Increase in corase gravel, 2 inch sub-rounded	DD= 108.6 pcf	>50	18
45						
10	10/6 11/6 18/6				29	
			Bottom of Boring			
40						
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

Test Boring: N-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/4/2019

Drill Type: CME-75

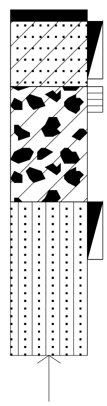
Elevation: 53 Feet AMSL

Auger Type: 6-5/8 inch hollow stem

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 pound auto trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 4 inches AB = none			
	24/6 33/6 5/6	SC	CLAYEY SAND with Gravel; dense, damp, fine grained sand to sub-angular coarse grained gravel, light brown		38	
50	20/6 50/2	GC	CLAYEY GRAVEL with Sand; very dense, moist	DD= 113.4 pcf	>50	7
5	13/6 17/6 27/6	SM	SILTY SAND with Gravel; dense, damp, fine sand to coarse gravel, light brown, interbedded with Clayey Sand layers		44	
45						
10			Auger refusal on cobbles and gravel			
40						
15						
35						
20						
30						
25						
25						



Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: N-7

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/2/2019

Elevation: 52 Feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	5/6 6/6 7/6	SC	AC = 5 inches AB = none		13	
50	7/6 16/6 7/6		CLAYEY SAND with Gravel; medium dense, damp, fine to medium grained, brown at 2 feet, no coarse gravel		23	
5	11/6 18/6 23/6		at 5 feet, color is brown mottled light brown with coarse gravel	DD= 106.4 pcf	41	9
45						
10	18/6 50/2		Grading to GRAVEL, color is reddish brown		>50	
40			Bottom of Boring			
15						
35						
20						
30						
25						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: N-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: 3/4/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0			AC = 6-3/4 inches AB = None			
	9/6 18/6 19/6	SC	FILL; CLAYEY SAND with Gravel; dense, damp, fine sand to coarse gravel, light brown, brick fragments		37	
	32/6 19/6 12/6			DD= 103.1 pcf	31	13
5	10/6 8/6 9/6	SC	NATIVE; CLAYEY SAND; medium dense, moist, fine grained, brown, trace gravel		17	
10	5/6 4/6 8/6		color is brown and light brown, trace coarse gravel, slightly cemented soils at 11 feet.		12	
			Bottom of Boring			

Elevation scale: 50, 45, 40, 35, 30, 25. Depth scale: 0, 5, 10, 15, 20, 25.

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 4, 2019

Elevation: 116 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
116 / 0	3/6 2/6 3/6	CL	FILL; SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity, brown and reddish brown, small wood debris		5	18
114 / 2	6/6 10/6 13/6		at 2 feet, stiff, light brown, increase in wood debris	DD= 107.8 pcf WD= 126.6 pcf -200= 49% ø = 18° C = 1,080 psf LL = 38 PI = 18	23	18
111 / 5	4/6 6/6 7/6	SC	FILL; CLAYEY SAND; stiff, moist, fine grained, low plastic, brown with some white calcification		13	15
106 / 10	9/6 12/6 15/6		less plastic to slight, some calcification, wood debris	DD= 91.7 pcf WD= 102.0 pcf	27	11
101 / 15	6/6 7/6 5/6	CL	NATIVE; SANDY LEAN CLAY; stiff, moist, low plasticity, brown, calcification, trace gravel		12	12
96 / 20	4/6 6/6 8/6		plasticity increase, 1 inch to ½ inch gravel		14	14
91 / 25	4/6 6/6 8/6	ML	SANDY SILT; stiff, moist, non-plastic, brown		14	16

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: S-1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 4, 2019

Elevation: 116 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
85 / 30	5/6 7/6 8/6		trace gravel and iron oxide staining		15	16
80 / 35	7/6 12/6 20/6				32	15
		SP	POORLY GRADED SAND; dense, moist, fine to medium grained, light brown			
75 / 40	20/0.5		Auger and Sampler Refusal due to suspected cobble		>50	
70 / 45						
65 / 50						
60 / 55						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

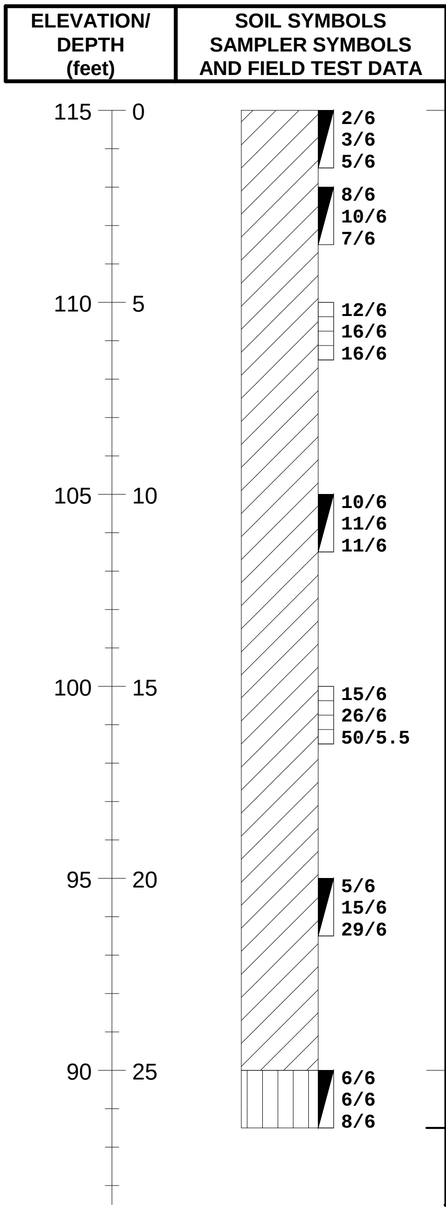
Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 4, 2019

Elevation: 115 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
CL	SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity, light brown, trace gravel, calcification		8	16
	at 2 feet, white weakly cemented calcification		17	9
	Hard drilling at 6 feet	DD= 93.9 pcf WD= 101.3 pcf ø = 36° C = 50 psf	36	8
			22	9
	Hard, color change to dark brown, low plasticity	DD= 110.5 pcf WD= 118.4 pcf	>76	7
	plasticity increase to moderate		44	12
ML	SANDY SILT; stiff, moist, non-plastic, dark brown		14	12
	Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 4, 2019

Elevation: 113 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6 3/6 4/6	CL	SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity; dark brown, trace gravel		7	16
110	6/6 8/6 7/6		Stiff at 2 feet, weakly cemented		15	8
5	9/6 10/6 12/6		Hard drilling	Low Recovery	22	14
105			Auger Refusal on Cobble			
10						
100						
15						
95						
20						
90						
25						
85						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: S-3A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 4, 2019

Elevation: 113 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		CL	SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity; dark brown, trace gravel			
110						
5	5/6 4/6 7/6		Hard drilling at 5.5 feet on a cobble		11	7
			Auger refusal on cobble at 6 feet			
			Bottom of Boring (sample extended below auger)			
105						
10						
100						
15						
95						
20						
90						
25						
85						

Notes: *Boring S-3A was drilled 3 feet southeast of S-3*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-4

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 4, 2019

Elevation: 112.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	4/6 12/6 16/6	CL	SANDY LEAN CLAY; very stiff, moist, low plasticity, reddish-brown to brown, trace gravel Hard drilling on cobbles at 1. 5 feet BSG		28	6
			Auger Refusal on Cobbles			
110						
5						
105						
10						
100						
15						
95						
20						
90						
25						
85						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-4A

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

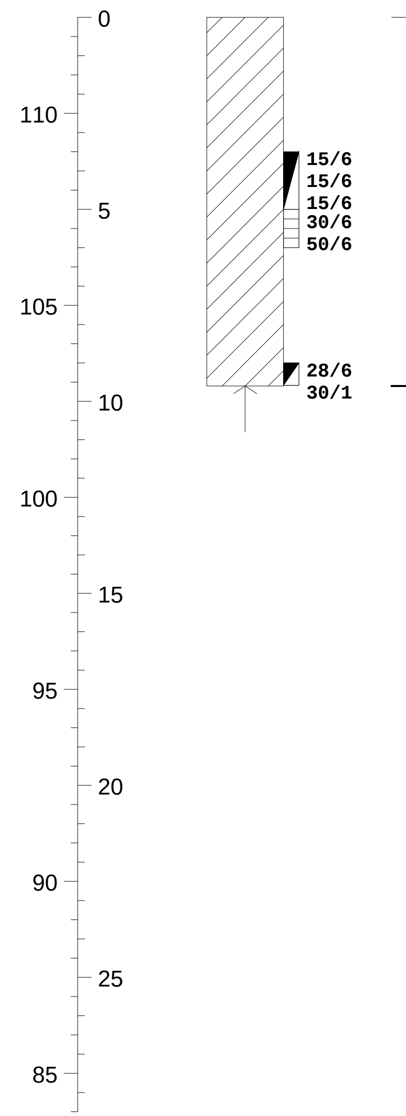
Logged By: JC

Date: March 4, 2019

Elevation: 112.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		CL	SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity; dark brown, trace gravel			
			very stiff, iron oxide staining, gravel increasing	minimal recovery	30	6
			Hard, color is brown, trace 1 to 1½ inch gravel	DD= 97.5 pcf WD= 109.2 pcf	>50	12
					>50	
			Auger and Sampler refusal on cobble			



Notes: *Boring S-4A was drilled 5 feet west of S-4*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 5, 2019

Elevation: 112 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6 3/6 2/6	CL	SANDY LEAN CLAY; medium stiff, moist, low to medium plasticity, dark brown, trace gravel		5	18
110	4/6 7/6 8/6		At 2 feet, stiff, light brown, weak cementation		15	13
5	6/6 10/6 9/6		gravel present		19	14
105						
10	11/6 13/6 10/6		color is light brown to dark brown		23	12
100						
15	9/6 15/6 20/6		Hard, color is brown, decrease in plasticity	DD= 102.4 pcf WD= 117.2 pcf	35	15
95						
20	7/6 8/6 8/6		Very stiff, low plasticity		16	13
90						
25	50/6		Hard, color is dark brown, less moisture	No Recovery	>50	
85						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.
Since 1898

Test Boring: S-5

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: Pacific Drilling

Date: March 5, 2019

Drill Type: Fastre SPT

Elevation: 112 feet AMSL

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	25/6 18/6 18/6	ML	SILT with Sand; very stiff, moist, Low plasticity, brown	DD= 102.9 pcf WD= 115.5 pcf ø = 27° c = 310 psf -200= 71% +4 = 0% LL = 37 PI = 19	36	12
80						
35	6/6 8/6 11/6		very stiff, moist, low plasticity, light brown to brown		19	15
75	38/6 50/6		Auger refusal on cobble		>50	6
			Sampler refusal			
40						
70						
45						
65						
50						
60						
55						
55						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-6

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 5, 2019

Elevation: 110.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
110 / 0	2/6 2/6 3/6	CL	SANDY LEAN CLAY; medium stiff, moist, low to moderate plasticity, dark brown, trace gravel		5	21
	5/6 6/6 7/6		at 2 feet, stiff, color is dark brown to black		13	13
105 / 5	5/6 5/6 10/6				15	16
100 / 10	8/6 13/6 15/6	GC	CLAYEY GRAVEL; very stiff, moist, black, coarse gravel (2 to 3 inch in cuttings) prevented full sample recovery	low recovery DD= 97.8 pcf WD= 110.6 pcf	28	13
95 / 15	4/6 7/6 7/6	CL	SANDY LEAN CLAY; stiff, moist, low plasticity, light brown		14	15
90 / 20	6/6 10/6 16/6		Very stiff, low to moderate plasticity, light brown to brown		26	12
85 / 25	5/6 5/6 5/6		color is light brown, decrease in plasticity to low/slight	DD= 107.5 pcf WD= 120.2 pcf	10	12
			Bottom of Boring			

Notes:

Figure Number

Test Boring: S-7

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 5, 2019

Elevation: 108.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6 2/6 2/6	CL	SANDY LEAN CLAY; soft, moist, low to moderate plasticity, dark-brown		4	22
	6/6 7/6 9/6		at 2 feet, very stiff, color is brown to dark brown		16	17
105 / 5	6/6 6/6 7/6		Color is light brown, calcification noted	DD= 98.8 pcf WD= 114.4 pcf ø = 32° C = 90 psf -200= 67% +4 = 0% LL = 42 PI = 21	13	16
100 / 10	3/6 3/6 4/6				7	16
95 / 15	5/6 8/6 12/6		Very stiff, color is light brown some calcification	DD= 105.1 pcf WD= 120.0 pcf	20	14
90 / 20	4/6 6/6 5/6		color is light orange, slight iron oxide staining		11	15
85 / 25	6/6 6/6 6/6	ML	SANDY SILT; stiff, moist, slightly plastic, light brown		12	12
80			Bottom of Boring			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 5, 2019

Elevation: 107 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	1/6 1/6 1/6	CL	SANDY LEAN CLAY; soft, moist, low to moderate plasticity, dark brown		2	19
105	3/6 6/6 9/6		at 2 feet, stiff, some red colored gravel, weak cementation,		15	17
5	7/6 9/6 11/6		at 5 feet, very stiff, brown to dark brown, cemented with some gravels	DD= 101.7 pcf WD= 115.6 pcf	20	14
100						
10	5/6 5/6 6/6		stiff, low plasticity, light gray to brown		11	14
95						
15	4/6 4/6 6/6				10	13
90						
20	3/6 4/6 4/6		medium stiff		8	16
85						
25	14/6 13/6 26/6		Hard, low plastic	DD= 96.6 pcf WD= 103.8 pcf	39	8
80						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: S-8

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: Pacific Drilling

Drill Type: Fastre SPT

Auger Type: 6 inch hollow stem

Hammer Type: 140 pound auto trip

Logged By: JC

Date: March 5, 2019

Elevation: 107 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	8/6 9/6 12/6		very stiff, color is light brown		21	20
75						
35	10/6 10/6 10/6	SP-SM	POORLY GRADED SAND with Silt; medium dense, moist, fine to medium grained, light brown	-200= 7.2% +4 = 10%	20	2
70						
40	8/6 8/6 8/6				16	2
65						
45	36/6 50/3		with gravel, very dense, trace clay		>50	2
60	50/5					
			Auger and sampler refusal in dense sands/gravel/cobbles			
50						
55						
55						
50						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: Perc - 1

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Logged By: JC

Drilled By: JC

Date: 3/12/2019

Drill Type: CME-75

Elevation: 50 feet AMSL

Auger Type: 6-5/8 inches hollow stem

Hammer Type: 140 pound Auto Trip

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
50 / 0						
		SM	SILTY SAND; moist, brown			
45 / 5	4/6 5/6 7/6		medium dense, moist, fine to medium grained, trace of coarse gravel		12	
			Bottom of Boring			
40 / 10						
35 / 15						
30 / 20						
25 / 25						

Notes: *Percolation test installed adjacent to L-8 location*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: Perc - 2

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inches hollow stem

Hammer Type: 140 pound Auto Trip

Logged By: JC

Date: 3/12/2019

Elevation: 49.5 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0						
1		CL	SANDY LEAN CLAY, very stiff, moist, dark brown, interbedded with Silty Sand layers			
8.5	3/6 9/6 10/6	SC	CLAYEY SAND interbedded with Silty Sand layers; medium dense, moist, fine to coarse grained brown		19	
12			Bottom of Boring			

Depth scale: 0, 5, 10, 15, 20, 25 feet; Elevation scale: 45, 40, 35, 30, 25

Notes: *Percolation test installed adjacent to I-8 location*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: Perc - 3

Project: Home Depot Store - Mission Valley - San Diego, CA

Project Number: D050R0.01

Drilled By: JC

Drill Type: CME-75

Auger Type: 6-5/8 inches hollow stem

Hammer Type: 140 pound Auto Trip

Logged By: JC

Date: 3/12/2019

Elevation: 51 feet AMSL

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0 / 51		SC	FILL; CLAYEY SAND with Gravel, brown			
6 / 45		SC	NATIVE; CLAYEY SAND, moist, fine grained, brown			
13.5 / 37.5	5/6 9/6 10/6	SM	SILTY SAND, medium dense, moist, fine grained, brown with some Sandy Silt zones	-200 = 43.6%	19	
16 / 35			Bottom of Boring			

Depth scale (feet): 0, 5, 10, 15, 20, 25

Elevation scale (feet): 50, 45, 40, 35, 30, 25

Notes: *Percolation test installed adjacent to M-8 location*

Figure Number

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	ASPHALTIC CONCRETE
	CLAYEY SAND FILL SOILS
	SC: Clayey sand
	SM: Silty sand
	ML: Silt
	CL: LEAN CLAY
	GP-GM: Poorly graded gravel with silt
	GE-GC: Well graded gravel with clay
	GC: Clayey gravel
Misc. Symbols	
	Drill rejection

Notes:

1. Test borings in the building area were drilled from February 25 through March 12, 2019 using a CME 75 drill rig quipped with 6 inch O.D. hollow stem augers. Test borings on the upper slope bench were drilled using a Fastre SPT track mounted drill rig quipped with 6 inch O.D. hollow stem augers. Test Borings in the parking structure area were dirlled from September 17 through the September 20, 2019 using a CME 75 drill rig quipped with 6 inch O.D. hollow stem augers. ALso a single boring M-8C was drilled on December 26, 2019 using a Yeti M10 drill rig quipped with 8 inch O.D. hollow stem augers.

2. Groundwater was encountered in deeper boings during drilled and depths are indicated on the borings logs.

3. Boring locations were located with reference to the existing site features.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. The "N-value" reported for the California Modified Split Barrel Sampler is the uncorrected field blow count. This value shold not be interpreted as an SPT equivalent N-value.

6. Results of tests conducted on samples recovered are reported on the logs. Abbreviations used are:

AMSL =	Above mean sea level	RV =	Ressistance Value
O.D. =	Outside diameter	WD =	Wet Density (pcf)
DD =	Dry density (pcf)	+4 =	Percent Retained on #4 sieve
-#200 =	Percent passing #200 sieve (%)	N/A =	Not applicable
N/E =	None encountered	pcf =	pounds per cubic foot
psf =	pounds per square foot	BSG =	below site grade
LL =	Liquid Limit	PI =	Plasticity Index
C =	Cohesion	ø =	Angle of Internal Friction

KEY TO SYMBOLS

Symbol	Description
Soil Samplers	
	Standard penetration test
	California Modified split barrel ring sampler

APPENDIX C

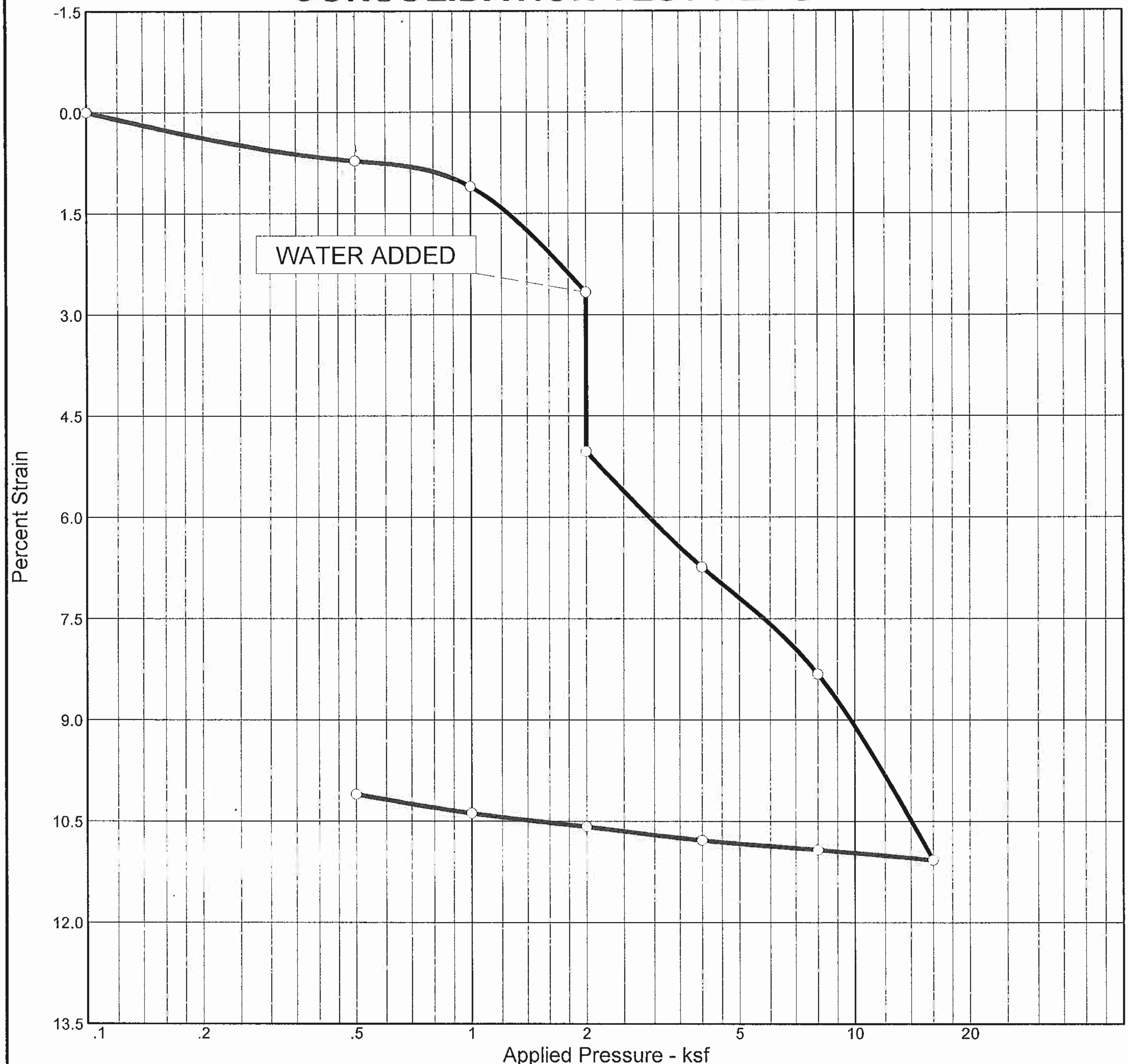
RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	To Determine:
Moisture Content (ASTM D2216)	Moisture contents representative of field conditions at the time the sample was taken.
Dry Density (ASTM D2937)	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Grain-Size Distribution (ASTM D422)	Size and distribution of soil particles, i.e., clay, silt, sand, and gravel.
Atterberg Limits (ASTM D4318)	Determines the moisture content at which the soil behaves as a viscous material (liquid limit) and the moisture content at which the soil reaches a plastic state.
Expansion Index (ASTM D4829)	Swell potential of soil with increases in moisture content.
Consolidation (ASTM D2435)	The amount and rate at which a soil sample compresses when loaded, and the influence of saturation on its behavior.
Direct Shear (ASTM D3080)	Soil shearing strength under varying loads and/or moisture conditions.

These Included:	To Determine:
Moisture-Density Relationship (ASTM D1557)	The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.
R-Value (ASTM D2844)	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Sulfate Content (ASTM D4327)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (ASTM D4327)	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM G187)	The potential of the soil to corrode metal.
pH (ASTM D4972)	The acidity or alkalinity of subgrade material.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
96.9 %	22.0 %	103.3			2.65		1.53	0.15	0.01		2.3	0.601

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy lean clay	CL	

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

Source: **Sample No.:** E-4 **Elev./Depth:** 10-11.5'

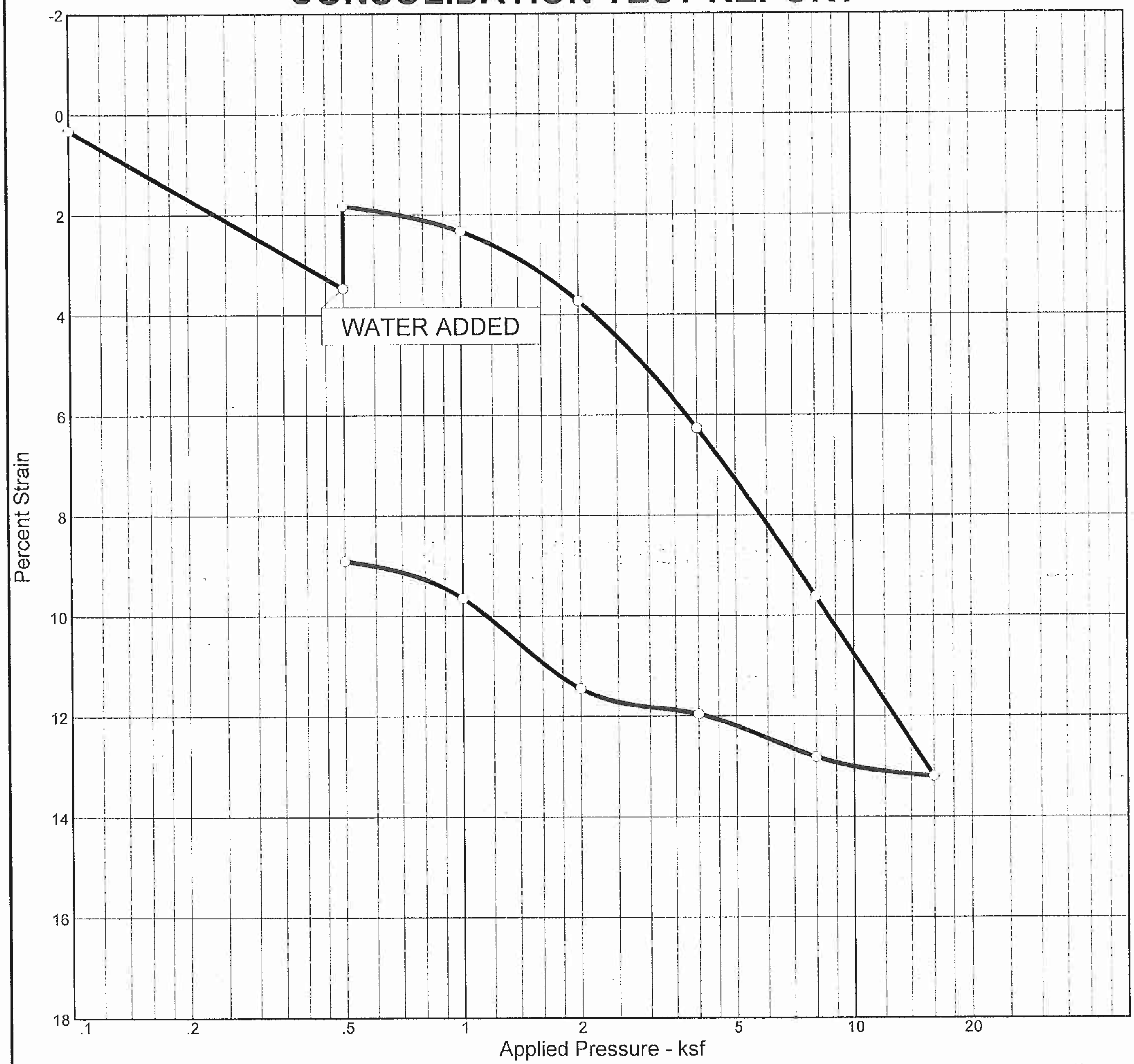
Remarks:

Moore Twining Associates, Inc.

Fresno, CA

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
90.8 %	21.4 %	101.8			2.65		2.31	0.19	0.05	1.83	1.7	0.625

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

Source: **Sample No.:** F-8 **Elev./Depth:** 5-6.5'

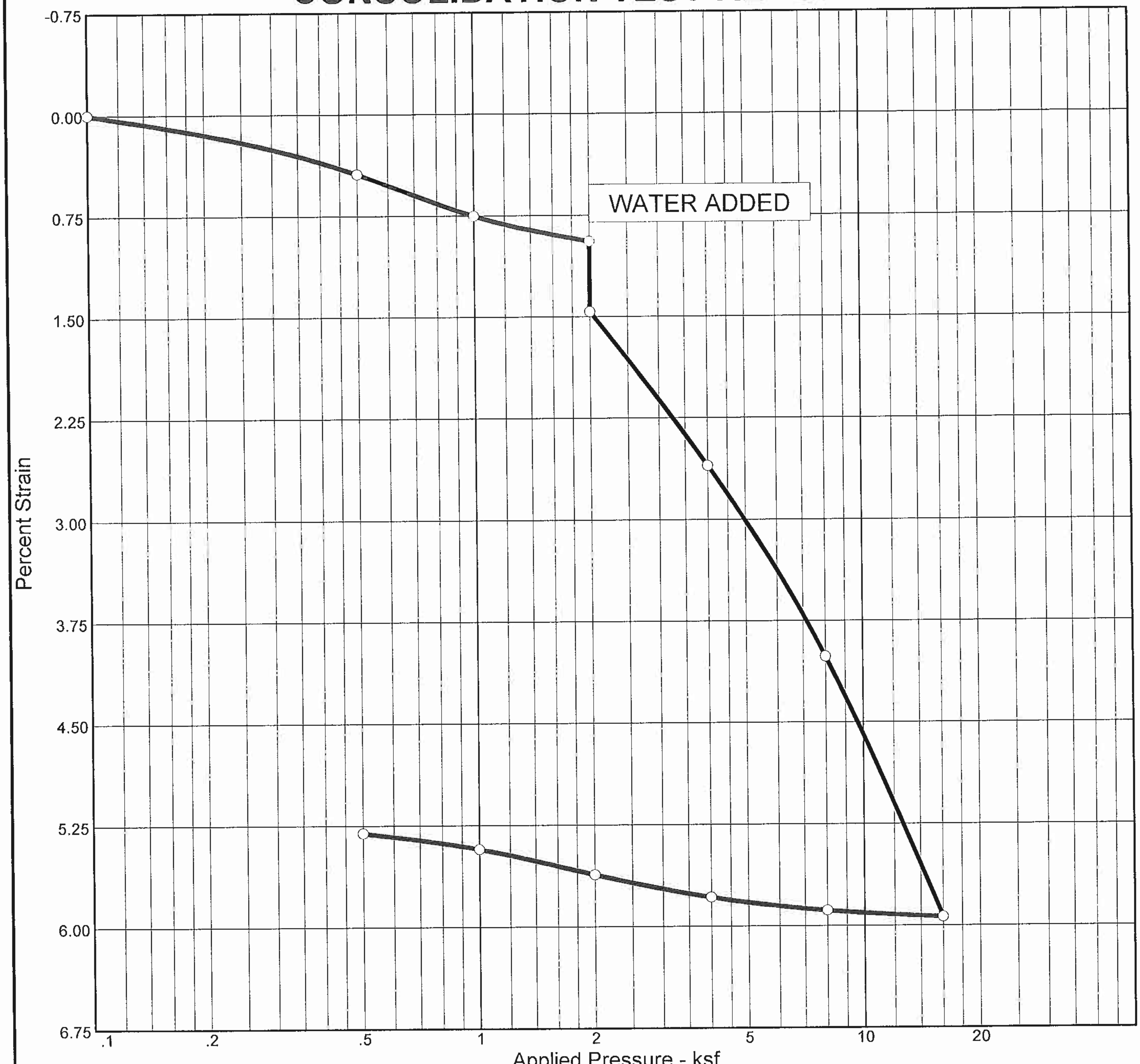
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
72.0 %	13.1 %	111.5	36	13	2.65		8.30	0.10	0.01		0.6	0.483

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy lean clay	CL	

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

Source: **Sample No.:** G-5 **Elev./Depth:** 2.5-4'

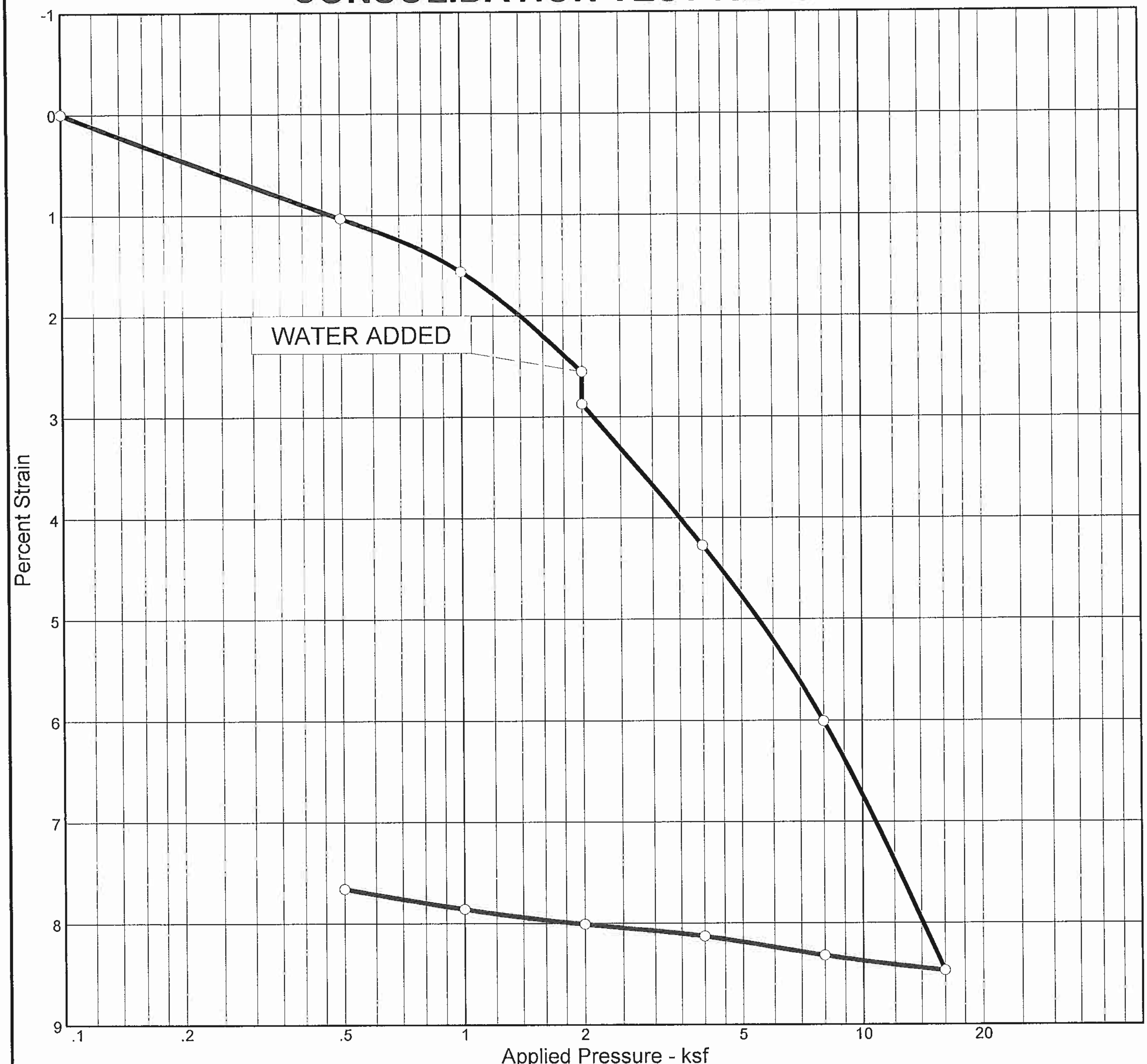
Remarks:

Moore Twining Associates, Inc.

Fresno, CA

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
80.3 %	18.1 %	103.6	36	13	2.65		2.80	0.13	0.01		0.3	0.596

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy lean clay	CL	

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

Source: **Sample No.:** G-6 **Elev./Depth:** 1-2.5'

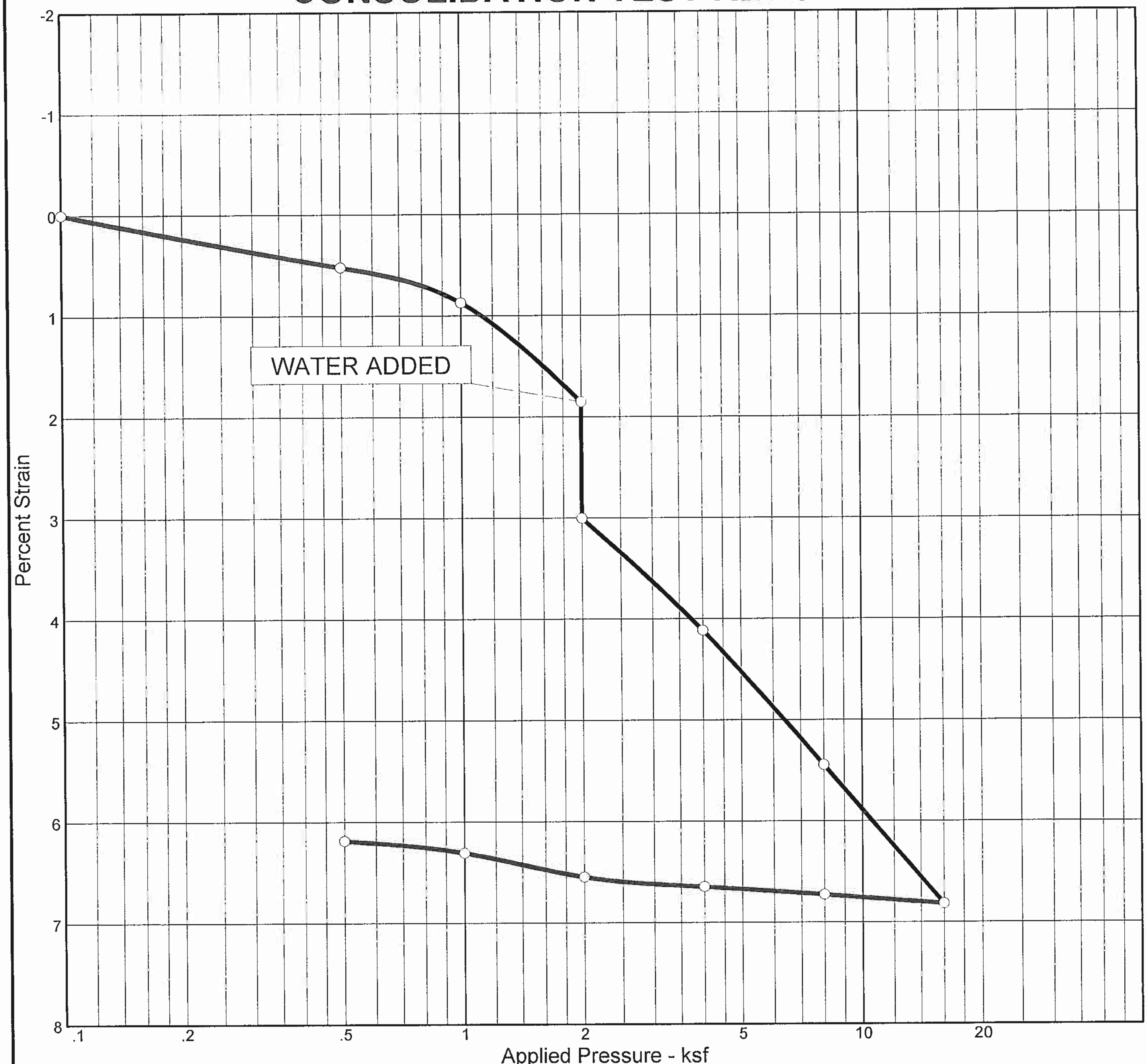
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
51.1 %	11.3 %	104.2			2.65		0.73	0.07	0.01		1.1	0.587

MATERIAL DESCRIPTION	USCS	AASHTO
Clayey sand	SC	

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

Source: **Sample No.:** H-6 **Elev./Depth:** 5-6.5'

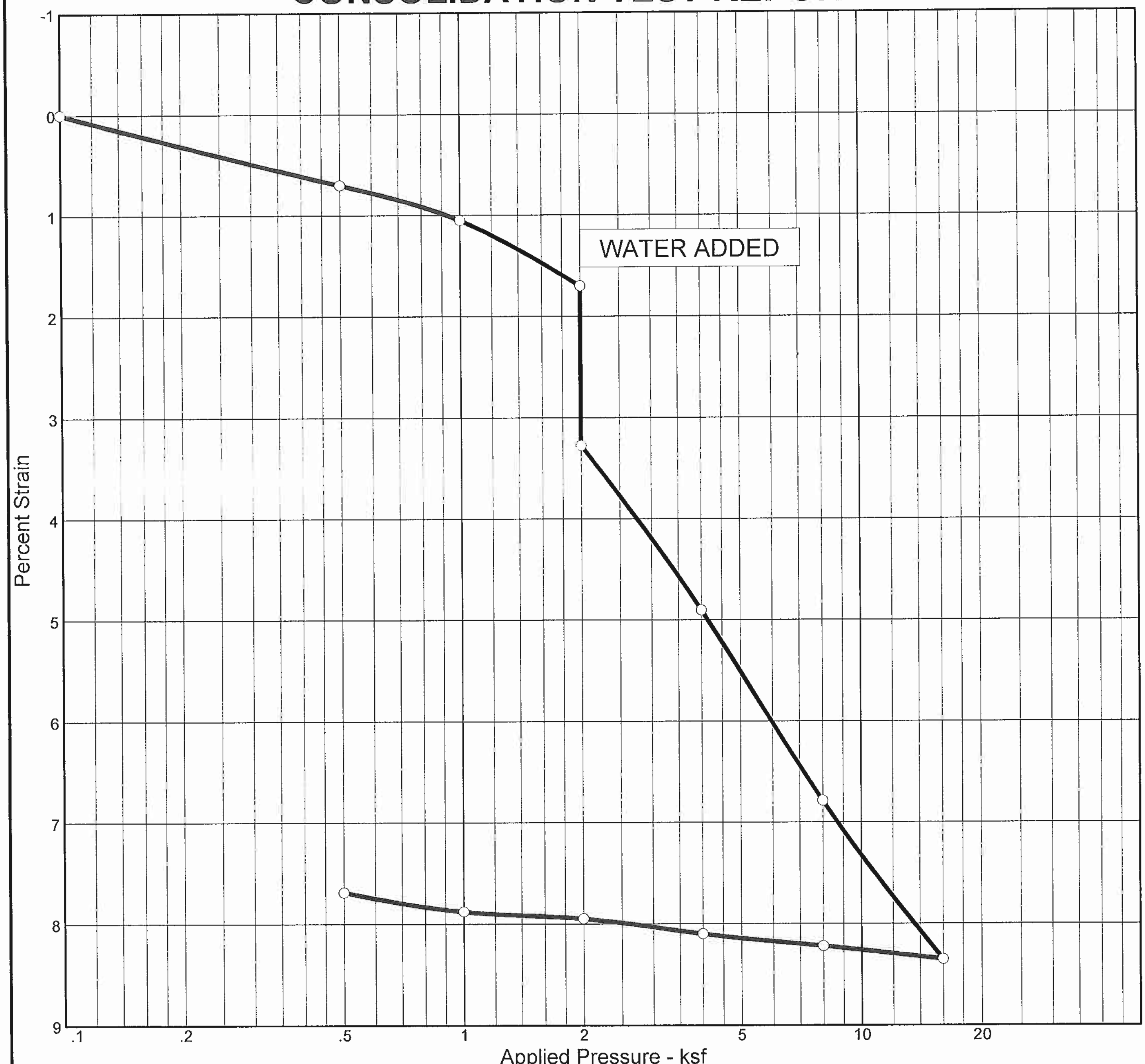
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
66.9 %	14.0 %	106.5			2.65		0.53	0.08	0.01		1.6	0.554

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy lean clay	CL	

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

Source: **Sample No.:** I-6 **Elev./Depth:** 1.5-3'

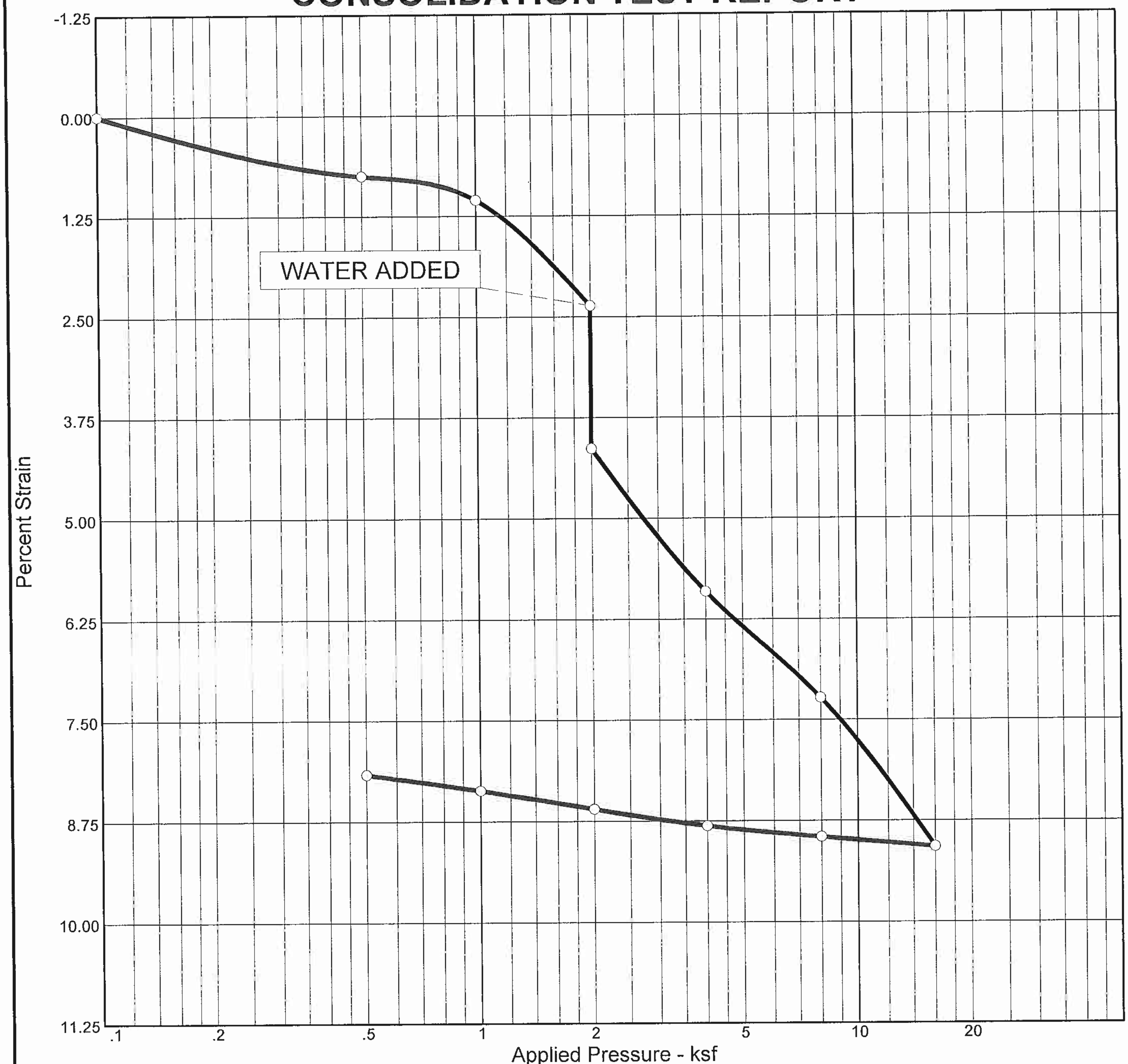
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
73.9 %	13.3 %	112.0			2.65		0.82	0.09	0.01		1.7	0.476

MATERIAL DESCRIPTION	USCS	AASHTO
Clayey gravel with sand	GC	

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

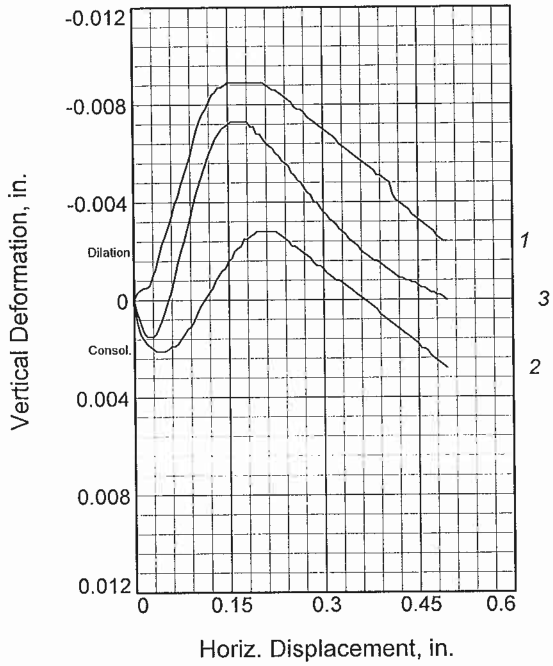
Source: **Sample No.:** J-4 **Elev./Depth:** 5-6.5'

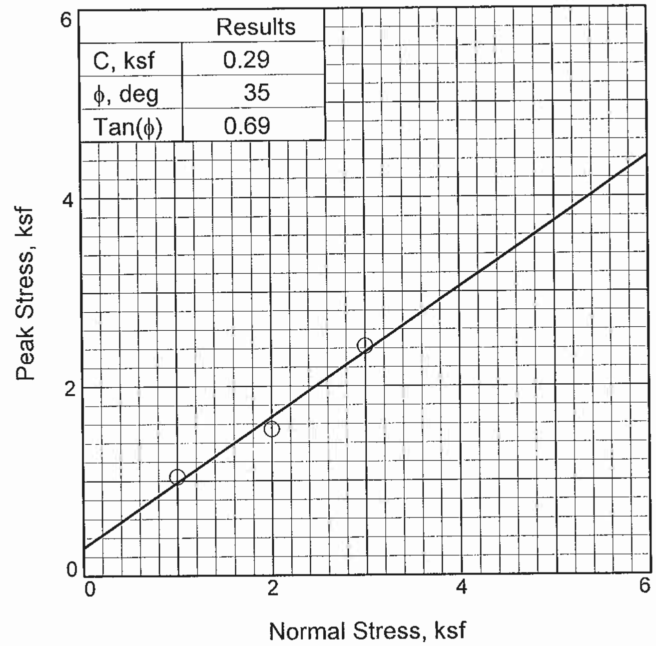
Moore Twining Associates, Inc.

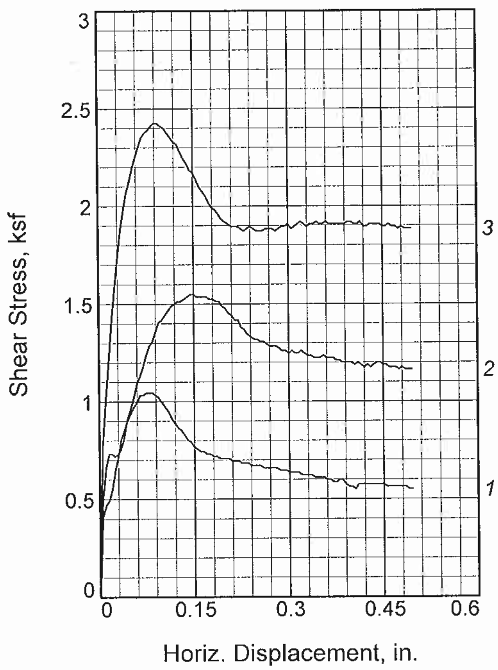
Fresno, CA

Remarks:

Figure







	Sample No.	1	2	3
Initial	Water Content, %	15.2	15.5	15.6
	Dry Density, pcf	103.8	103.9	104.0
	Saturation, %	67.9	69.4	69.8
	Void Ratio	0.5939	0.5927	0.5914
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	21.1	20.3	19.7
	Dry Density, pcf	105.6	106.1	106.8
	Saturation, %	98.5	96.3	95.0
	Void Ratio	0.5667	0.5592	0.5484
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.98	0.97
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	1.04	1.55	2.42
	Displacement, in.	0.08	0.14	0.09
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	3.1	6.0	3.7

Sample Type:

Description: Sandy lean clay

Specific Gravity= 2.65

Remarks:

Figure ________

Client:

Project: Mission Valley - Home Depot

Sample Number: C-5 **Depth:** 5-6.5'

Proj. No.: D050R0.01 **Date Sampled:** 2/25/19

DIRECT SHEAR TEST REPORT

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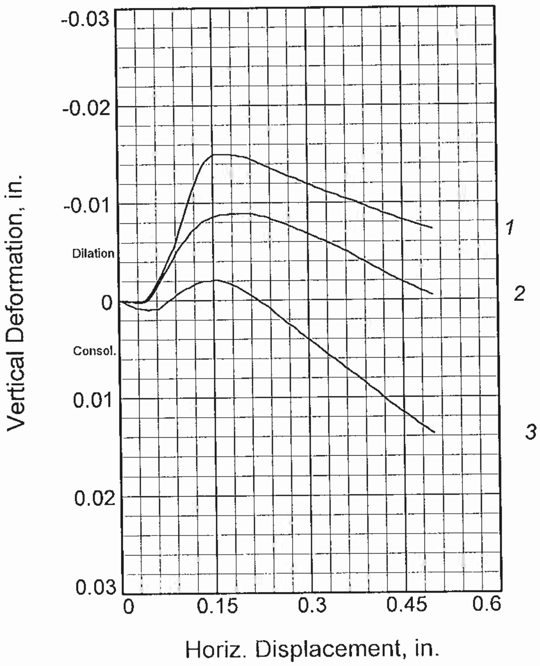

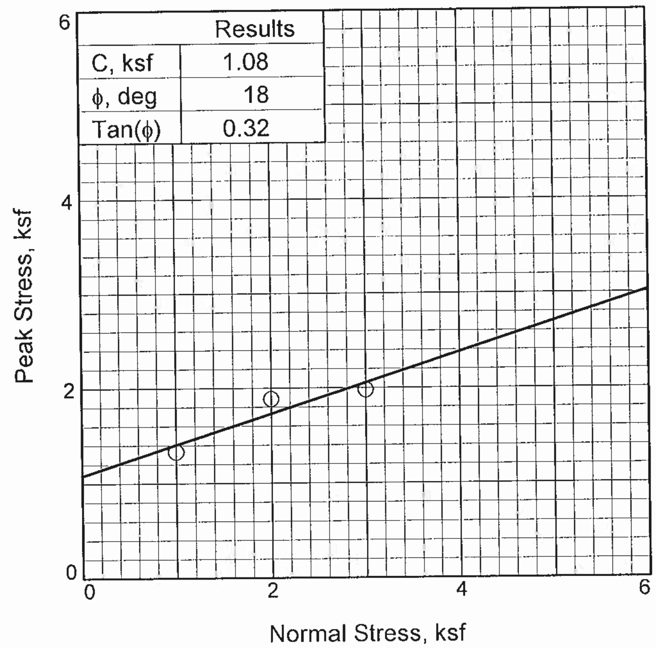

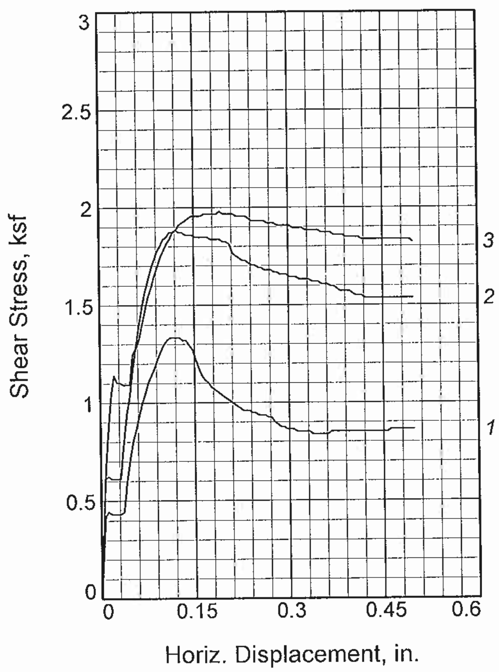

	Sample No.	1	2	3
Initial	Water Content, %	17.3	17.3	17.4
	Dry Density, pcf	108.2	108.5	109.5
	Saturation, %	86.6	87.5	90.2
	Void Ratio	0.5290	0.5244	0.5105
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	18.6	17.9	17.2
	Dry Density, pcf	109.9	111.1	112.6
	Saturation, %	97.5	97.1	97.2
	Void Ratio	0.5052	0.4890	0.4695
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.98	0.97
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	1.33	1.88	1.98
	Displacement, in.	0.11	0.12	0.19
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	4.6	5.0	7.9

Sample Type:
Description: Clayey sand (Fill)

Specific Gravity= 2.65
Remarks:

Figure ________

Client:

Project: Mission Valley - Home Depot

Sample Number: S-1 **Depth:** 2-3.5'

Proj. No.: D050R0.01 **Date Sampled:** 3/4/19

DIRECT SHEAR TEST REPORT
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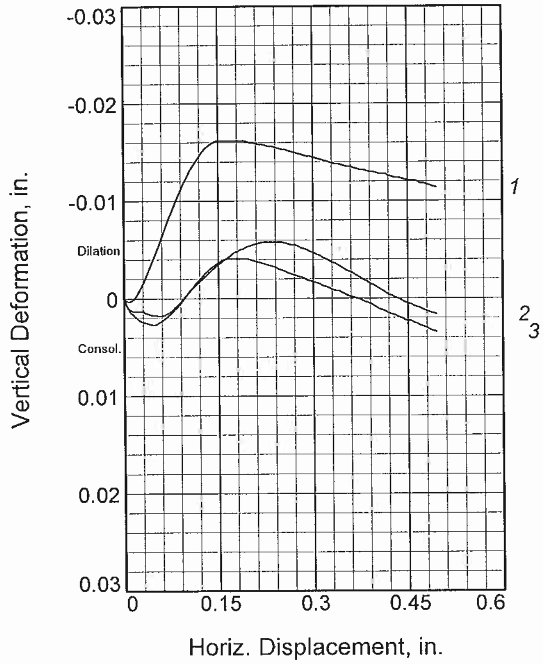

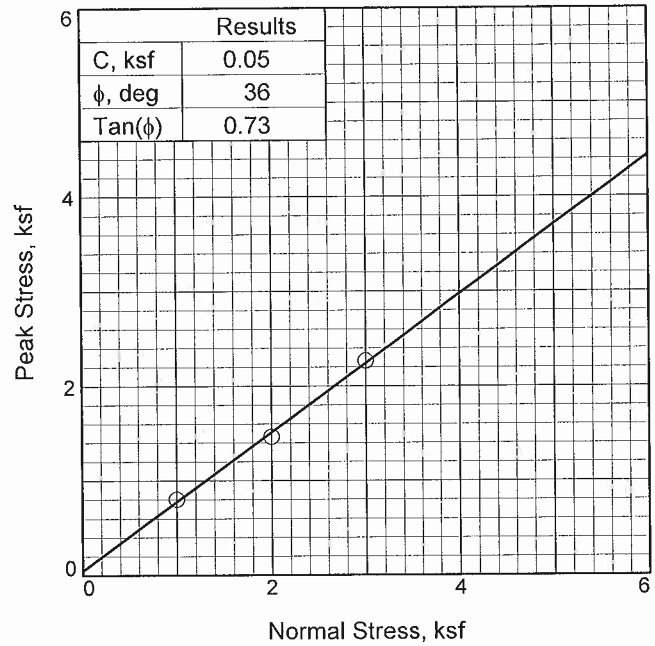

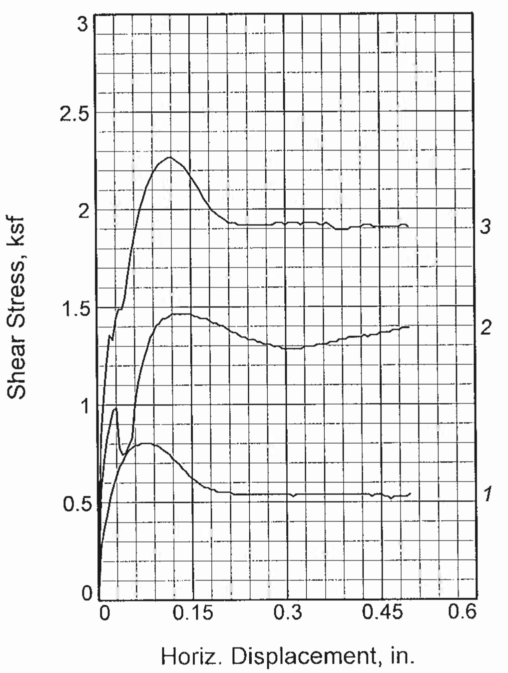

	Sample No.	1	2	3
Initial	Water Content, %	7.8	7.7	7.7
	Dry Density, pcf	94.6	96.1	95.1
	Saturation, %	27.7	28.3	27.6
	Void Ratio	0.7494	0.7222	0.7387
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	26.6	24.8	25.2
	Dry Density, pcf	96.1	98.1	97.4
	Saturation, %	97.7	95.8	95.5
	Void Ratio	0.7219	0.6869	0.6982
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.98	0.98
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.80	1.46	2.27
	Displacement, in.	0.07	0.12	0.12
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	2.9	5.0	4.8

Sample Type:

Description: Sandy lean clay

Specific Gravity= 2.65

Remarks:

Figure ________

Client:

Project: Mission Valley - Home Depot

Sample Number: S-2 **Depth:** 5-6.5'

Proj. No.: D050R0.01 **Date Sampled:** 3/4/19

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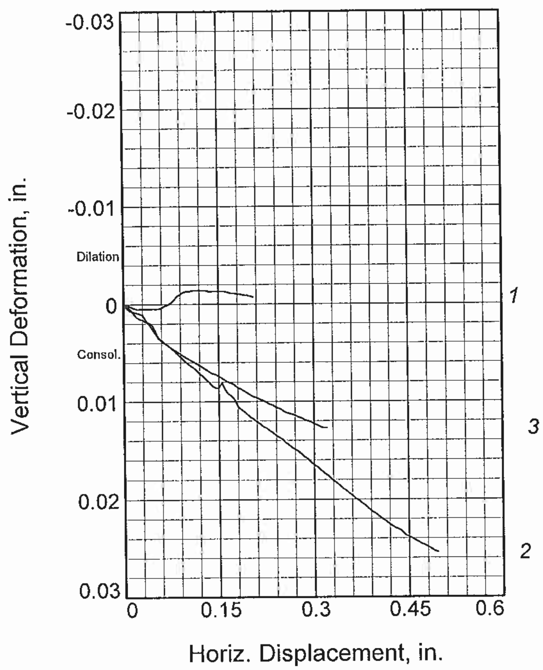
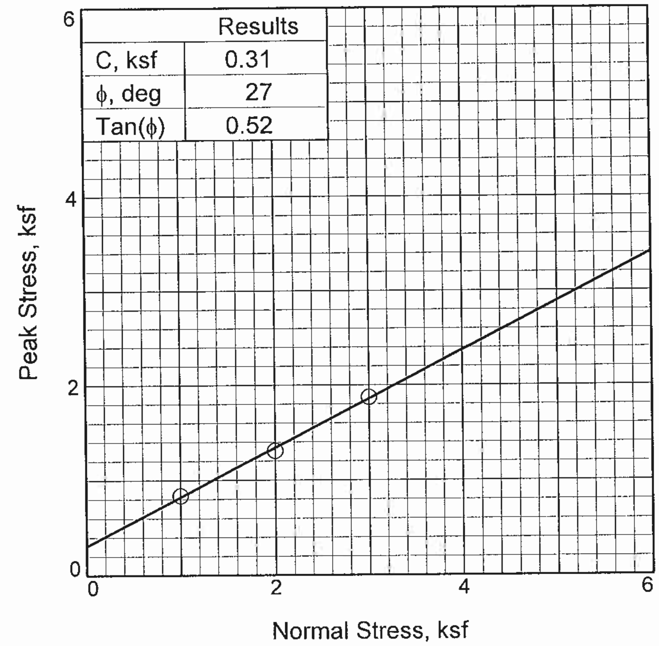
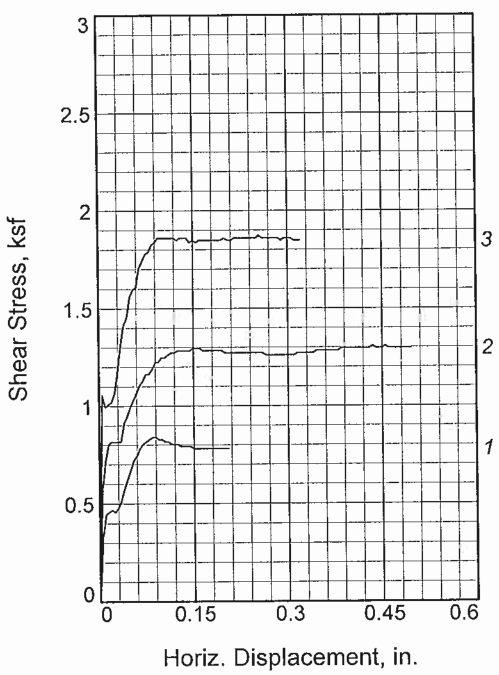


Results	
C, ksf	0.31
ϕ, deg	27
Tan(ϕ)	0.52





	Sample No.	1	2	3
Initial	Water Content, %	14.0	14.2	14.3
	Dry Density, pcf	104.4	104.1	103.9
	Saturation, %	63.5	63.9	64.0
	Void Ratio	0.5850	0.5888	0.5927
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	20.6	20.3	19.9
	Dry Density, pcf	106.1	106.4	106.7
	Saturation, %	97.7	96.7	95.9
	Void Ratio	0.5594	0.5553	0.5503
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.98	0.97
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.84	1.31	1.87
	Displacement, in.	0.09	0.44	0.26
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	3.5	18.0	10.6

Sample Type:

Description: Silt with sand

LL= 37 **PL=** 18 **PI=** 19

Specific Gravity= 2.65

Remarks:

Figure ________

Client:

Project: Mission Valley - Home Depot

Sample Number: S-5 **Depth:** 30-31.5'

Proj. No.: D050R0.01 **Date Sampled:** 3/4/19

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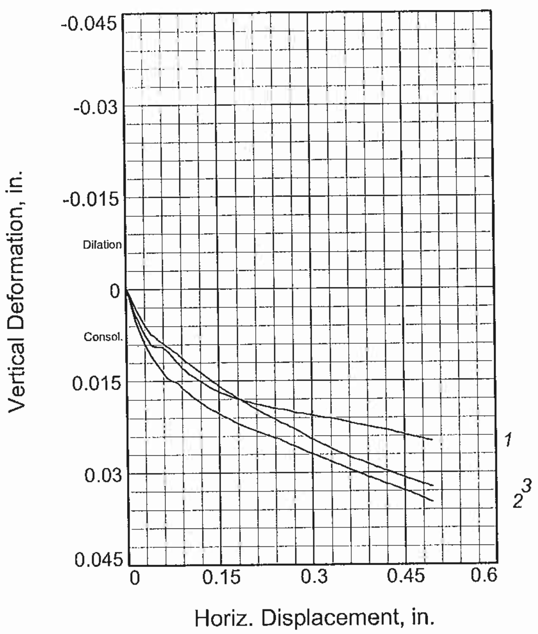

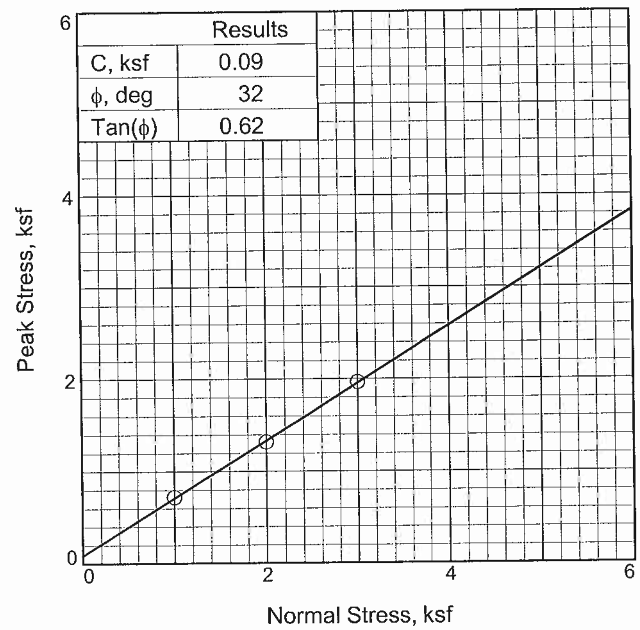

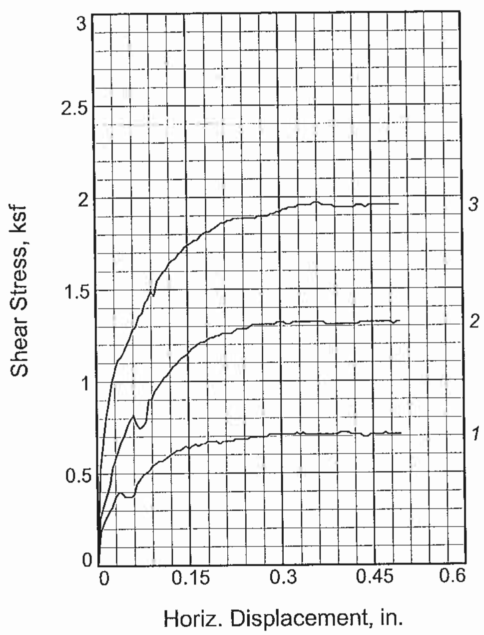

	Sample No.	1	2	3
Initial	Water Content, %	15.5	15.6	15.5
	Dry Density, pcf	100.0	100.2	101.3
	Saturation, %	62.9	63.5	64.9
	Void Ratio	0.6549	0.6508	0.6332
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	22.4	22.4	22.4
	Dry Density, pcf	101.2	102.1	103.4
	Saturation, %	93.8	95.6	99.0
	Void Ratio	0.6340	0.6207	0.5992
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.98	0.98
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.72	1.32	1.97
	Displacement, in.	0.33	0.29	0.35
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	13.5	12.2	14.7

Sample Type:
Description: Sandy lean clay

Specific Gravity= 2.65
Remarks:

Figure ________

Client:

Project: Mission Valley - Home Depot

Sample Number: S-7 **Depth:** 5-6.5'

Proj. No.: D050R0.01 **Date Sampled:** 3/4/19

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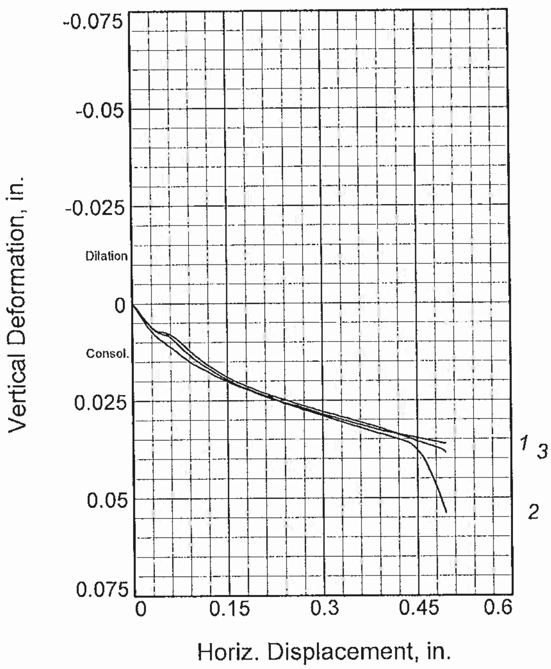

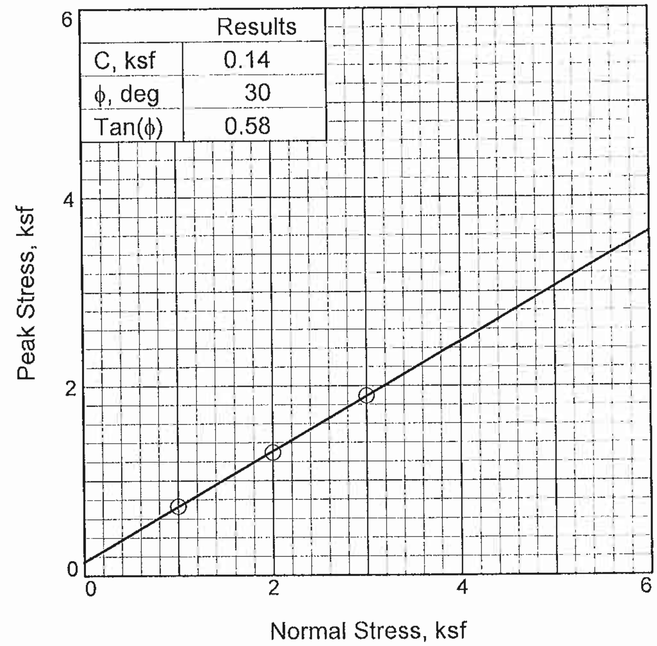

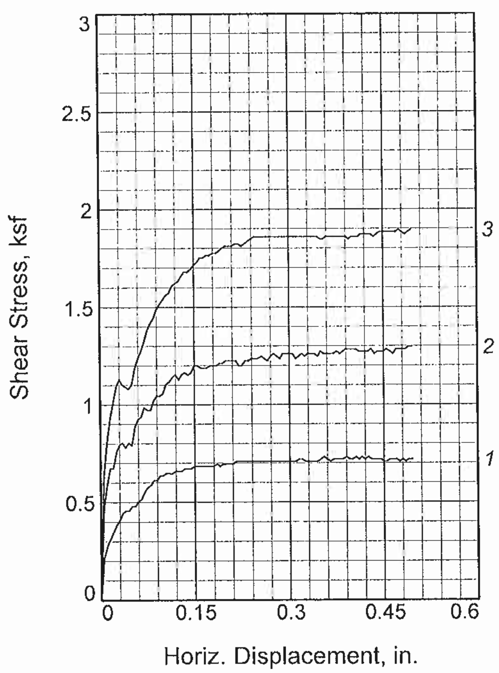

	Sample No.	1	2	3
Initial	Water Content, %	2.6	7.0	8.8
	Dry Density, pcf	90.2	93.2	94.2
	Saturation, %	8.2	23.8	30.9
	Void Ratio	0.8332	0.7759	0.7571
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	26.5	23.2	21.7
	Dry Density, pcf	96.1	100.8	103.1
	Saturation, %	97.5	95.9	95.1
	Void Ratio	0.7208	0.6406	0.6051
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.94	0.92	0.91
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.73	1.30	1.90
	Displacement, in.	0.35	0.49	0.47
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	14.7	20.3	19.7

Sample Type:
Description:

Specific Gravity= 2.65
Remarks:

Figure ______

Client:

Project: Mission Valley - Home Depot

Sample Number: ES-1 **Depth:** 0.5-1'

Proj. No.: D050R0.01 **Date Sampled:** 9/17/19

DIRECT SHEAR TEST REPORT
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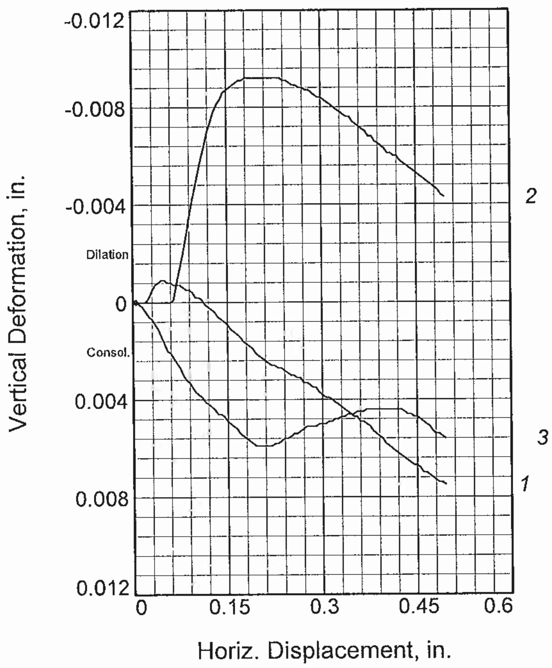
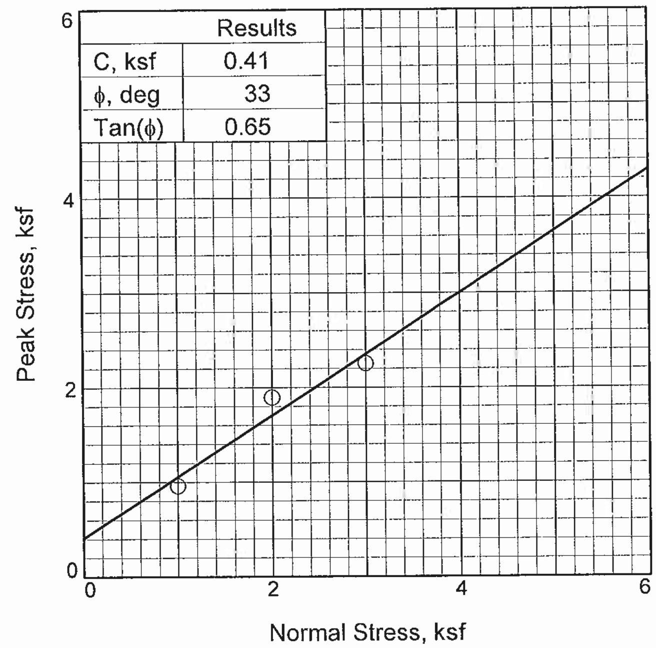
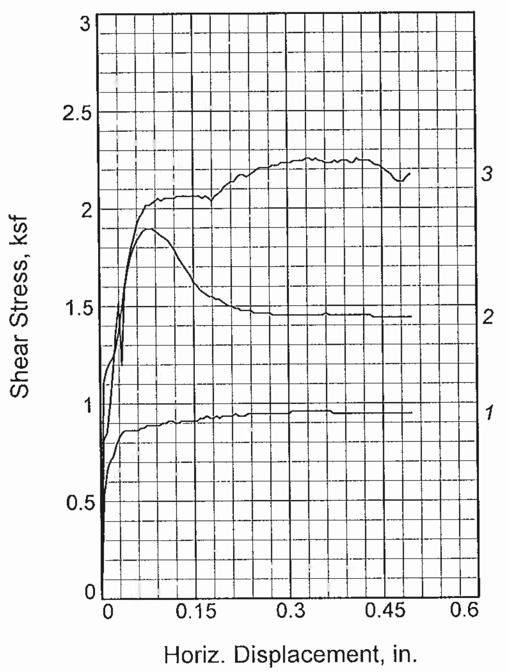
Fresno, CA







	Sample No.	1	2	3
Initial	Water Content, %	8.4	8.3	8.1
	Dry Density, pcf	108.9	114.7	115.9
	Saturation, %	42.8	49.5	50.3
	Void Ratio	0.5197	0.4429	0.4271
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	18.2	14.6	13.8
	Dry Density, pcf	111.5	118.1	119.6
	Saturation, %	100.0	96.6	95.3
	Void Ratio	0.4835	0.4009	0.3827
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.97	0.97
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.96	1.90	2.26
	Displacement, in.	0.30	0.08	0.33
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	12.6	3.1	13.7

Sample Type:

Description: Clayey sand

LL= 31 **PL=** 15 **PI=** 16

Specific Gravity= 2.65

Remarks: Remolded to 90% of the maximum dry density of 129.3 pcf and opt. moisture of 8.3%.

Figure ________

Client:

Project: Mission Valley - Home Depot

Sample Number: B-7 **Depth:** 0.8-5.0'

Proj. No.: D050R0.01 **Date Sampled:** 2/27/19

DIRECT SHEAR TEST REPORT

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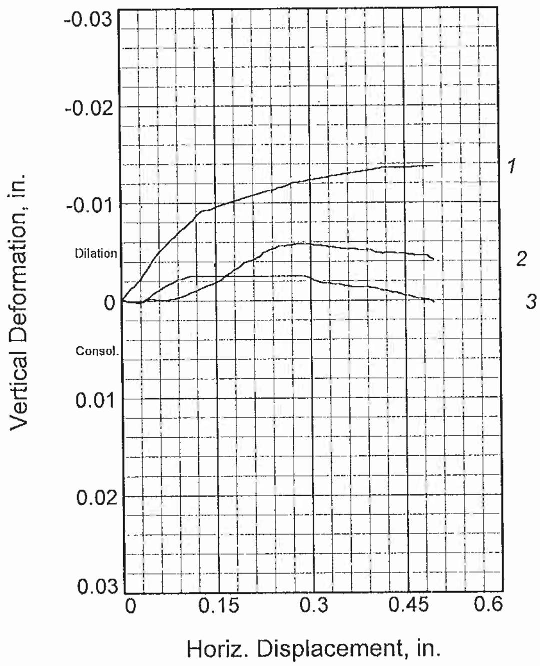

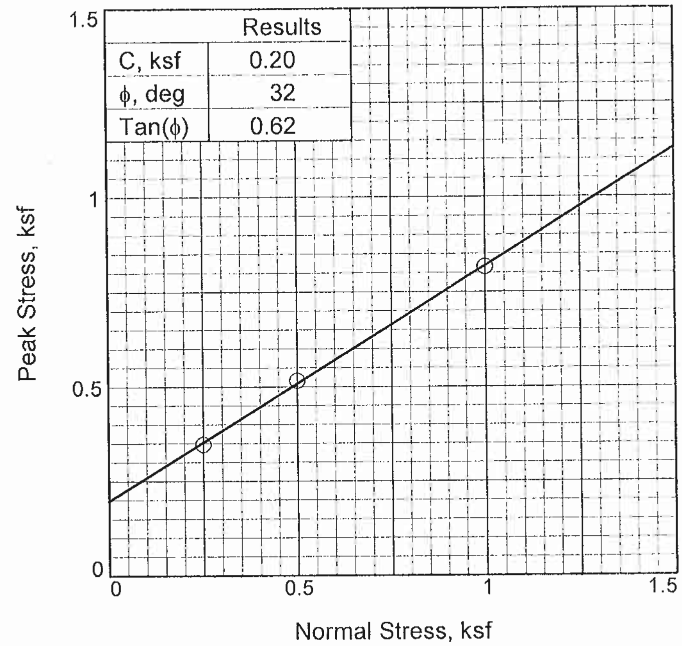

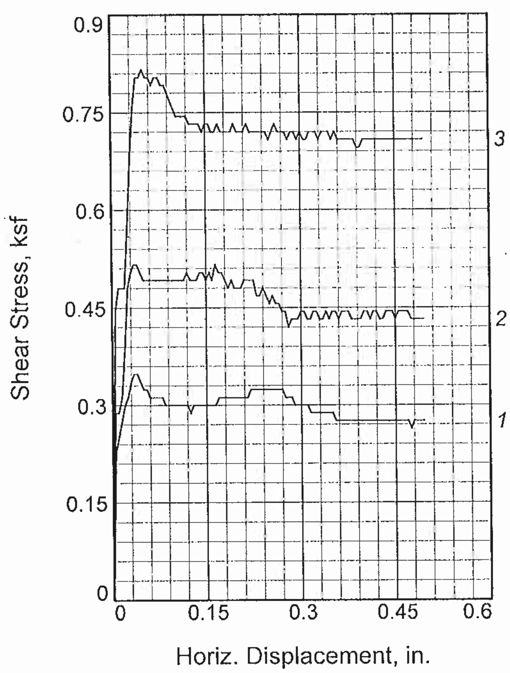

	Sample No.	1	2	3
Initial	Water Content, %	7.6	6.1	7.4
	Dry Density, pcf	118.8	120.6	119.1
	Saturation, %	51.5	43.2	50.4
	Void Ratio	0.3922	0.3720	0.3893
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	14.1	13.0	13.3
	Dry Density, pcf	120.3	122.4	121.3
	Saturation, %	99.4	97.7	96.6
	Void Ratio	0.3746	0.3516	0.3638
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.99	0.98
	Normal Stress, ksf	0.25	0.50	1.00
	Peak Stress, ksf	0.35	0.52	0.82
	Displacement, in.	0.04	0.04	0.05
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	1.4	1.4	2.1

Sample Type:

Description: Clayey sand

LL= 27 **PL=** 15 **PI=** 12

Specific Gravity= 2.65

Remarks: Shear points were remolded to 90% of the max. dry density of 132.2 & opt. moisture of 7.5%

Figure ______

Client:

Project: Mission Valley - Home Depot

Sample Number: E-8 **Depth:** 1-3'

Proj. No.: D050R0.01 **Date Sampled:** 9/17/19

DIRECT SHEAR TEST REPORT

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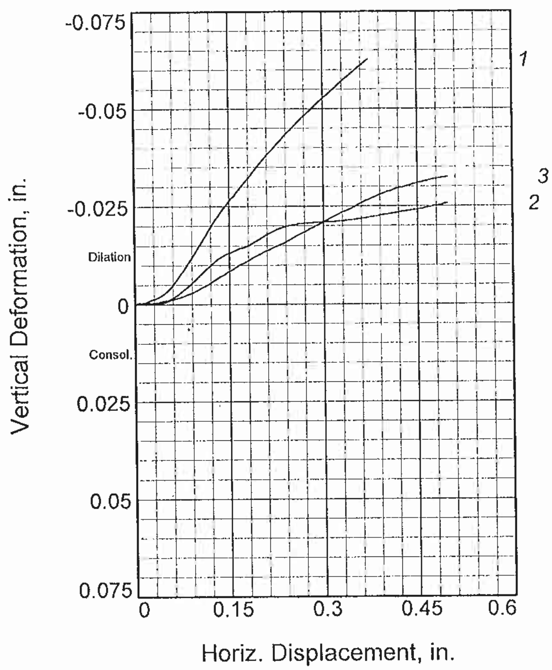

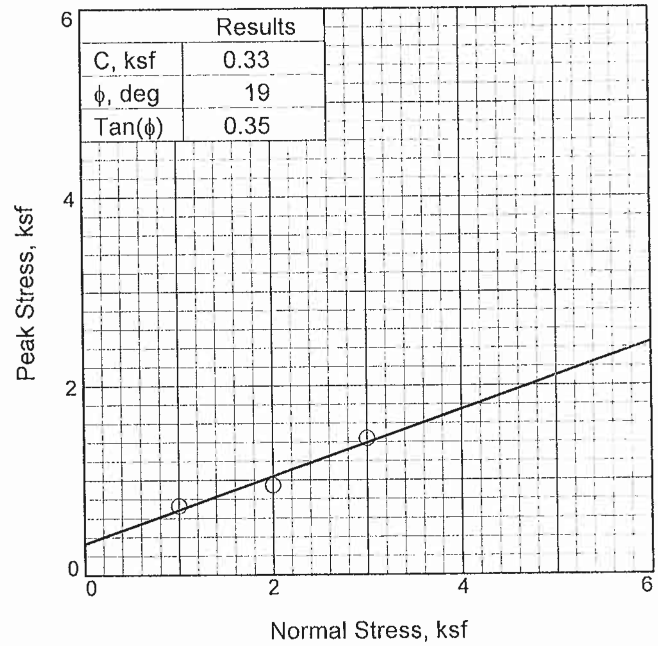

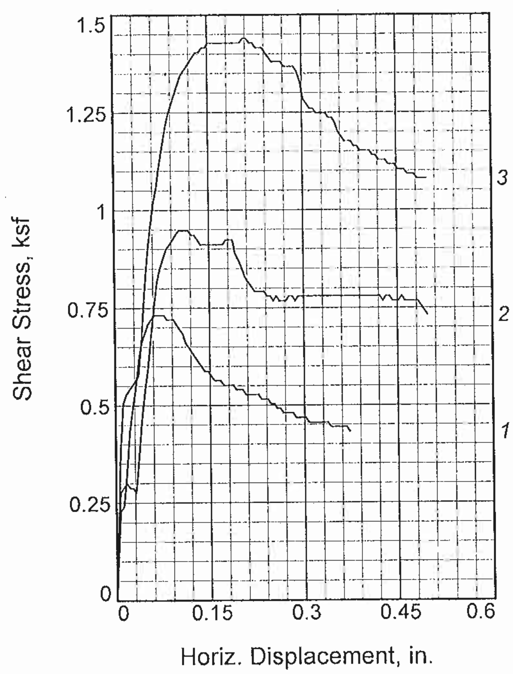

	Sample No.	1	2	3
Initial	Water Content, %	12.5	11.9	12.0
	Dry Density, pcf	110.7	111.3	111.1
	Saturation, %	66.8	64.6	65.3
	Void Ratio	0.4946	0.4868	0.4890
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	16.8	15.5	14.7
	Dry Density, pcf	114.3	116.2	117.7
	Saturation, %	99.5	96.6	95.7
	Void Ratio	0.4480	0.4240	0.4058
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.97	0.96	0.94
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.73	0.95	1.44
	Displacement, in.	0.06	0.10	0.21
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	2.6	4.2	8.6

Sample Type:

Description: Sandy lean clay

LL= 30 **PL=** 18 **PI=** 12

Specific Gravity= 2.65

Remarks: Shear point were remolded to 90 of the max. dry density of 124.8 pcf and opt. moisture of 10.8%.

Figure ________

Client:

Project: Mission Valley - Home Depot

Sample Number: G-7.6 **Depth:** 1-5'

Proj. No.: D050R0.01 **Date Sampled:** 9/17/19

DIRECT SHEAR TEST REPORT

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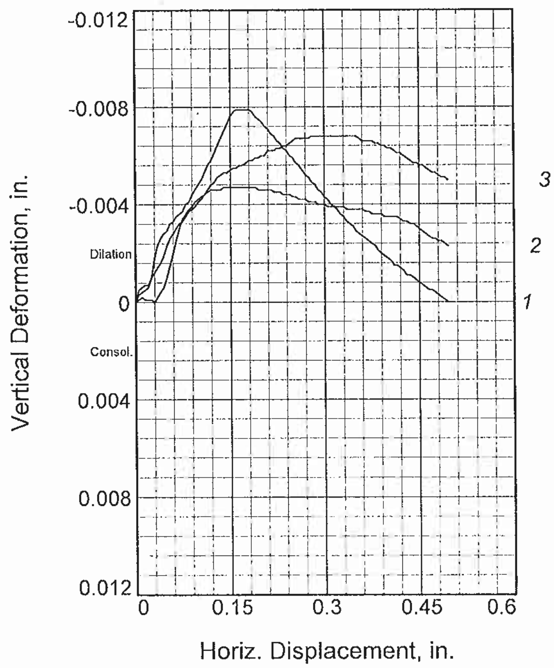
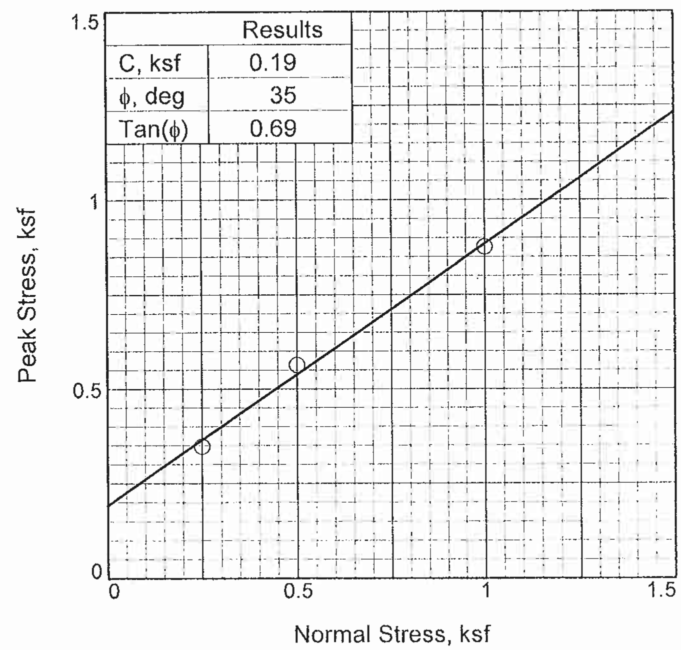
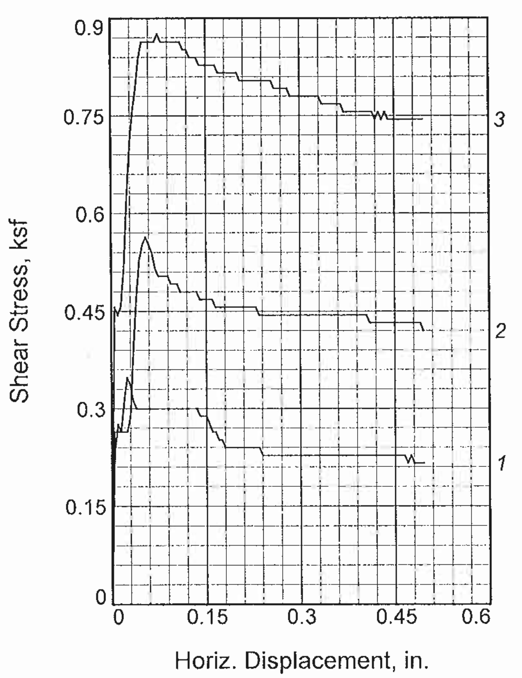






Sample No.		1	2	3
Initial	Water Content, %	12.5	10.1	9.3
	Dry Density, pcf	107.5	109.9	110.7
	Saturation, %	61.6	52.8	49.9
	Void Ratio	0.5385	0.5048	0.4946
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	17.7	15.9	15.2
	Dry Density, pcf	111.8	115.1	116.5
	Saturation, %	97.6	96.7	95.5
	Void Ratio	0.4803	0.4369	0.4206
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.96	0.95	0.95
Normal Stress, ksf		0.25	0.50	1.00
Peak Stress, ksf		0.35	0.56	0.88
Displacement, in.		0.03	0.06	0.08
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		1.0	2.3	3.1

Sample Type:

Description: Sandy lean clay

LL= 31 **PL=** 17 **PI=** 14

Specific Gravity= 2.65

Remarks: Shear points remolded to 90% of the max. dry density of 122.1 pcf and 10.1% opt. moisture.

Figure ______

Client:

Project: Mission Valley - Home Depot

Sample Number: Top Cut Slope (CST)

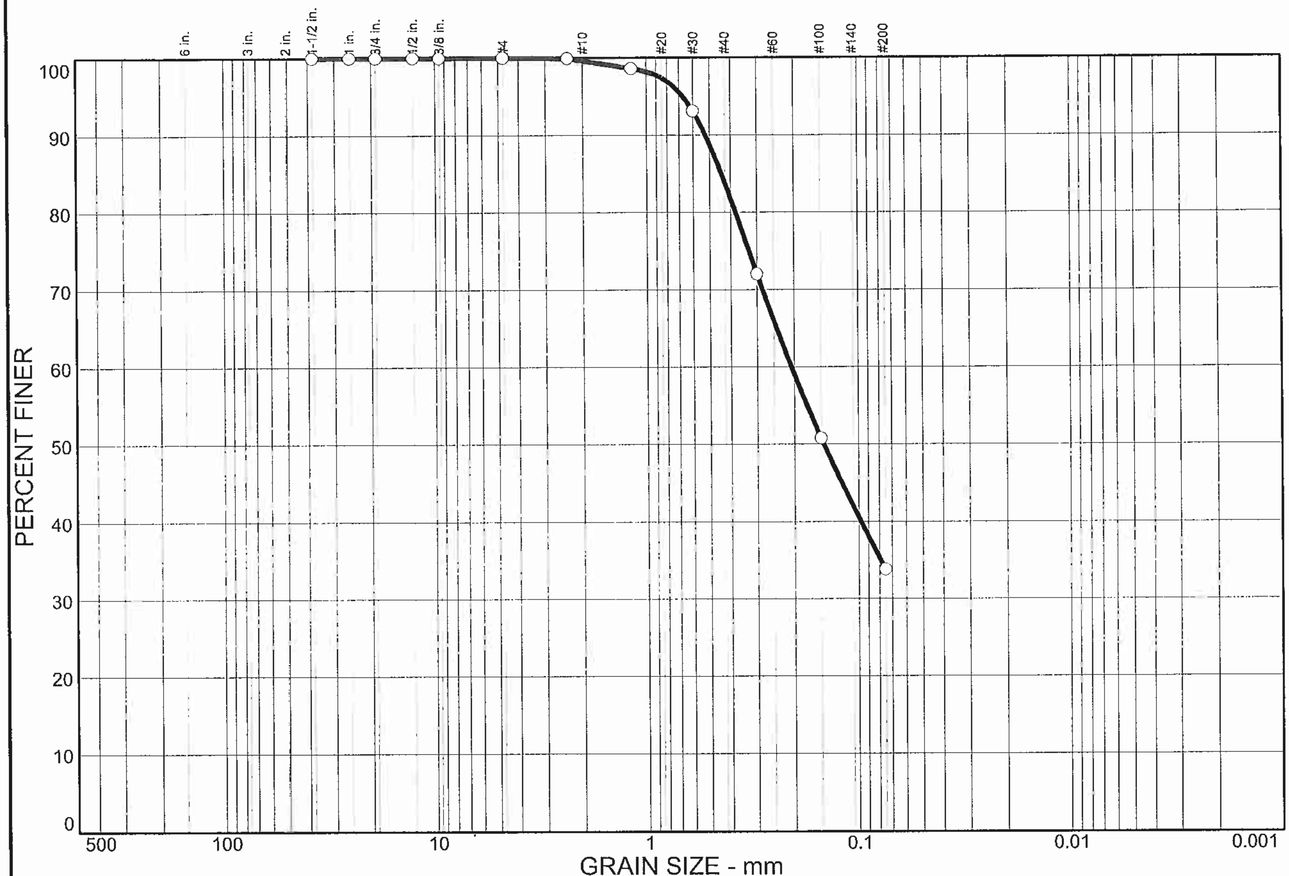
Proj. No.: D050R0.01 **Date Sampled:** 9/17/19

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

Fresno, CA

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	66.2	33.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	100.0		
3/4 in.	100.0		
1/2 in.	100.0		
3/8 in.	100.0		
#4	100.0		
#8	99.9		
#16	98.6		
#30	93.1		
#50	72.0		
#100	50.8		
#200	33.8		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.441 D_{60}= 0.207 D_{50}= 0.146
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: A-2 **Source of Sample:** **Date:**
Location: **Elev./Depth:** 10-11.5'

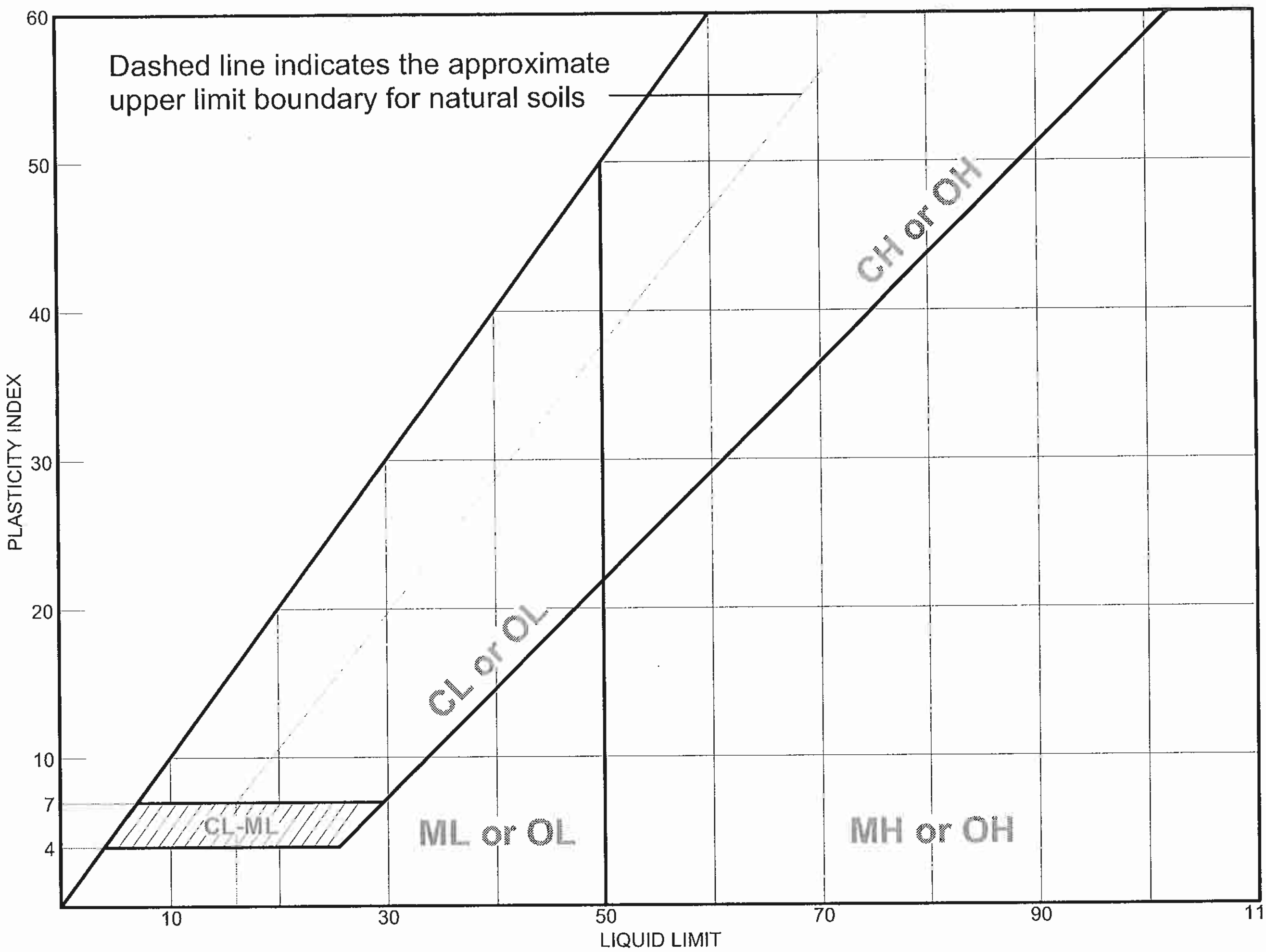
Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot

Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty sand	NV	NP	NP	83.8	33.8	SM

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** A-2 **Elev./Depth:** 10-11.5'

Remarks:

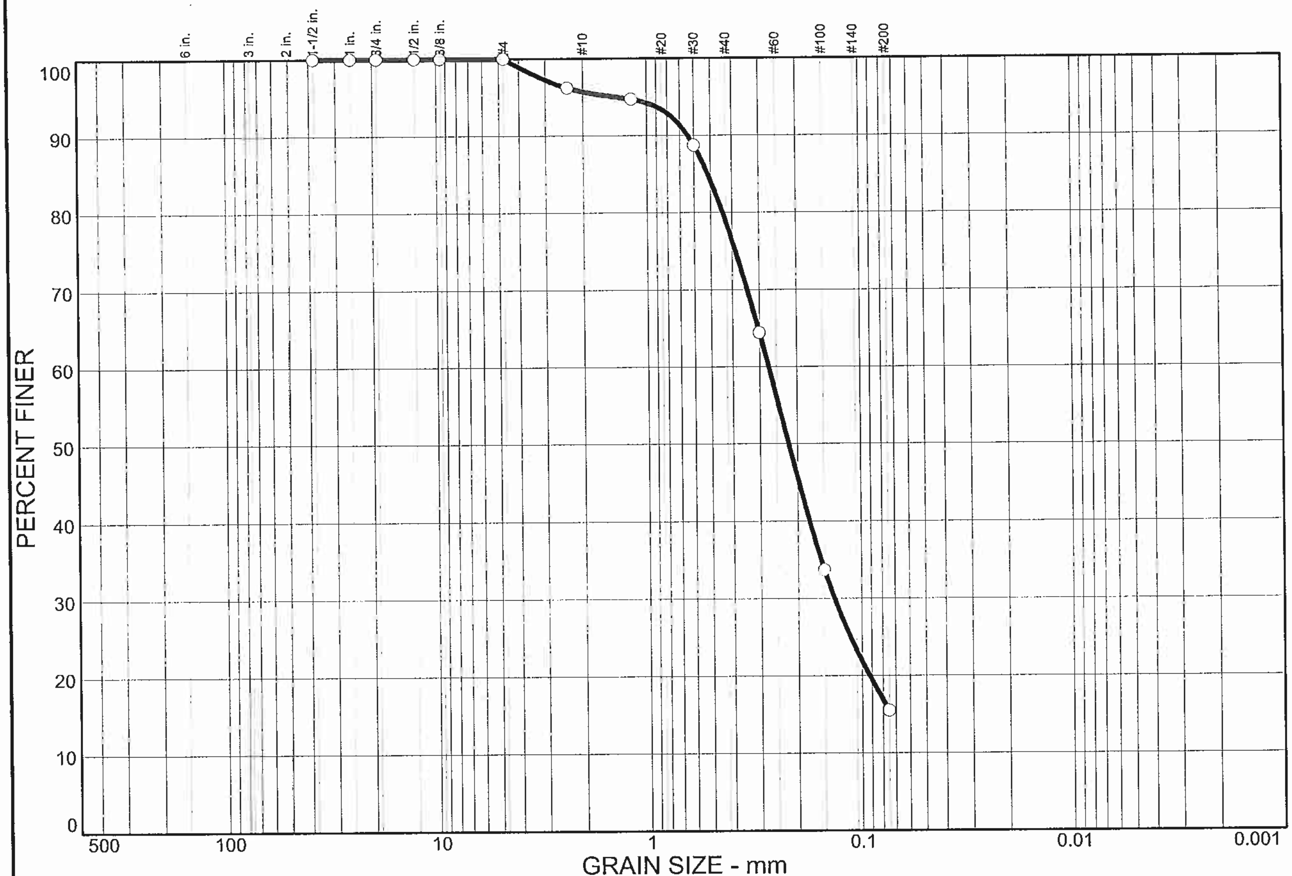
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	84.5	15.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	100.0		
3/4 in.	100.0		
1/2 in.	100.0		
3/8 in.	100.0		
#4	100.0		
#8	96.2		
#16	94.7		
#30	88.7		
#50	64.4		
#100	33.7		
#200	15.5		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.517 D_{60}= 0.273 D_{50}= 0.220
D_{30}= 0.135 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

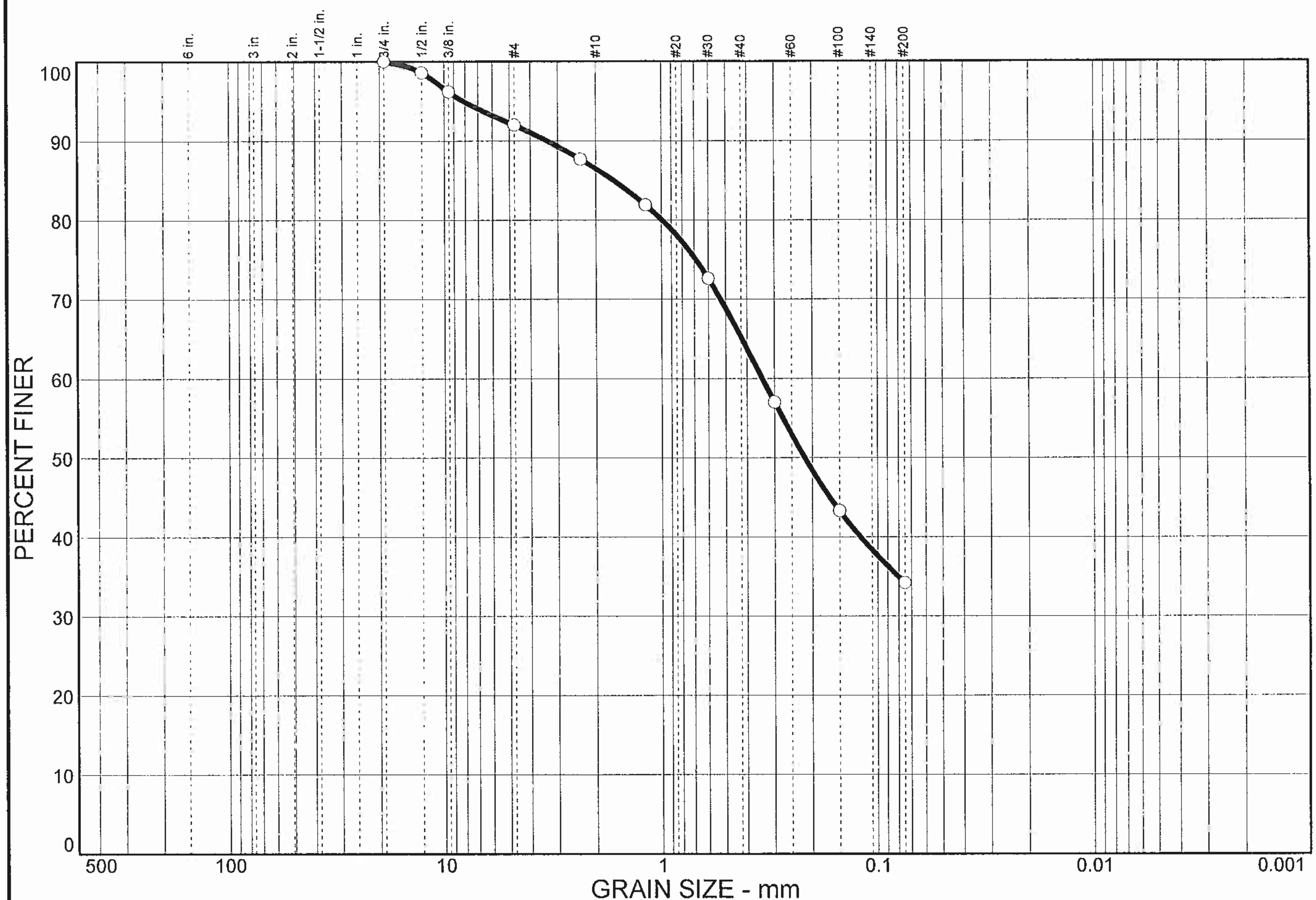
Sample No.: A-2 **Source of Sample:** **Date:**
Location: **Elev./Depth:** 40-41.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	8.0	57.8	34.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	98.6		
3/8 in.	96.2		
#4	92.0		
#8	87.7		
#16	81.9		
#30	72.6		
#50	57.0		
#100	43.3		
#200	34.2		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= 15 LL= 31 PI= 16

Coefficients

D_{85}= 1.66 D_{60}= 0.341 D_{50}= 0.217
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

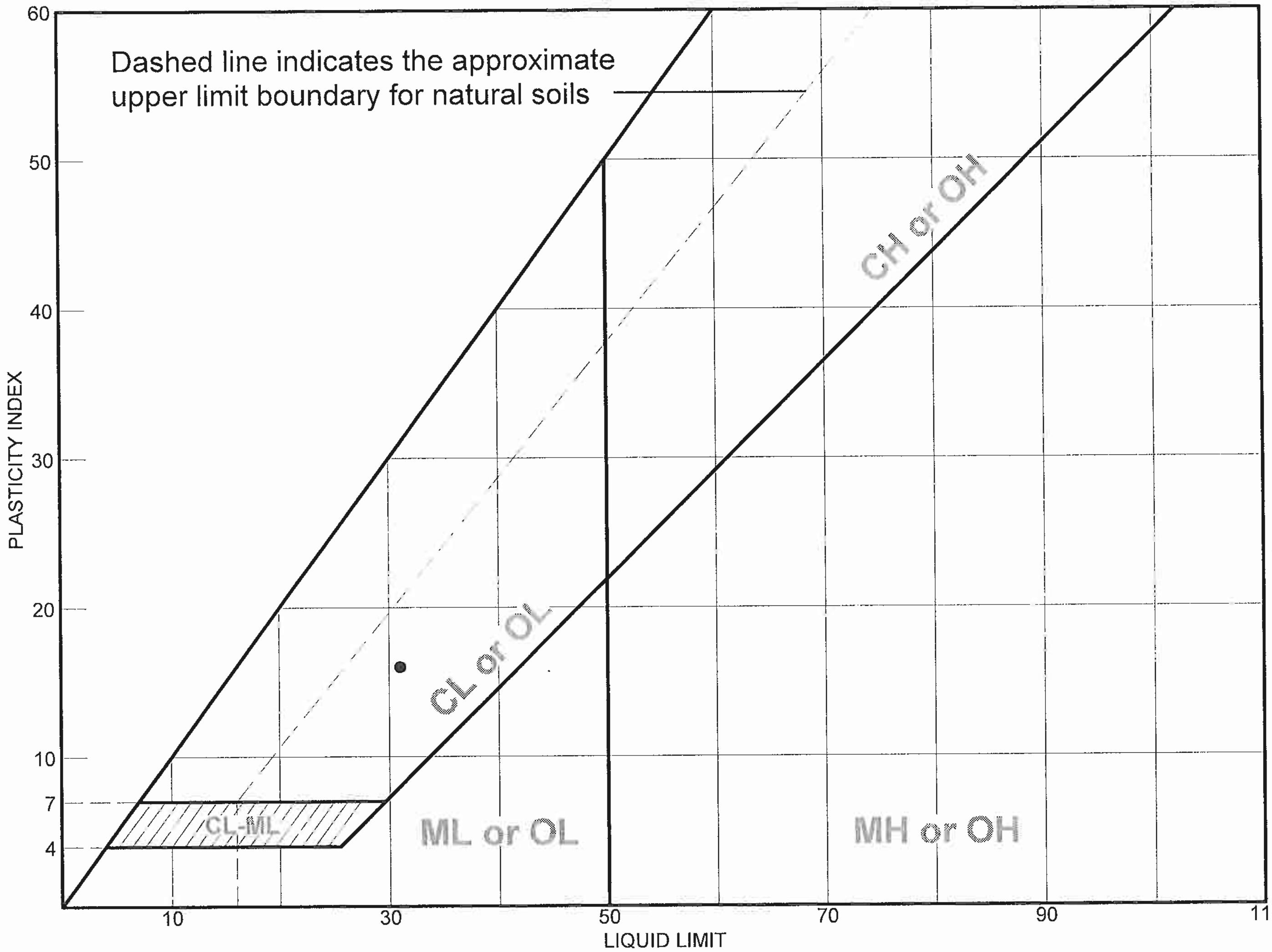
Sample No.: B-7 **Source of Sample:** **Date:** 2/27/19
Location: **Elev./Depth:** 0.8-5.0'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand	31	15	16	65.2	34.2	SC

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** B-7 **Elev./Depth:** 0.8-5.0'

Remarks:

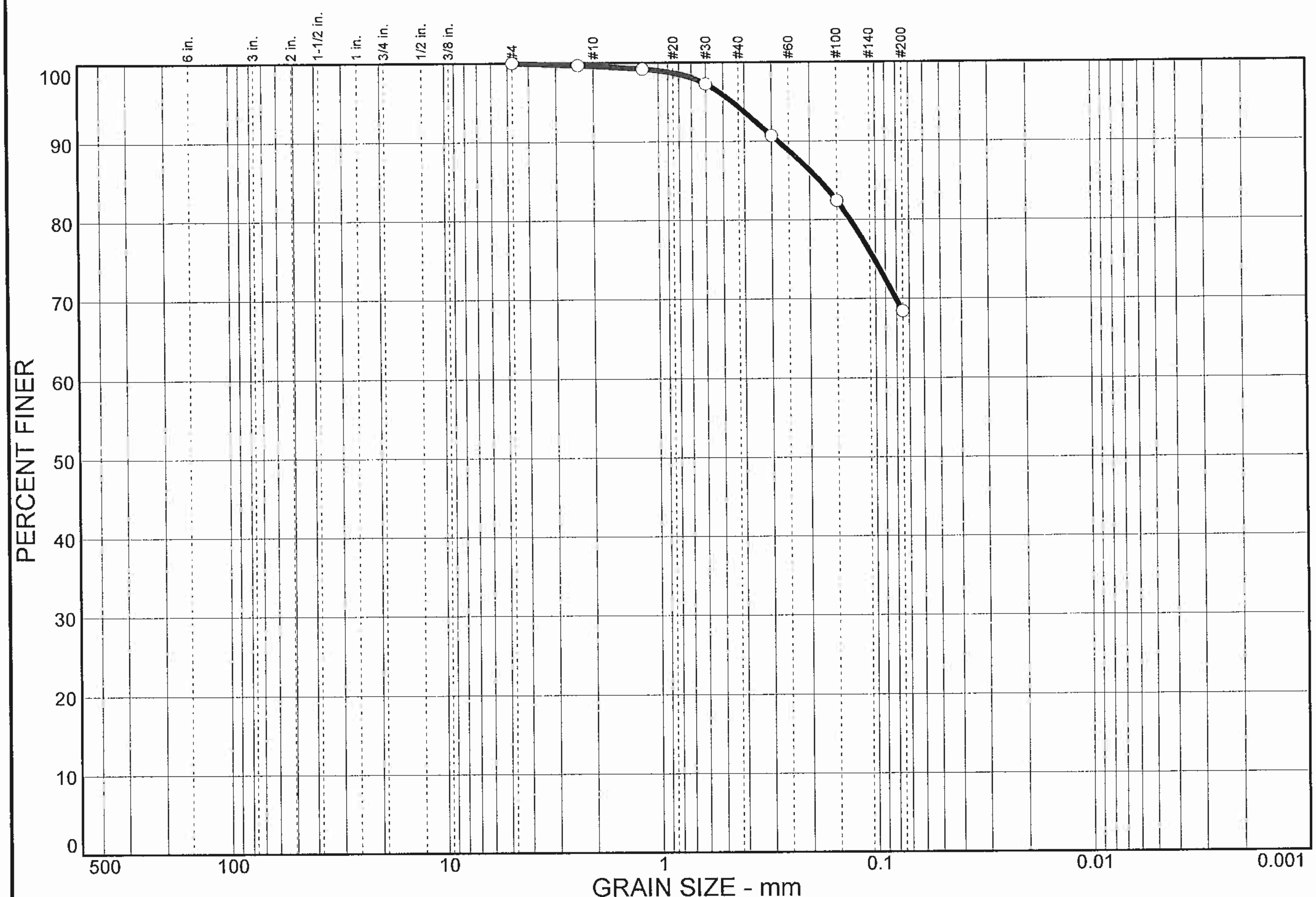
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	31.6	68.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.7		
#16	99.2		
#30	97.3		
#50	90.7		
#100	82.4		
#200	68.4		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.180 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

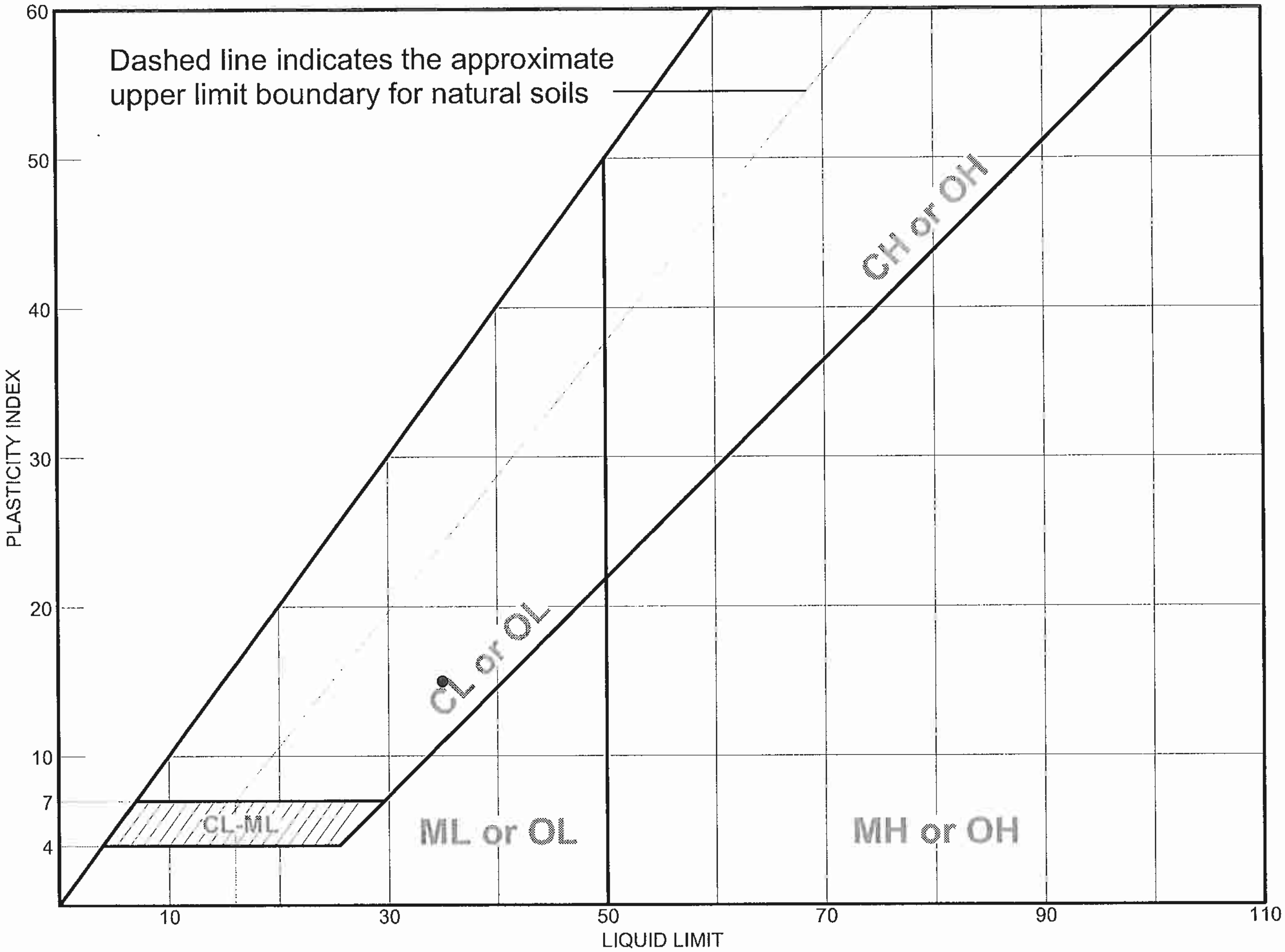
Remarks

Sample No.: E-4 **Source of Sample:** **Date:** 3/4/19
Location: **Elev./Depth:** 10-11.5'

Moore Twining Associates, Inc.
Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	35	20	15	94.4	68.4	CL

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** E-4 **Elev./Depth:** 10-11.5'

Remarks:

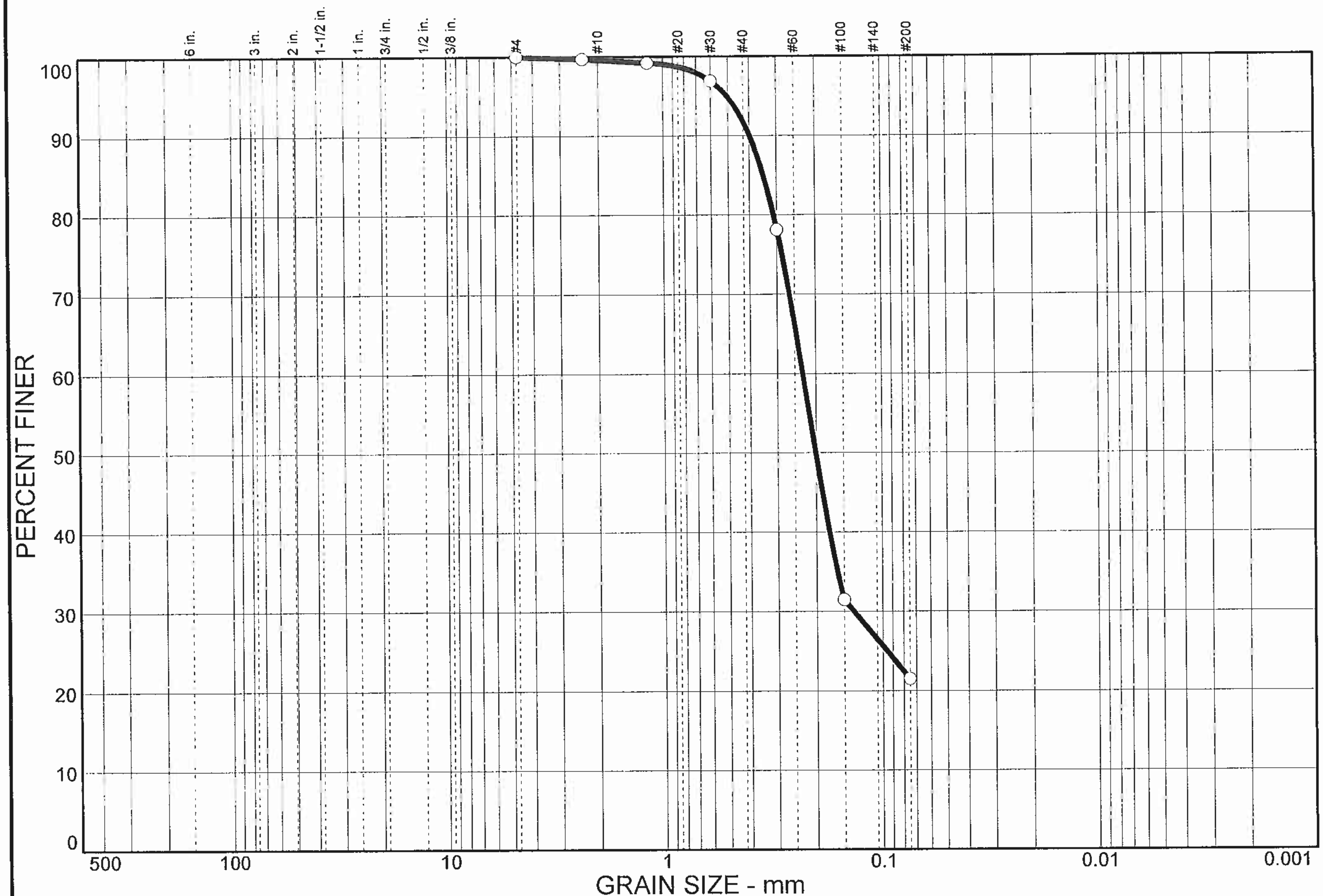
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	78.5	21.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.7		
#16	99.2		
#30	96.9		
#50	78.1		
#100	31.5		
#200	21.5		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.345 D_{60}= 0.230 D_{50}= 0.201
D_{30}= 0.135 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

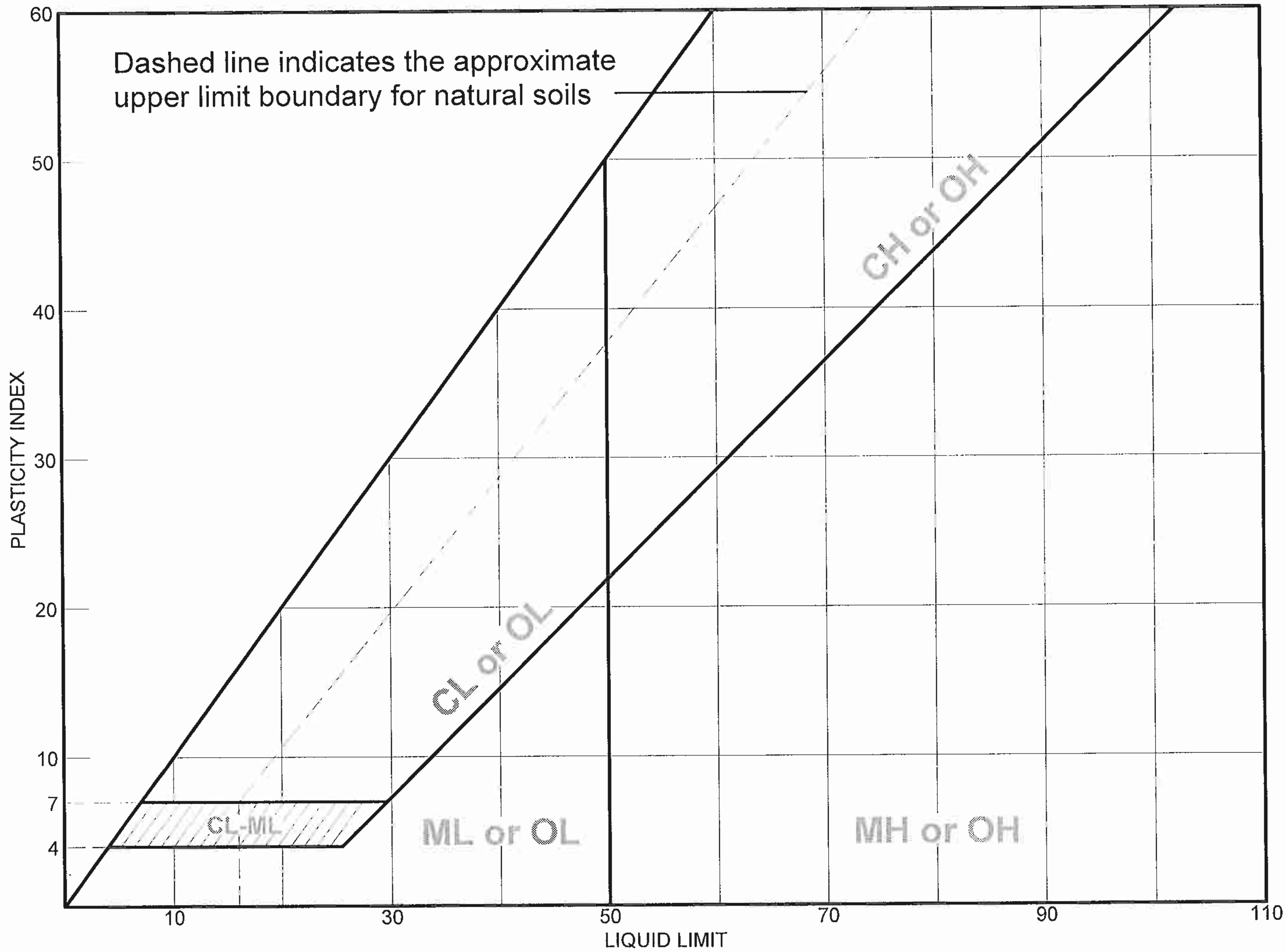
Sample No.: E-6 **Source of Sample:** **Date:** 2/25/19
Location: **Elev./Depth:** 0.9-5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty sand	NV	NP	NP	91.7	21.5	SM

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** E-6 **Elev./Depth:** 0.9-5'

Remarks:

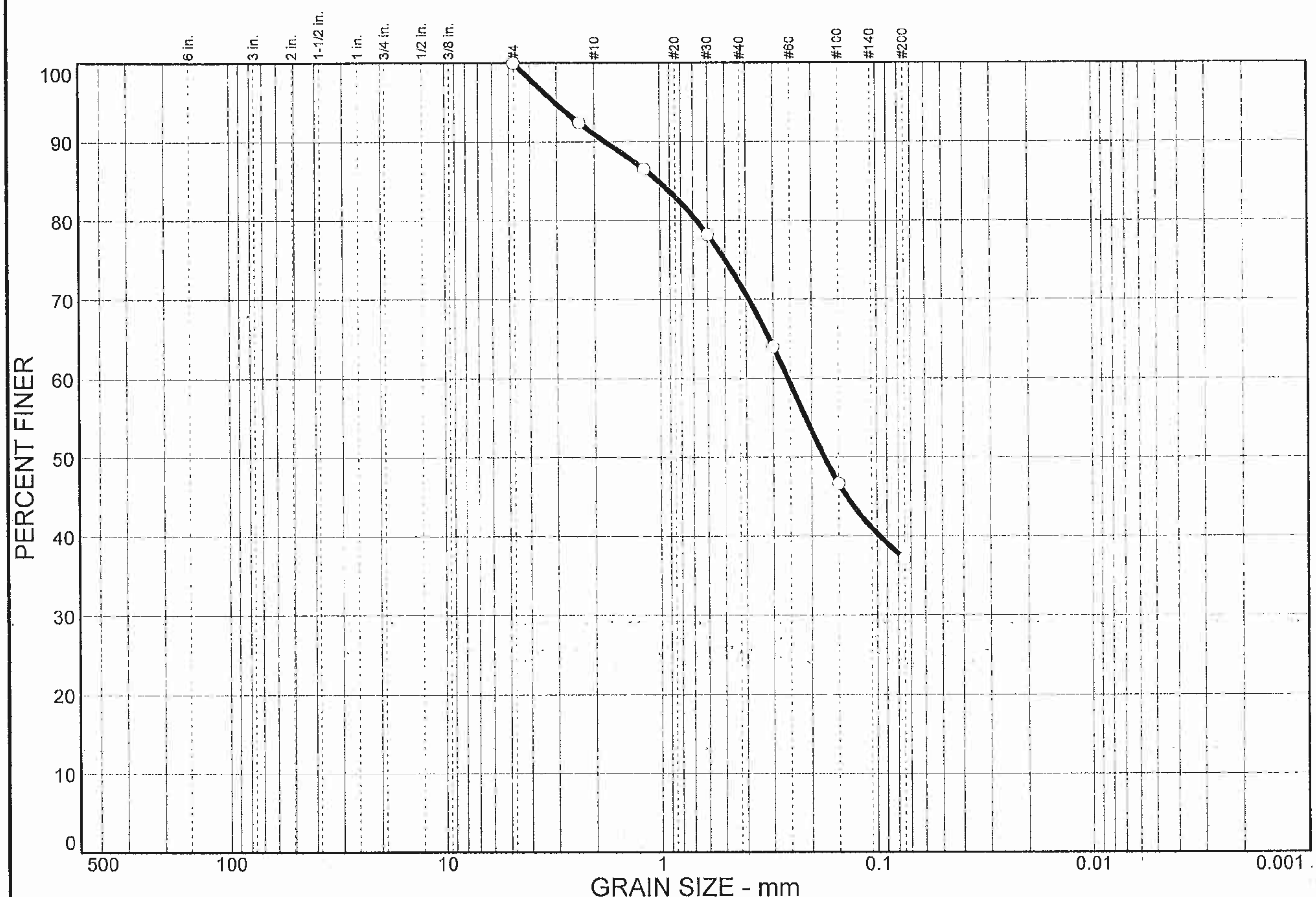
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	62.9	37.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	92.4		
#16	86.5		
#30	78.2		
#50	64.0		
#100	46.7		
#200	37.1		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= 15 LL= 27 PI= 12

Coefficients

D_{85}= 1.01 D_{60}= 0.257 D_{50}= 0.174
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

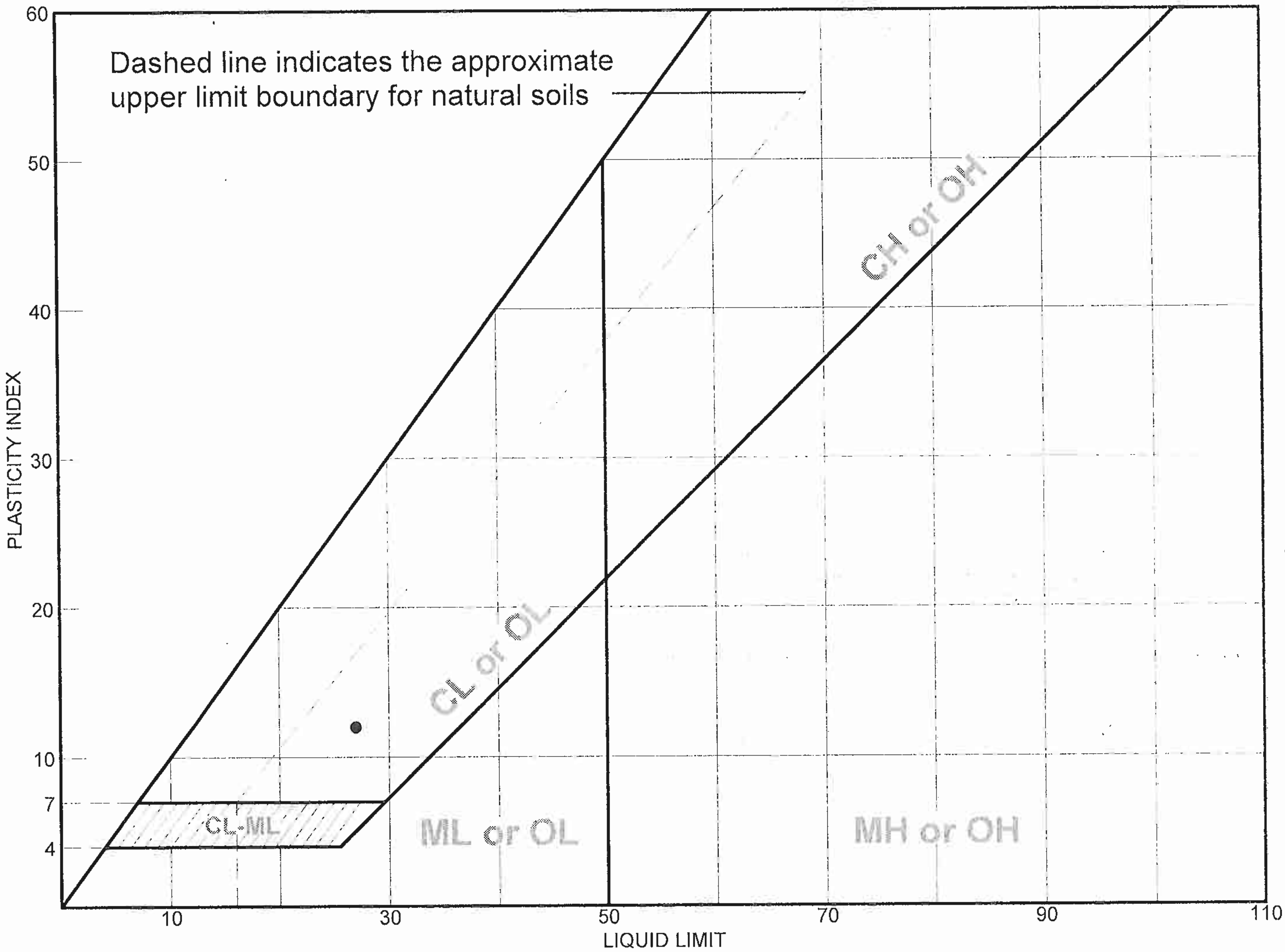
Sample No.: E-8 **Source of Sample:** **Date:** 9/17/19
Location: **Elev./Depth:** 1-3'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand	27	15	12	71.9	37.1	SC

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** E-8 **Elev./Depth:** 1-3'

Remarks:

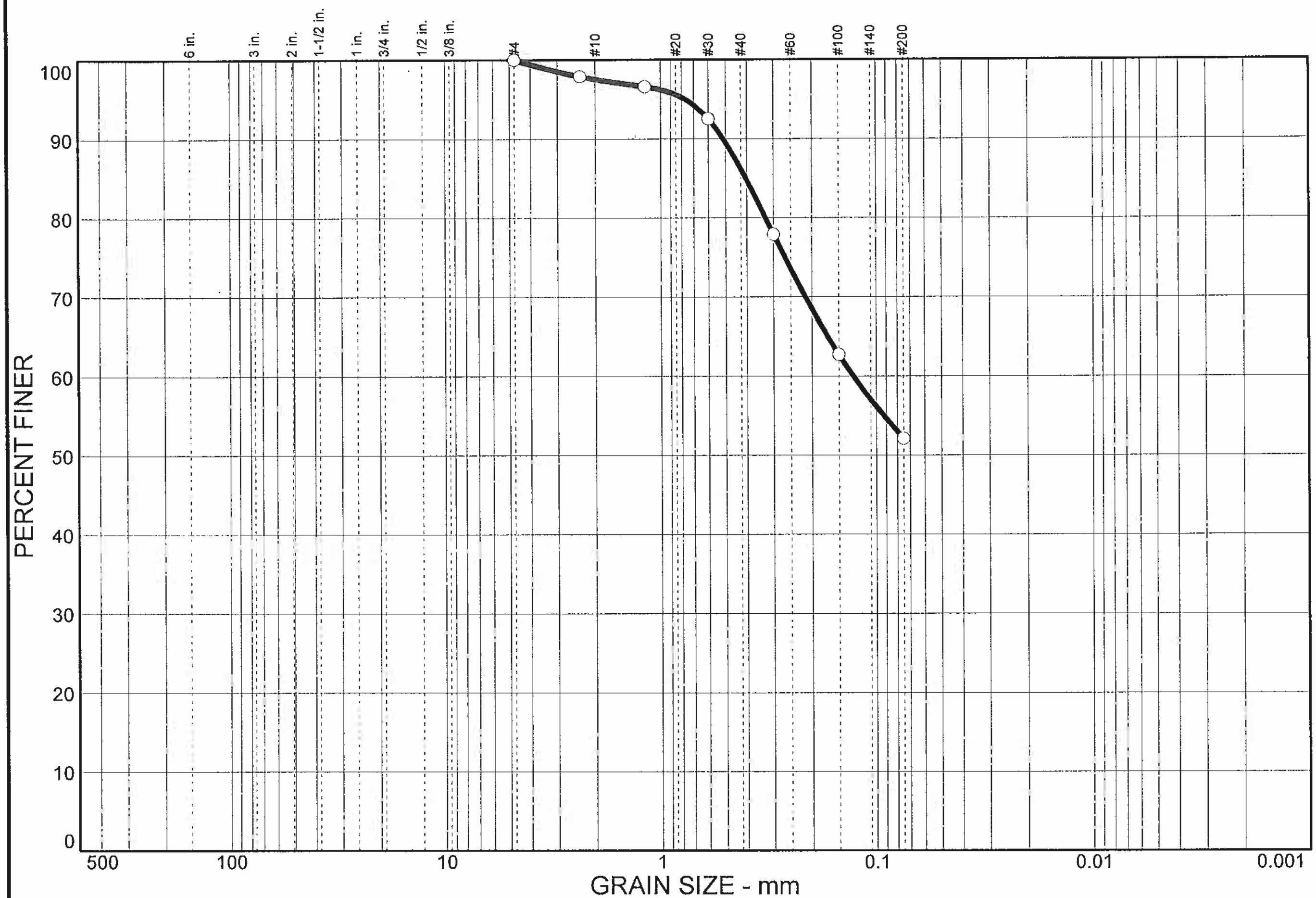
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	47.9	52.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	97.9		
#16	96.6		
#30	92.5		
#50	77.9		
#100	62.7		
#200	52.1		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= 23 LL= 36 PI= 13

Coefficients

D_{85}= 0.404 D_{60}= 0.129 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

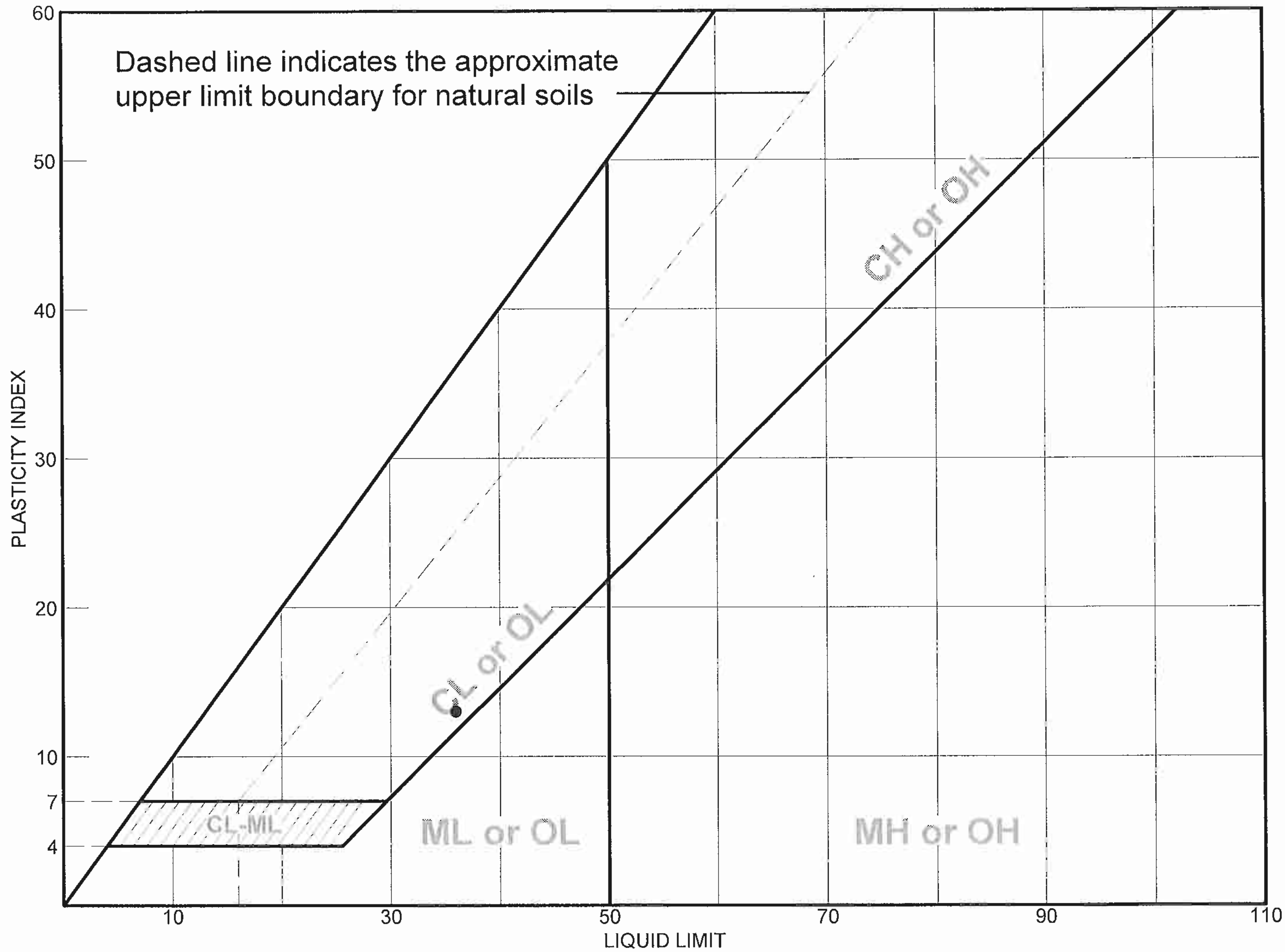
Remarks

Sample No.: G-5 **Source of Sample:** **Date:** 2/25/19
Location: **Elev./Depth:** 2.5-4'

Moore Twining Associates, Inc.
Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	36	23	13	86.1	52.1	CL

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** G-5 **Elev./Depth:** 2.5-4'

Remarks:

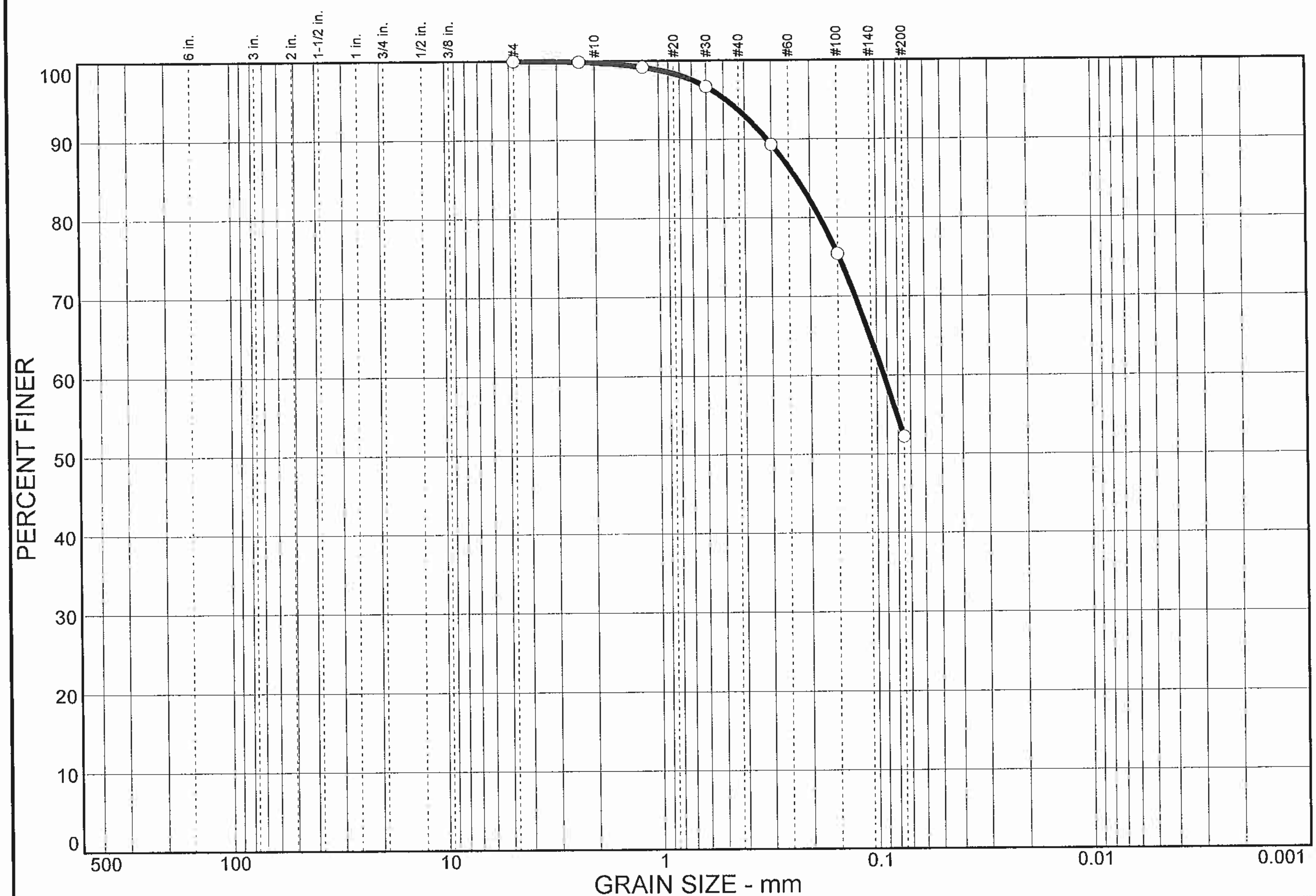
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	47.8	52.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#16	99.2		
#30	96.7		
#50	89.3		
#100	75.4		
#200	52.2		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= 23 LL= 36 PI= 13

Coefficients

D_{85}= 0.230 D_{60}= 0.0930 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

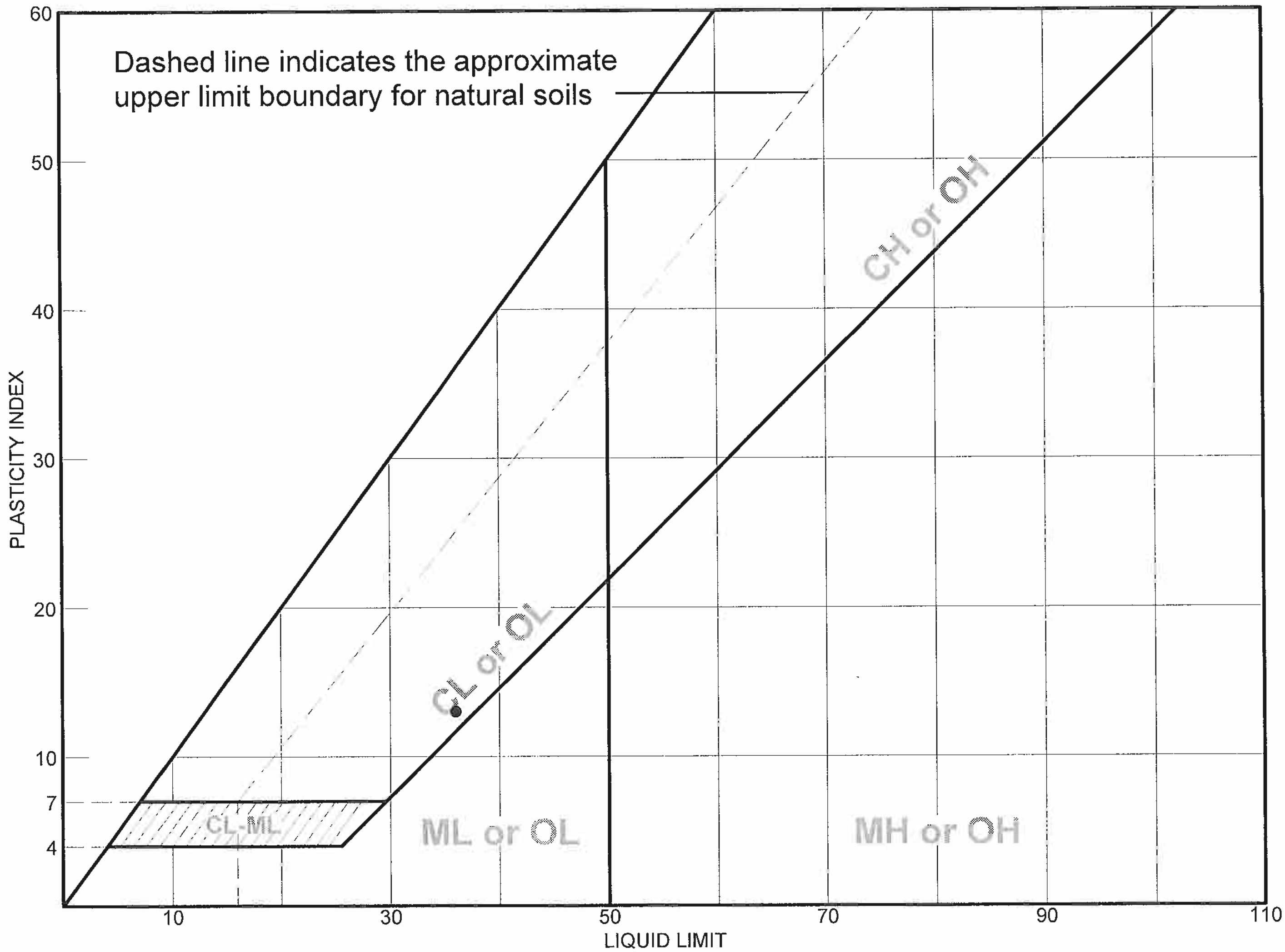
Sample No.: G-6 **Source of Sample:** **Date:** 2/25/19
Location: **Elev./Depth:** 1-2.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	36	23	13	93.7	52.2	CL

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** G-6 **Elev./Depth:** 1-2.5'

Remarks:

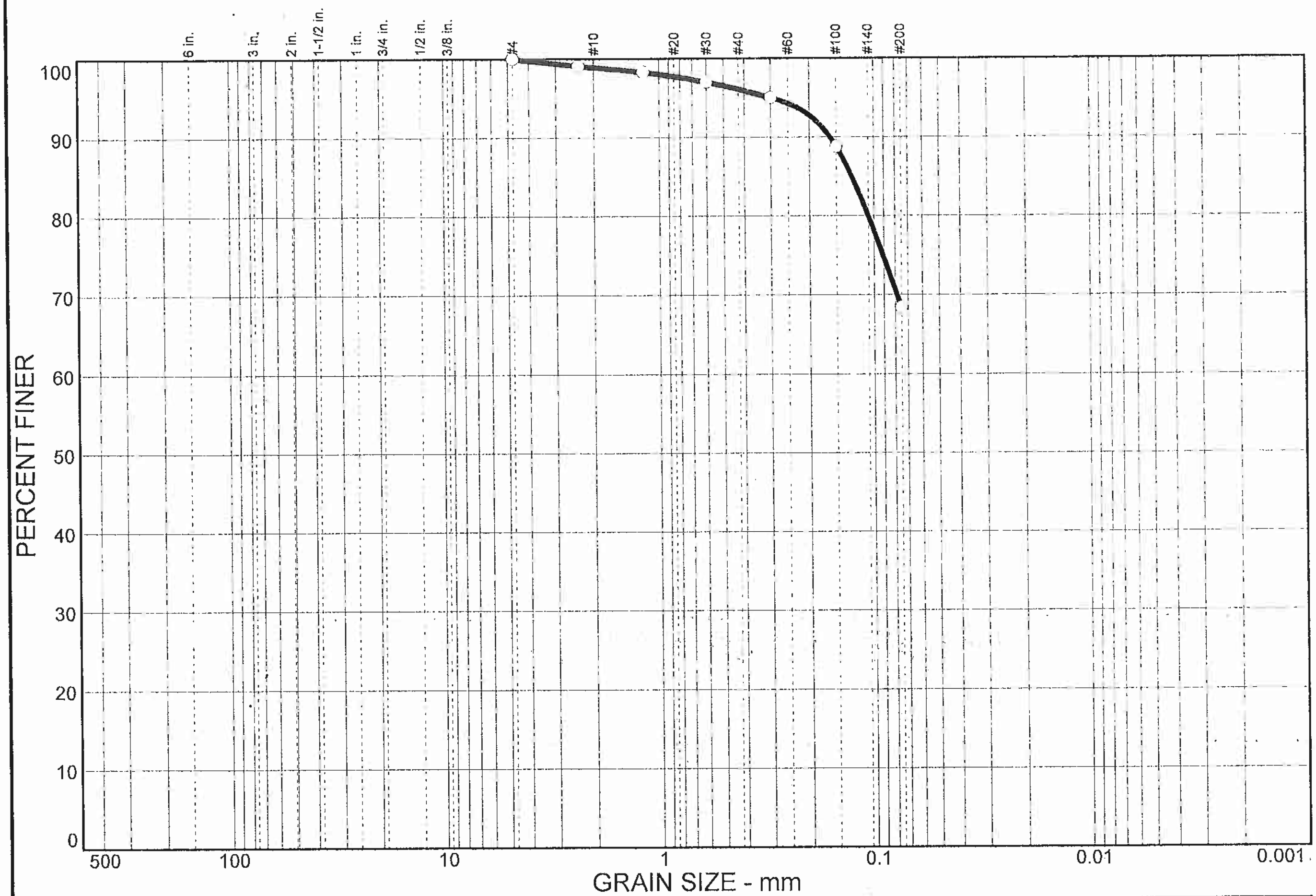
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	31.6	68.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.1		
#16	98.3		
#30	97.0		
#50	95.0		
#100	88.8		
#200	68.4		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= 18 LL= 30 PI= 12

Coefficients

D_{85}= 0.127 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

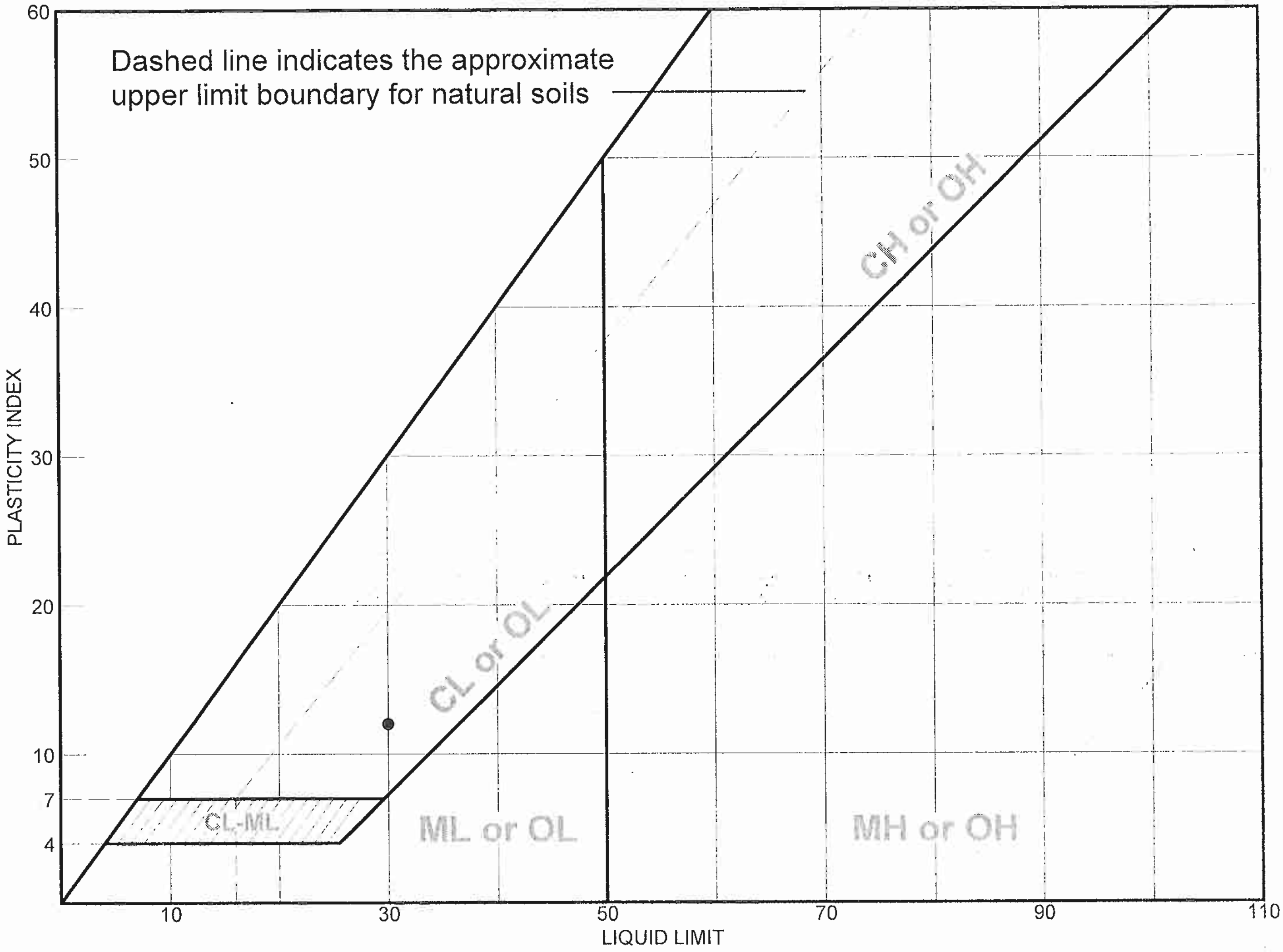
Sample No.: G-7.6 **Source of Sample:** **Date:** 9/17/19
Location: **Elev./Depth:** 1-5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	30	18	12	96.1	68.4	CL

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** G-7.6 **Elev./Depth:** 1-5'

Remarks:

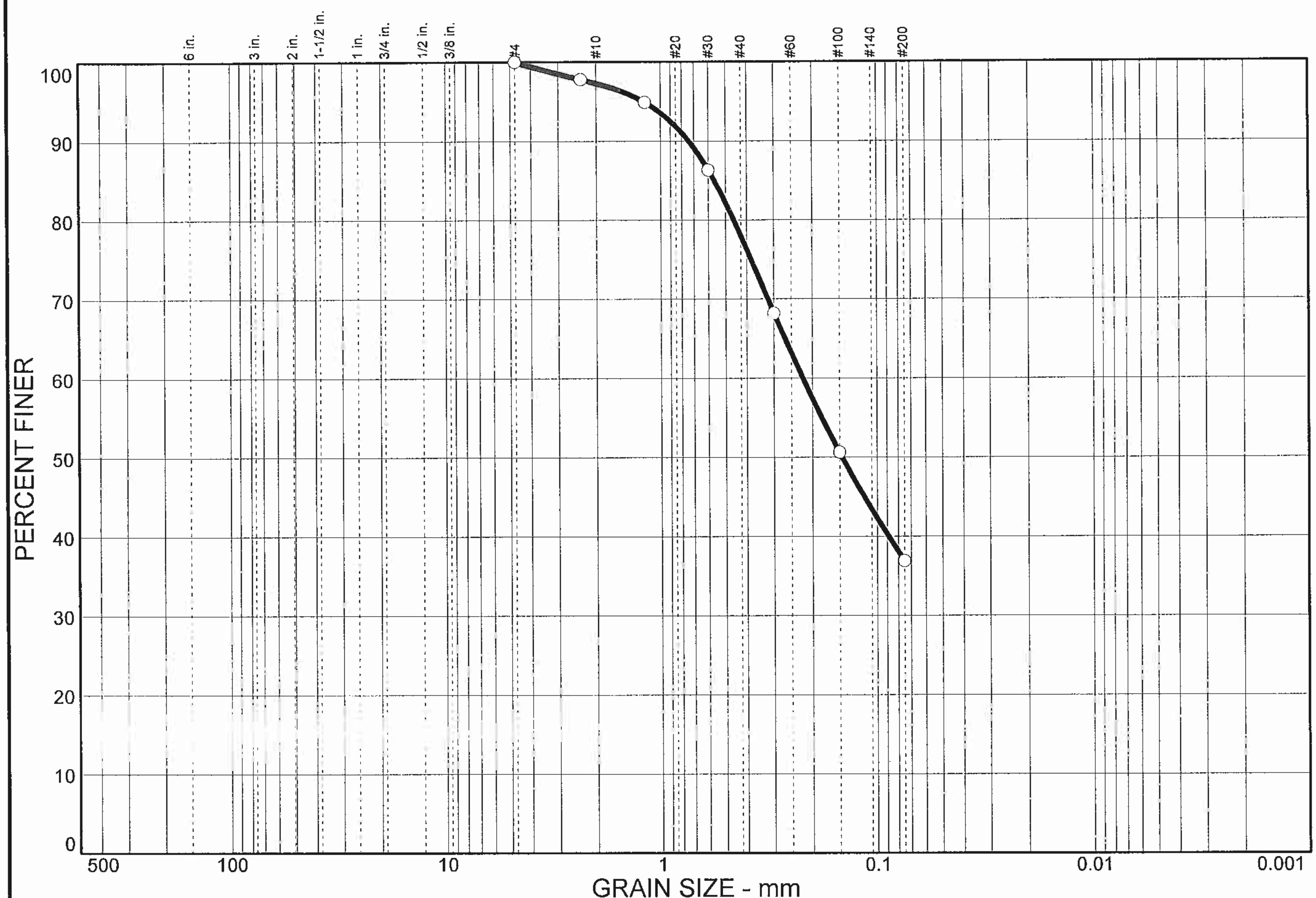
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	63.1	36.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	97.8		
#16	94.9		
#30	86.3		
#50	68.2		
#100	50.6		
#200	36.9		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.564 D_{60}= 0.221 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

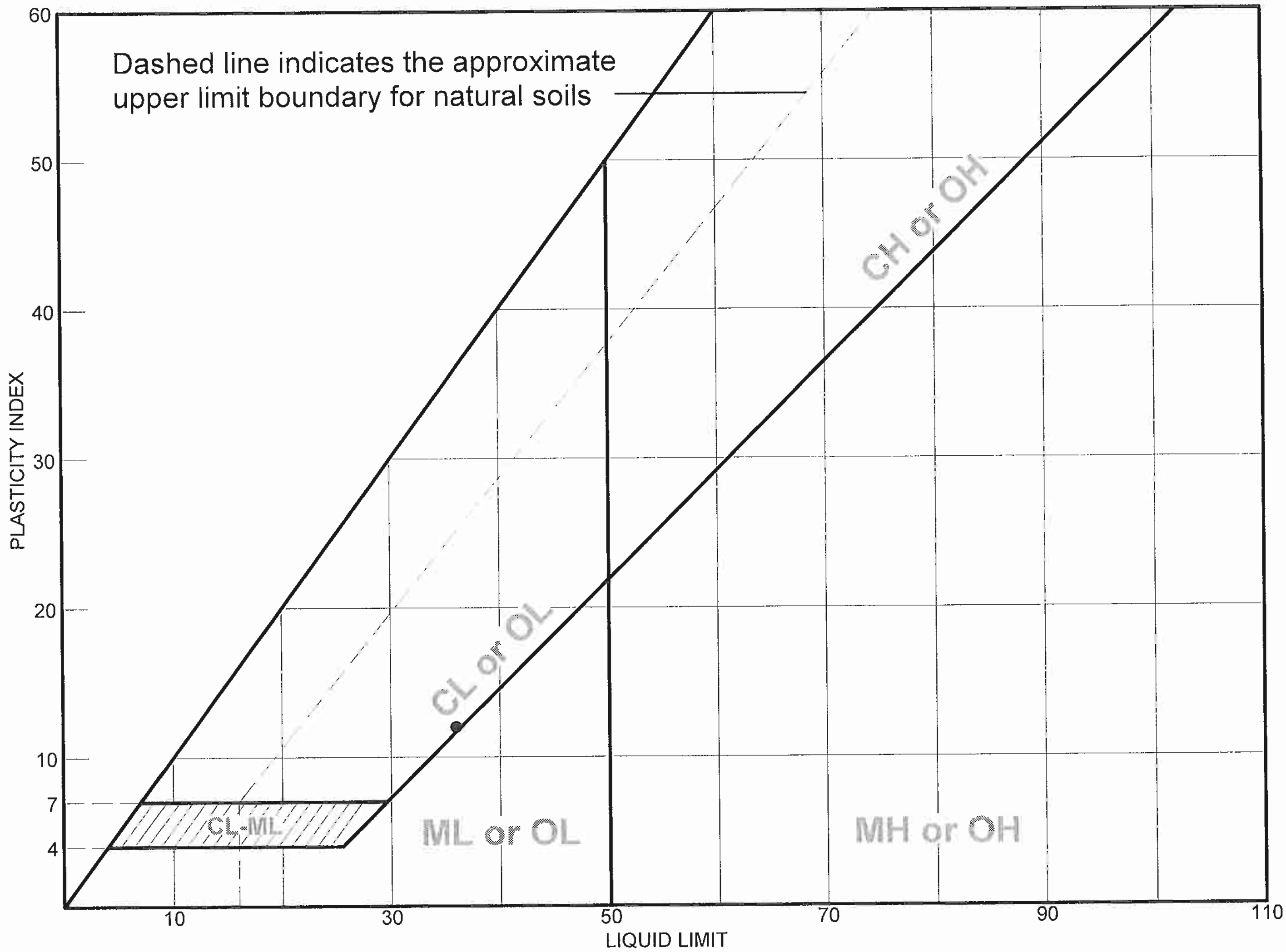
Sample No.: H-6 **Source of Sample:** **Date:** 2/25/19
Location: **Elev./Depth:** 5-6.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	36	24	12			CL

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** H-6 **Elev./Depth:** 1-2.5'

Remarks:

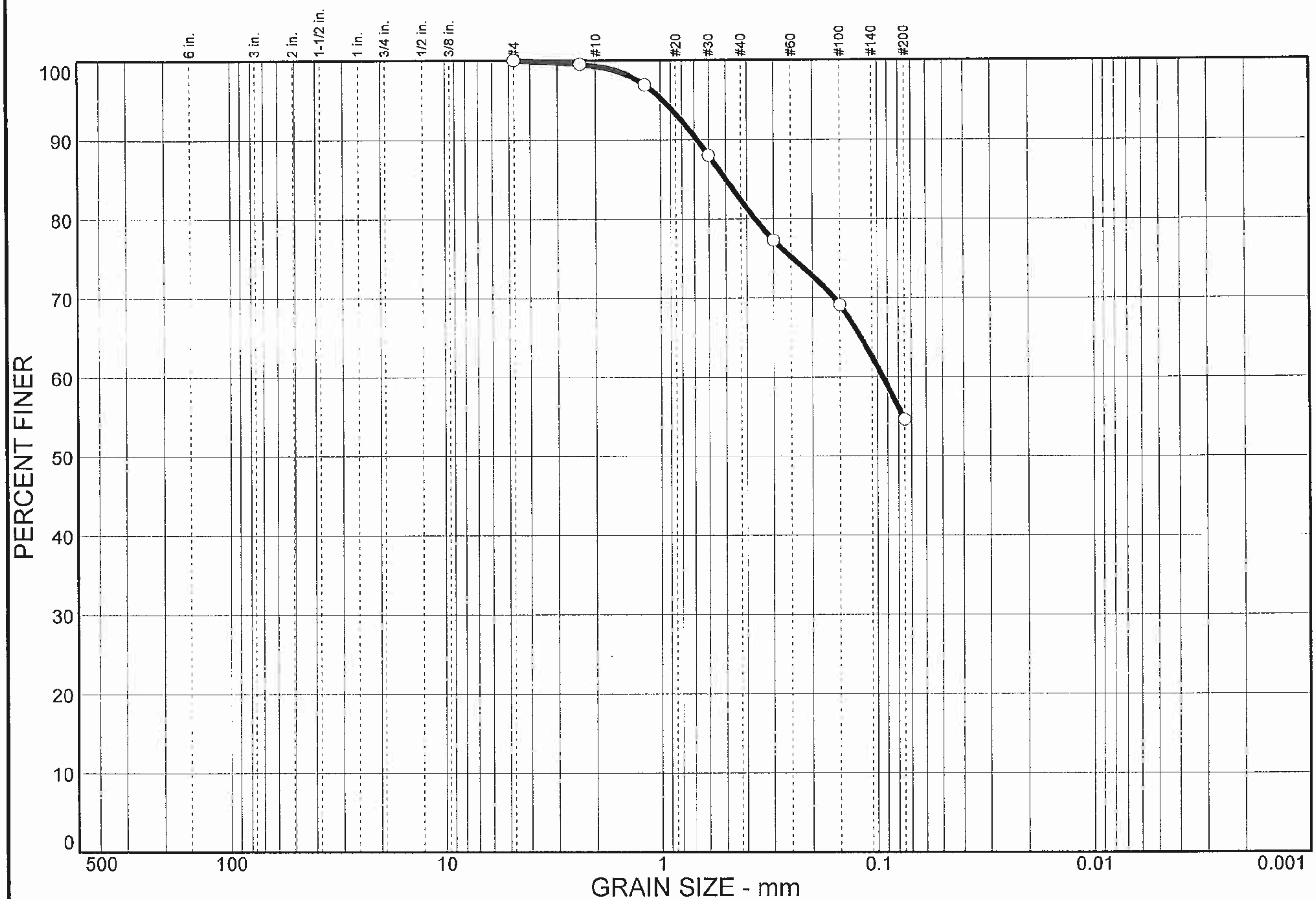
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	45.4	54.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.5		
#16	96.9		
#30	88.0		
#50	77.3		
#100	69.1		
#200	54.6		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.500 D_{60}= 0.0945 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample No.: H-8 **Source of Sample:** **Date:** 2/25/19
Location: **Elev./Depth:** 1-5'

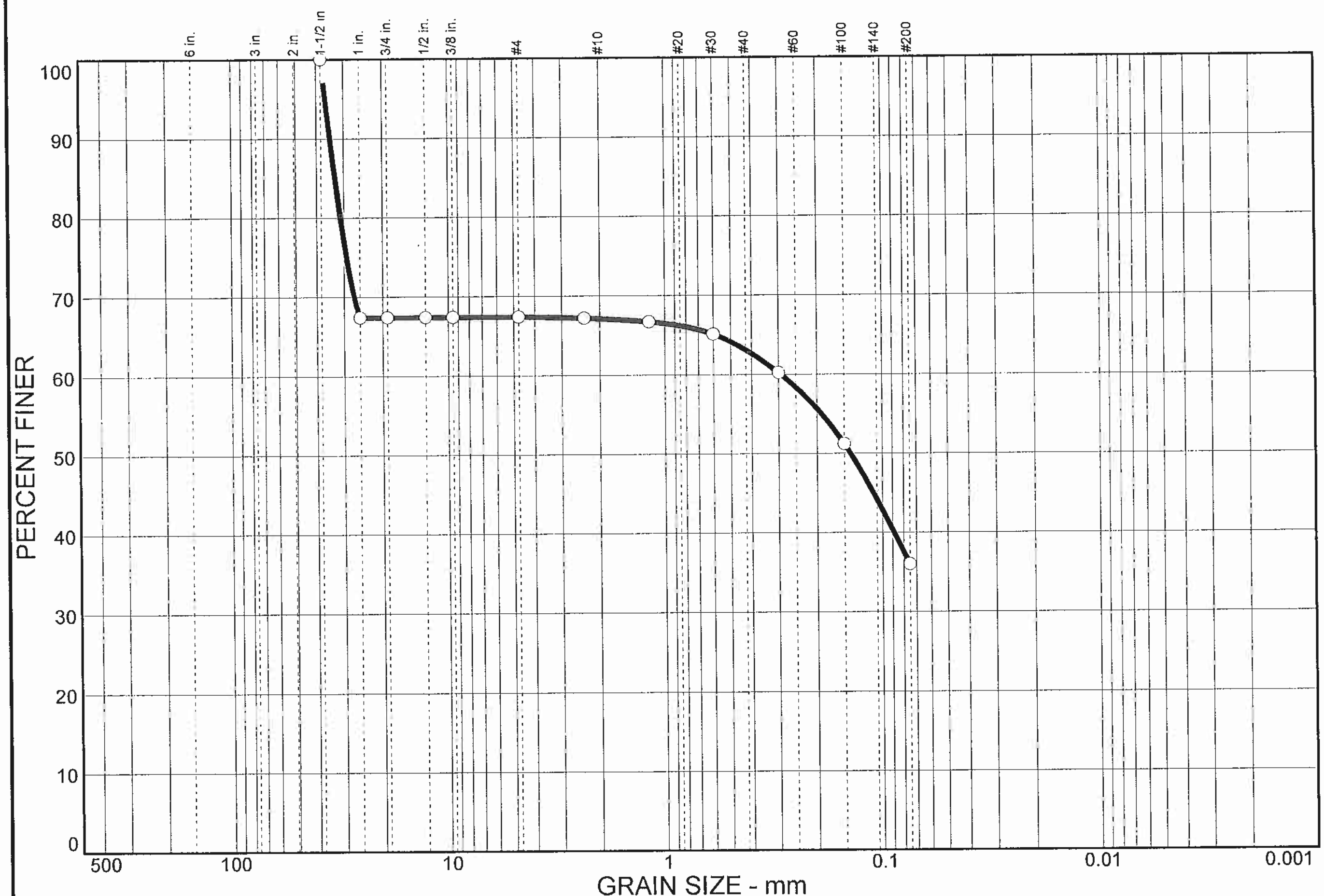
Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot

Project No: D050R0.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	32.6	31.4	36.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	67.4		
3/4 in.	67.4		
1/2 in.	67.4		
3/8 in.	67.4		
#4	67.4		
#8	67.2		
#16	66.7		
#30	65.1		
#50	60.2		
#100	51.2		
#200	36.0		

* (no specification provided)

Material Description

Clayey gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 32.9 D_{60}= 0.294 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= GC AASHTO=

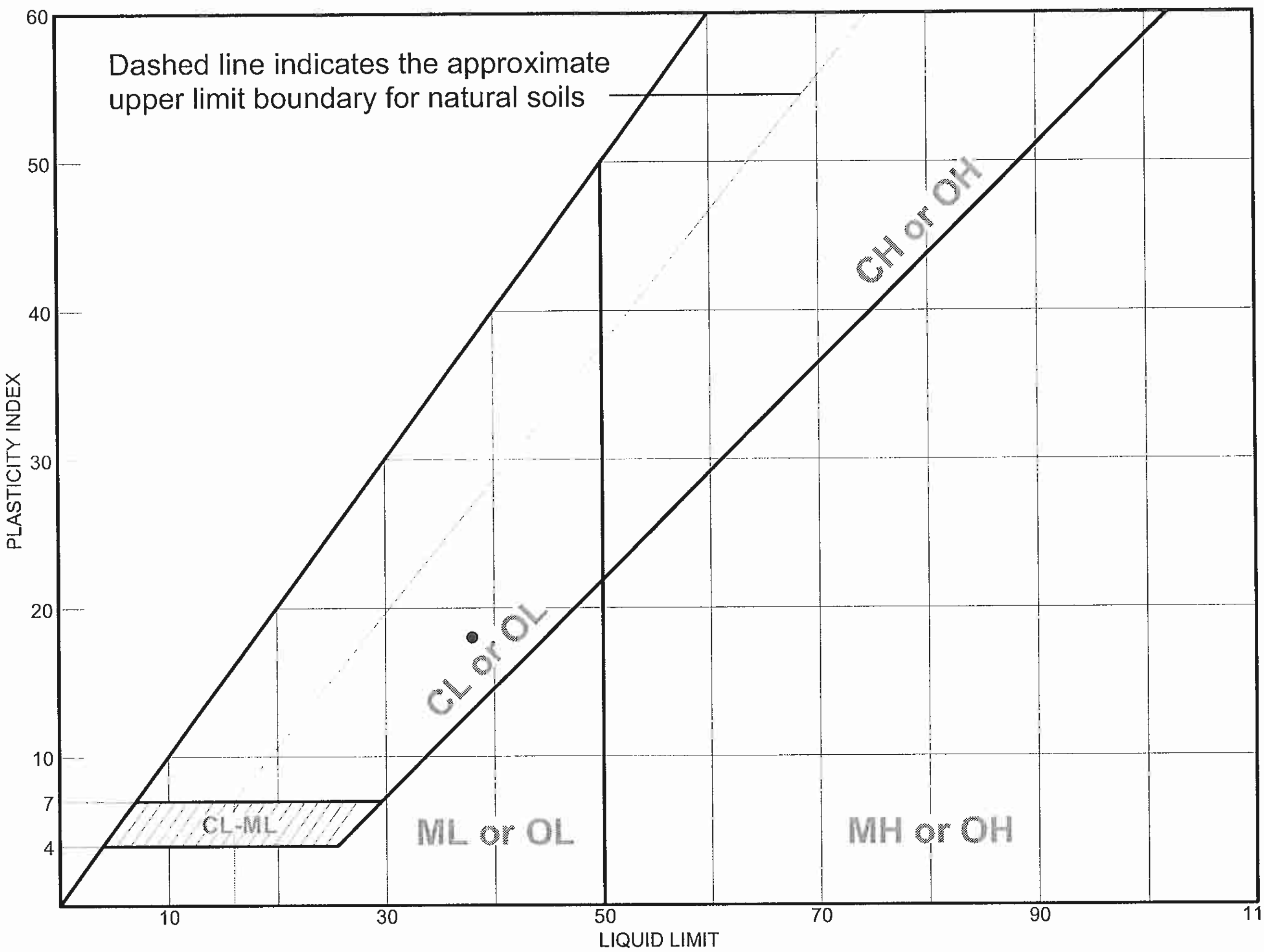
Remarks

Sample No.: J-4 **Source of Sample:** **Date:** 3/4/19
Location: **Elev./Depth:** 5-6.5'

Moore Twining Associates, Inc.
Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey gravel with sand	38	20	18	63.1	36.0	GC

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** J-4 **Elev./Depth:** 5-6.5'

Remarks:

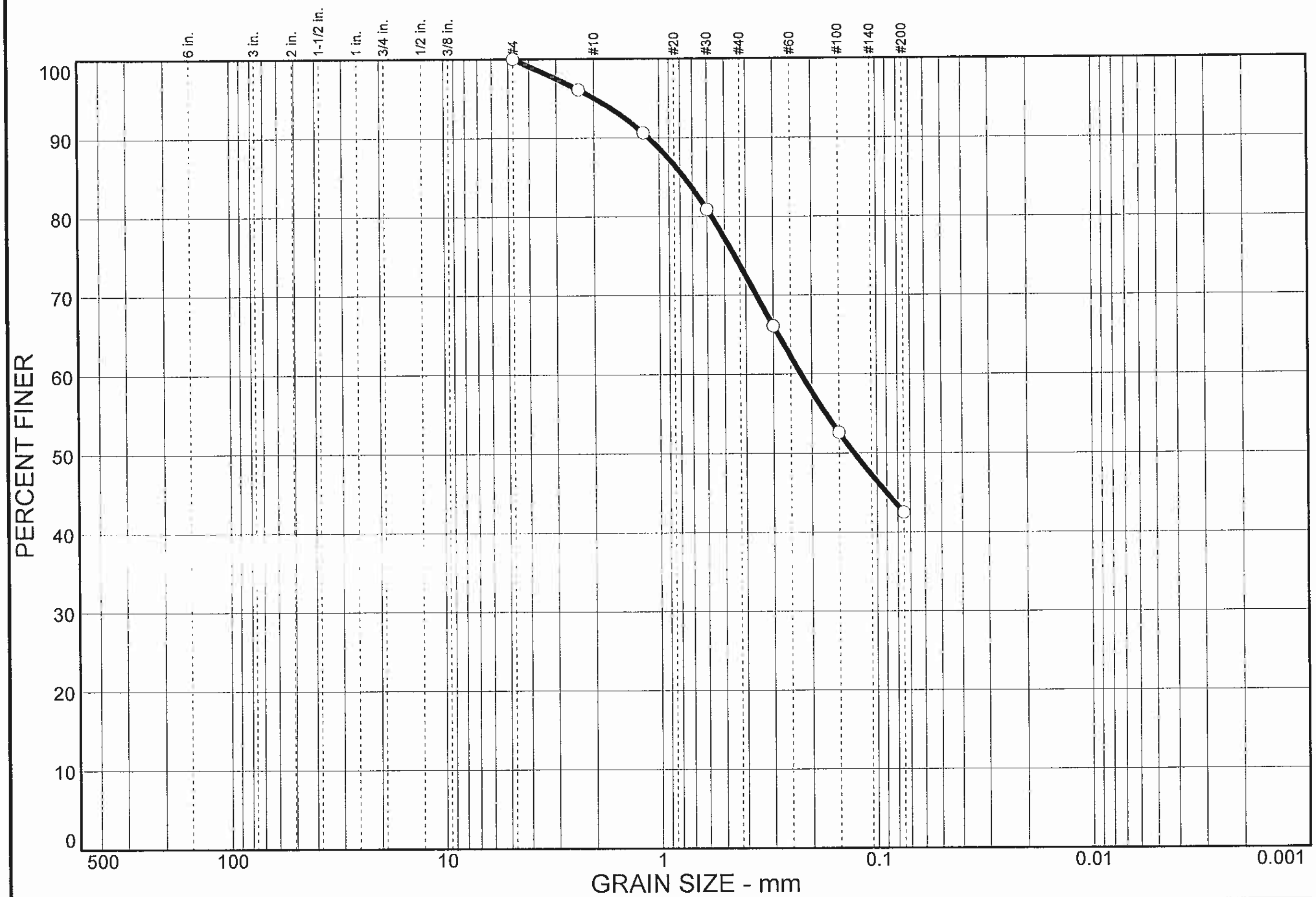
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	57.5	42.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	96.1		
#16	90.6		
#30	80.9		
#50	66.1		
#100	52.6		
#200	42.5		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= 20 LL= 42 PI= 22

Coefficients

D_{85}= 0.764 D_{60}= 0.224 D_{50}= 0.128
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

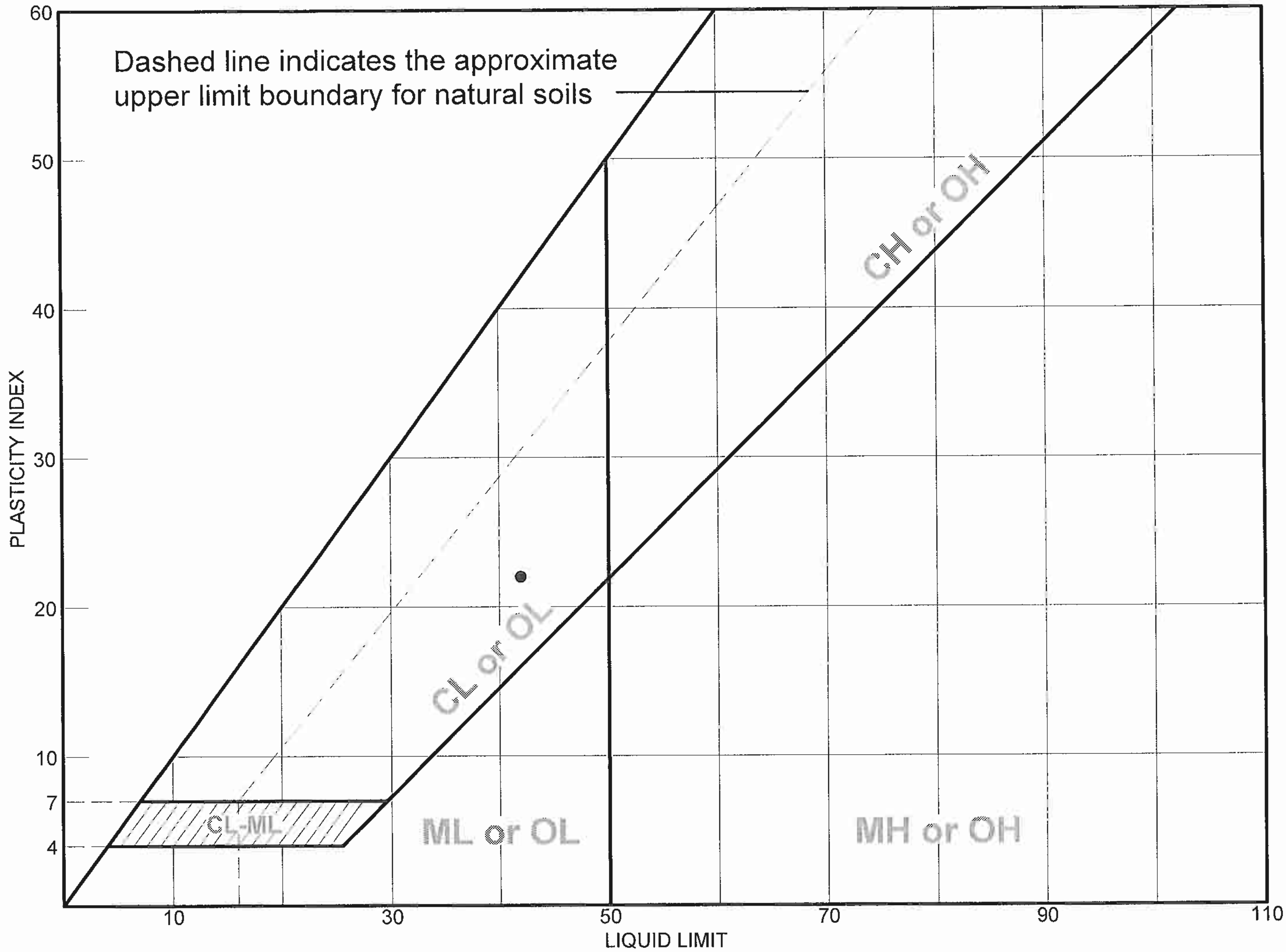
Remarks

Sample No.: J-8 **Source of Sample:** **Date:** 2/25/19
Location: **Elev./Depth:** 0.9-5'

Moore Twining Associates, Inc.
Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand	42	20	22	73.8	42.5	SC

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** J-8 **Elev./Depth:** 0.9-5'

Remarks:

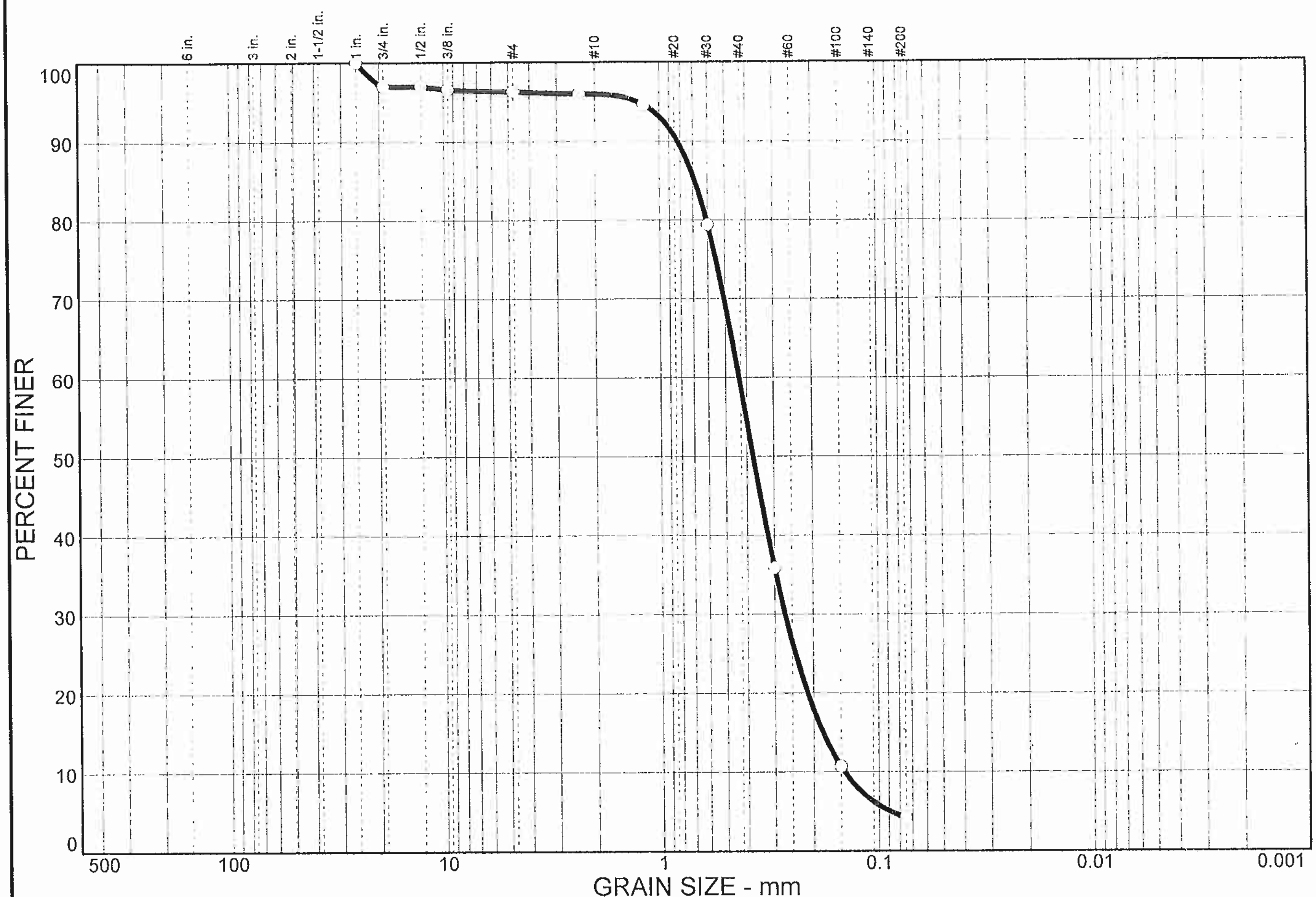
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.7	92.1	4.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	97.0		
1/2 in.	97.0		
3/8 in.	96.6		
#4	96.3		
#8	96.0		
#16	94.6		
#30	79.4		
#50	35.9		
#100	10.7		
#200	4.2		

* (no specification provided)

Material Description

Poorly graded sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.688 D_{60}= 0.434 D_{50}= 0.374
D_{30}= 0.269 D_{15}= 0.181 D_{10}= 0.144
C_u= 3.01 C_c= 1.16

Classification

USCS= SP AASHTO=

Remarks

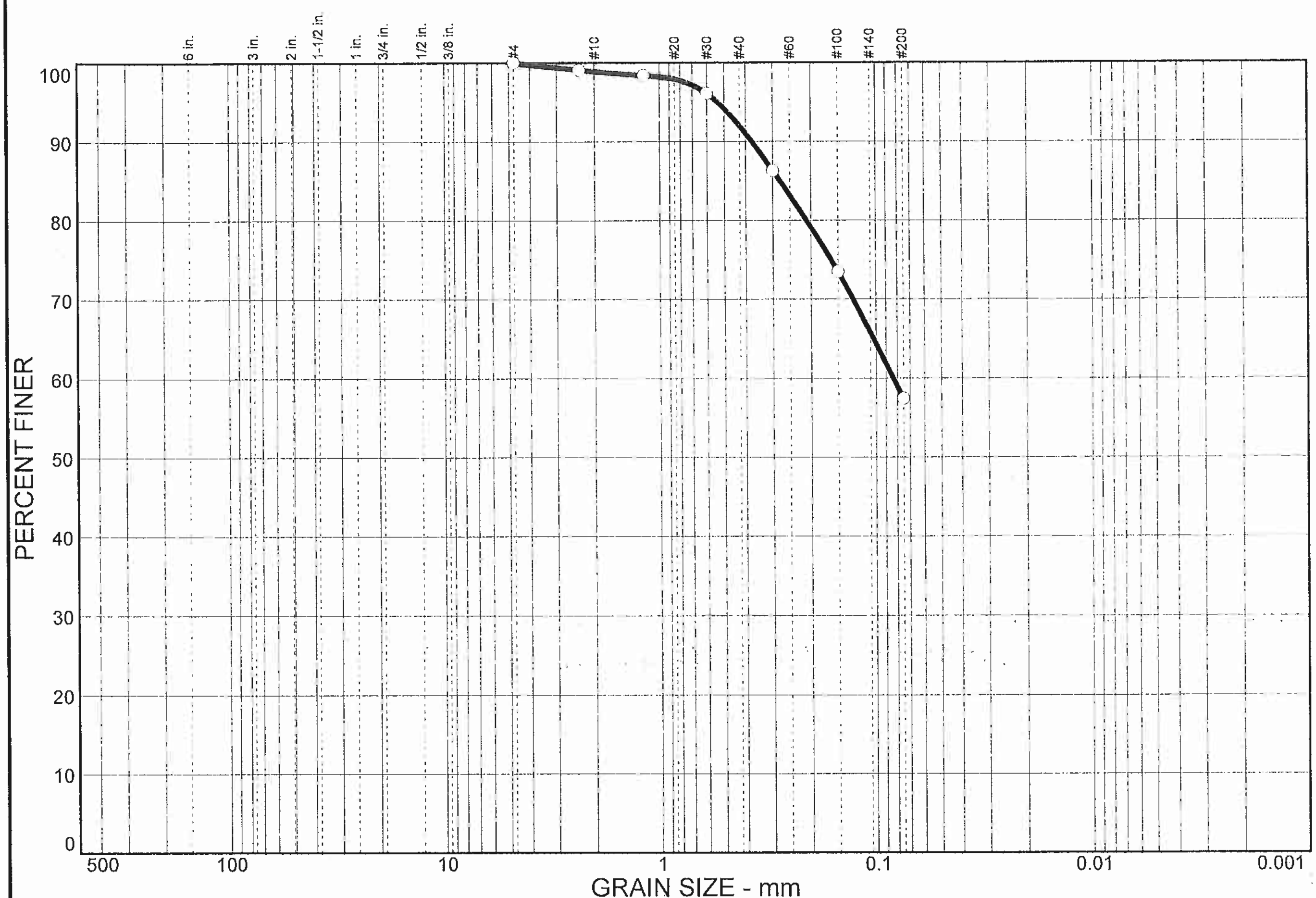
Sample No.: M-8 **Source of Sample:** **Date:** 9/17/19
Location: **Elev./Depth:** 30-31.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	42.6	57.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.1		
#16	98.4		
#30	96.1		
#50	86.3		
#100	73.5		
#200	57.4		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= 17 LL= PI=

Coefficients

D_{85}= 0.278 D_{60}= 0.0834 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

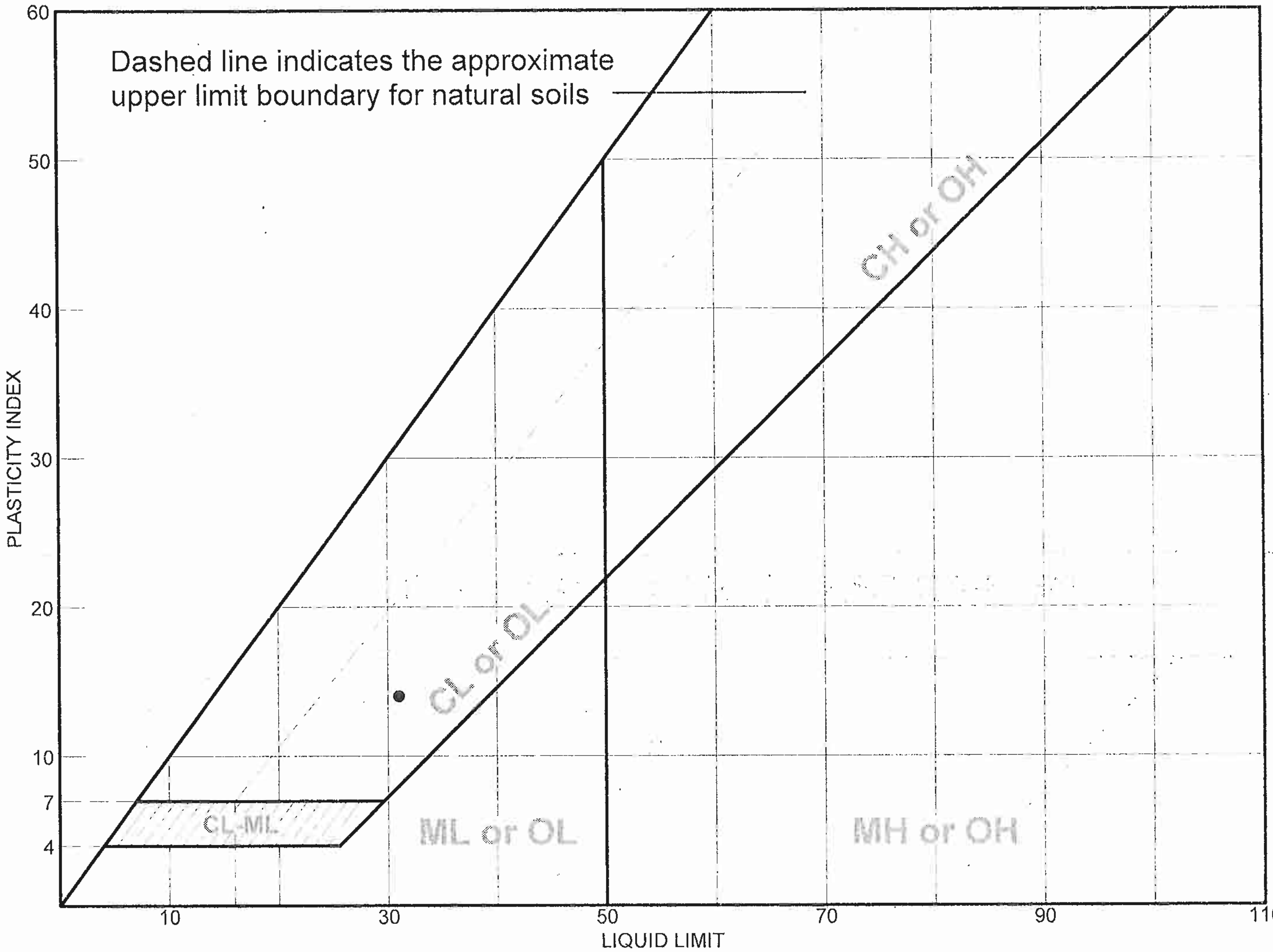
Sample No.: Top Cut Slope (CST) **Source of Sample:** **Date:** 9/17/19
Location: **Elev./Depth:**

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	31	17	14	92.0	57.4	CL

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** Top Cut Slope (CST)

Remarks:

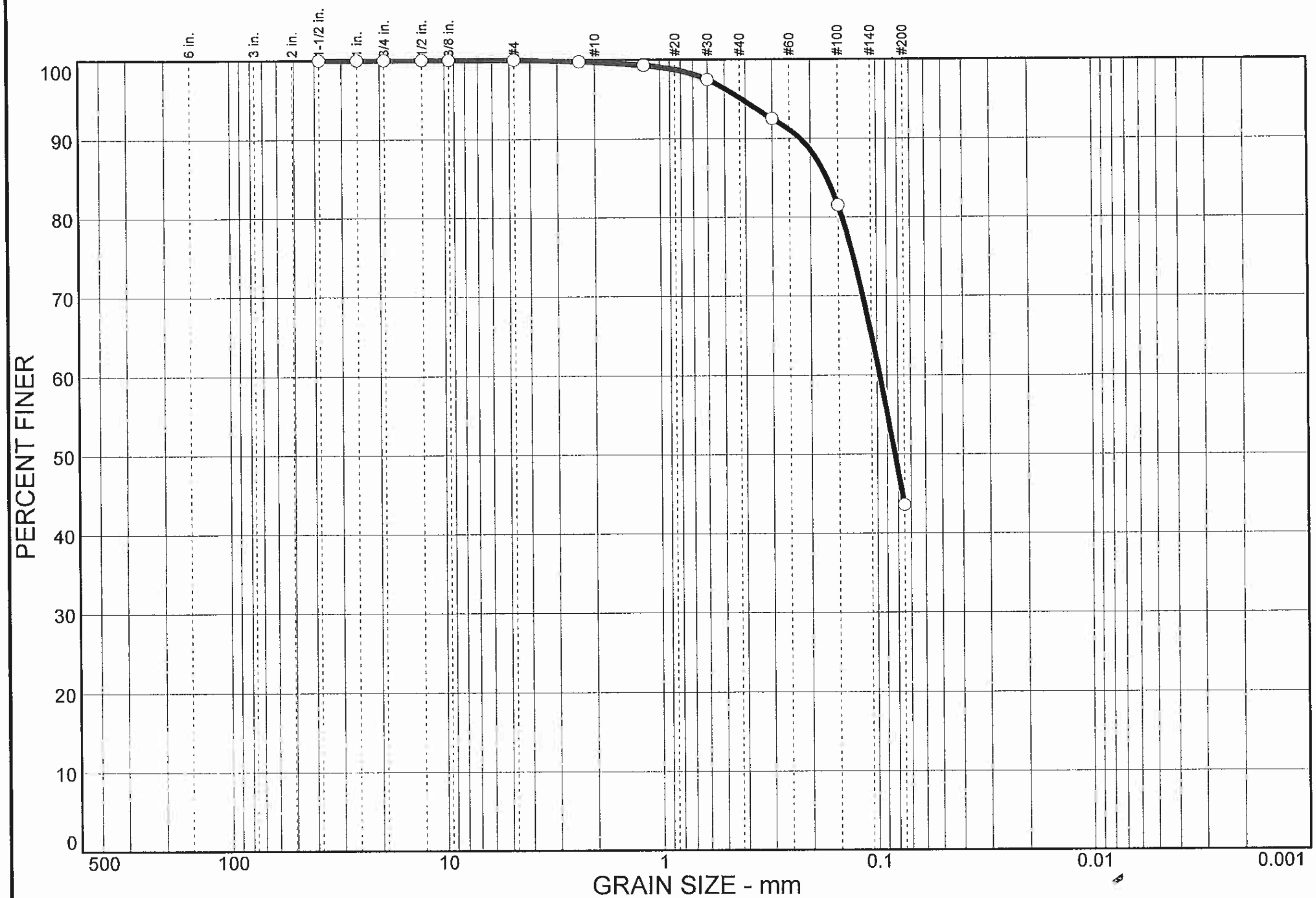
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	56.4	43.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	100.0		
3/4 in.	100.0		
1/2 in.	100.0		
3/8 in.	100.0		
#4	100.0		
#8	99.8		
#16	99.3		
#30	97.5		
#50	92.5		
#100	81.5		
#200	43.6		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.168 D_{60}= 0.0972 D_{50}= 0.0828
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

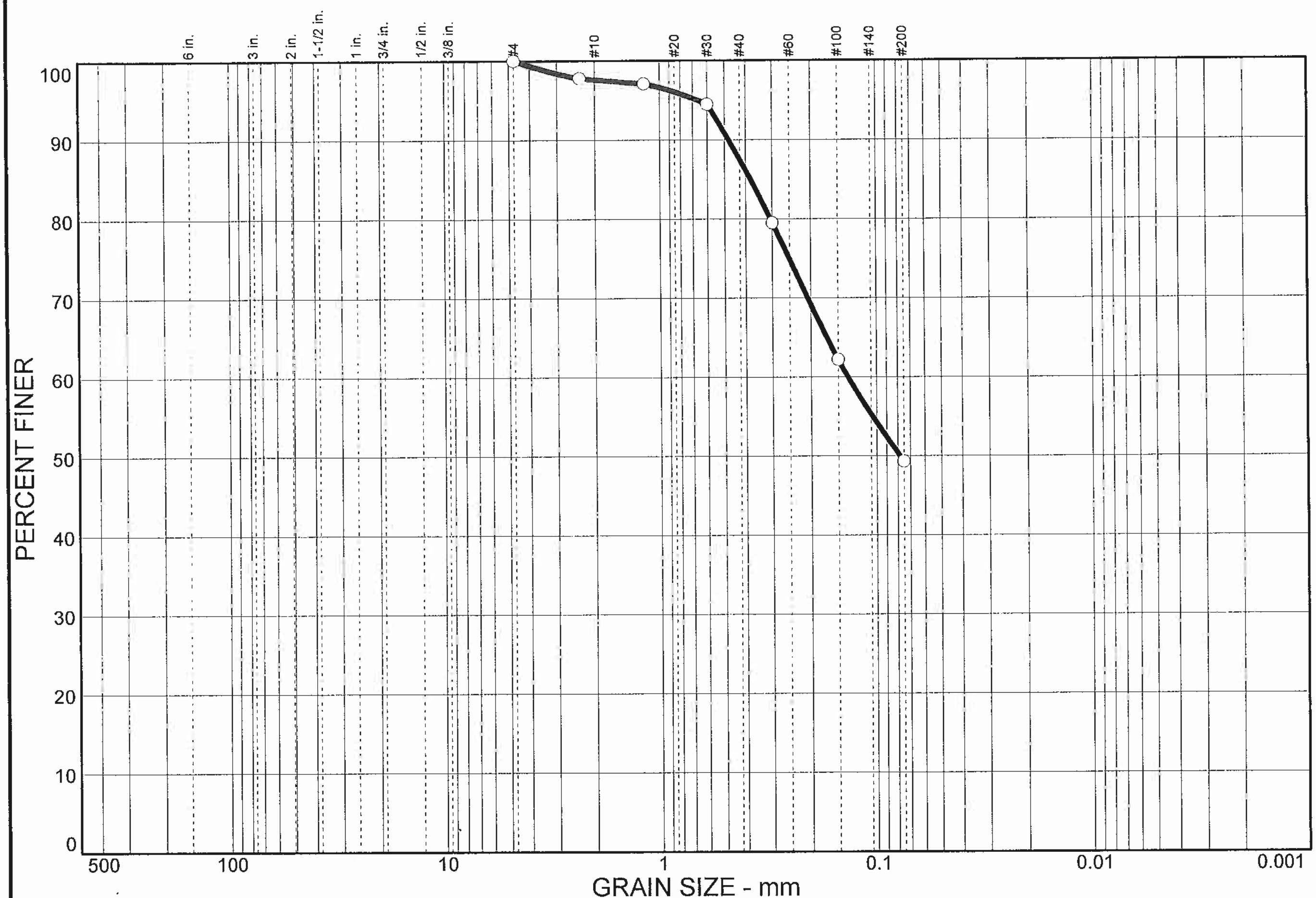
USCS= SM AASHTO=

Remarks

Sample No.: P-3 **Source of Sample:** **Date:**
Location: **Elev./Depth:** 13.5-15'

Moore Twining Associates, Inc.	**Client:**
	Project: Mission Valley - Home Depot
Fresno, CA	**Project No:** D050R0.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	50.7	49.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	97.8		
#16	97.1		
#30	94.5		
#50	79.5		
#100	62.2		
#200	49.3		

* (no specification provided)

Material Description

Clayey sand (Fill)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.380 D_{60}= 0.135 D_{50}= 0.0782
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

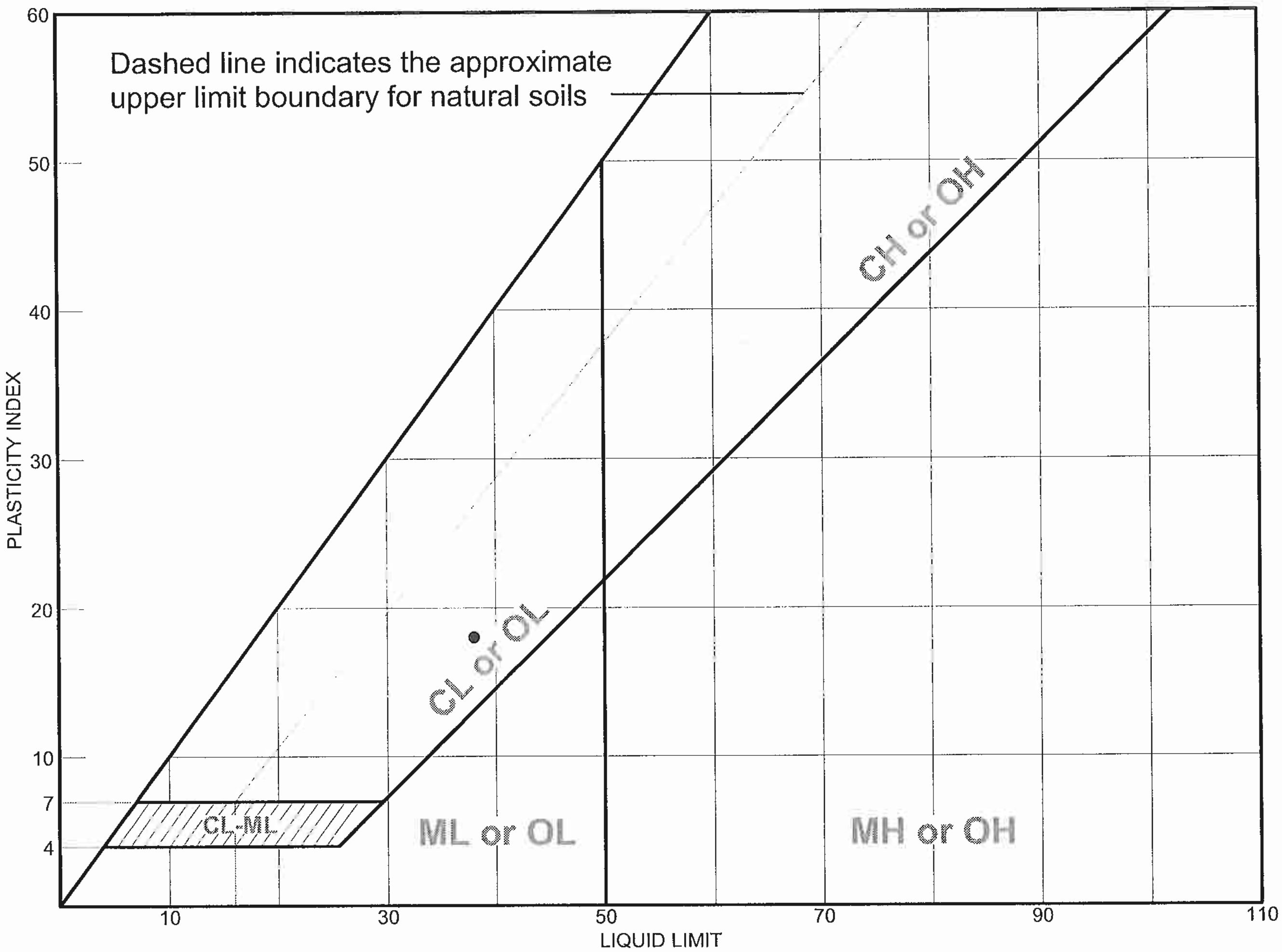
USCS= SC AASHTO=

Remarks

Sample No.: S-1 **Source of Sample:** **Date:** 3/4/19
Location: **Elev./Depth:** 2-3.5'

Moore Twining Associates, Inc. **Fresno, CA**	**Client:** **Project:** Mission Valley - Home Depot **Project No:** D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand (Fill)	38	20	18	87.5	49.3	SC

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** S-1 **Elev./Depth:** 2-3.5'

Remarks:

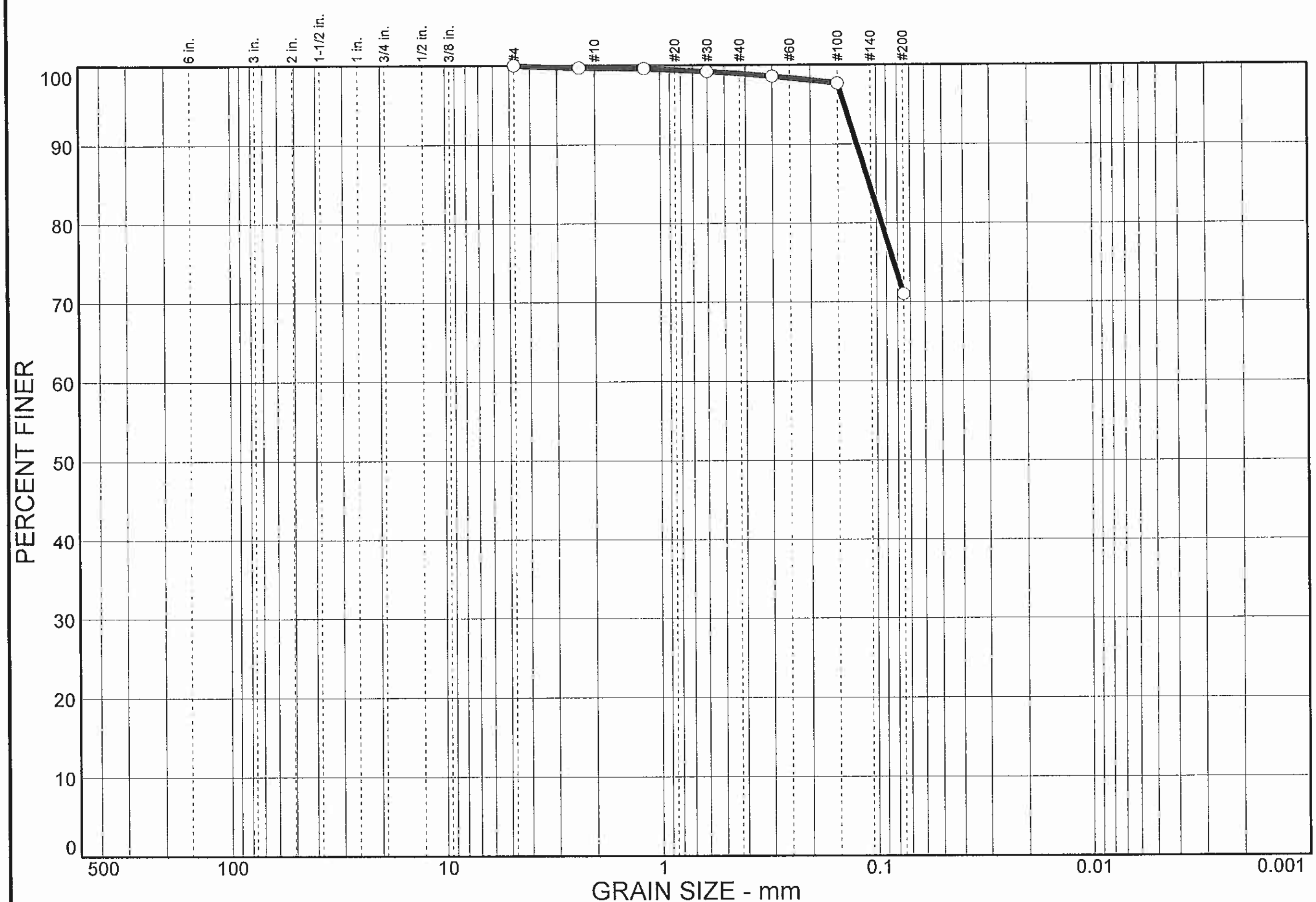
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	29.0	71.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.7		
#16	99.6		
#30	99.2		
#50	98.6		
#100	97.7		
#200	71.0		

* (no specification provided)

Material Description

Silt with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.108 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

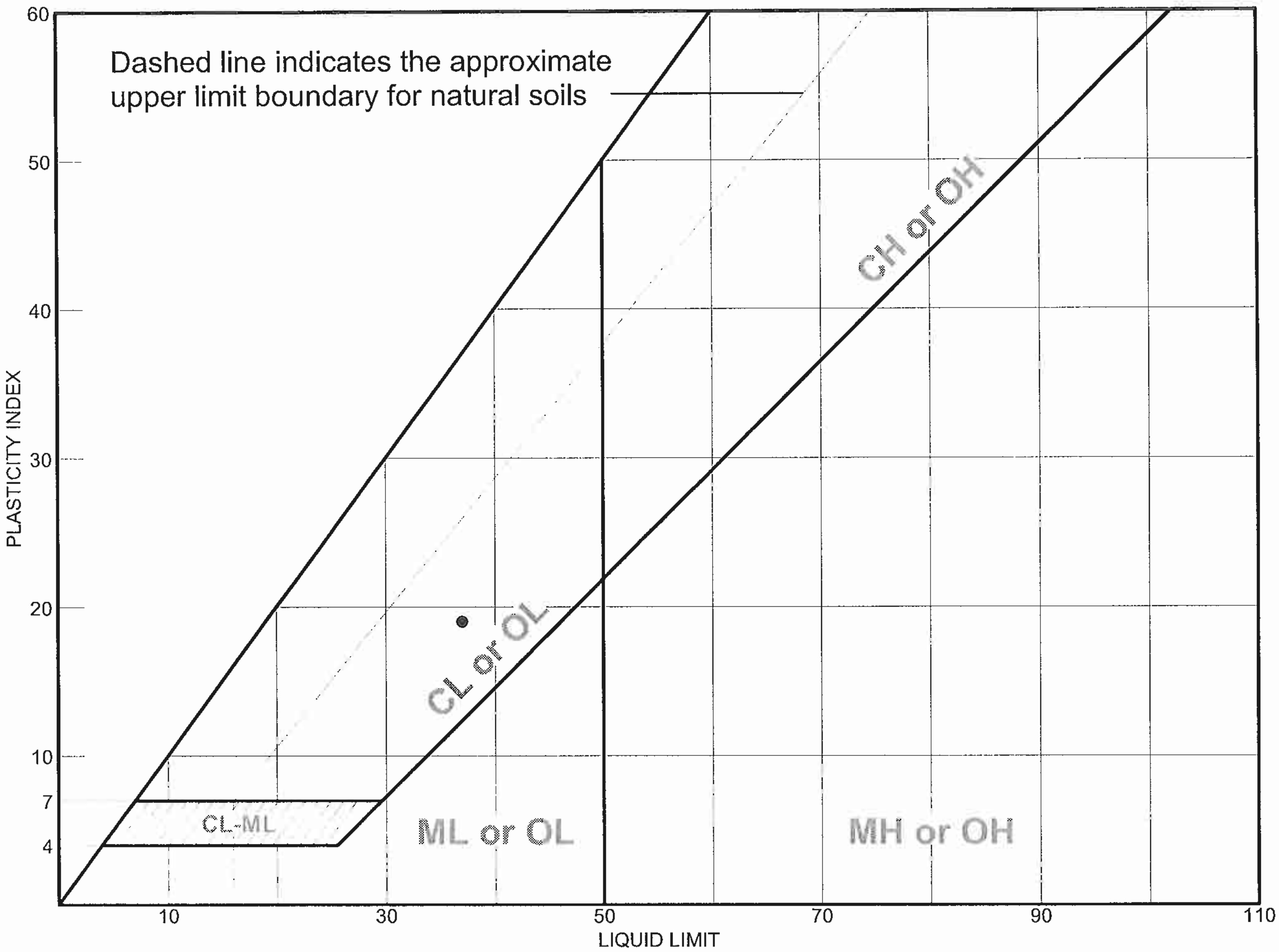
Sample No.: S-5 **Source of Sample:** **Date:** 3/4/19
Location: **Elev./Depth:** 30-31.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt with sand	37	18	19	98.9	71.0	ML

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** S-5 **Elev./Depth:** 30-31.5'

Remarks:

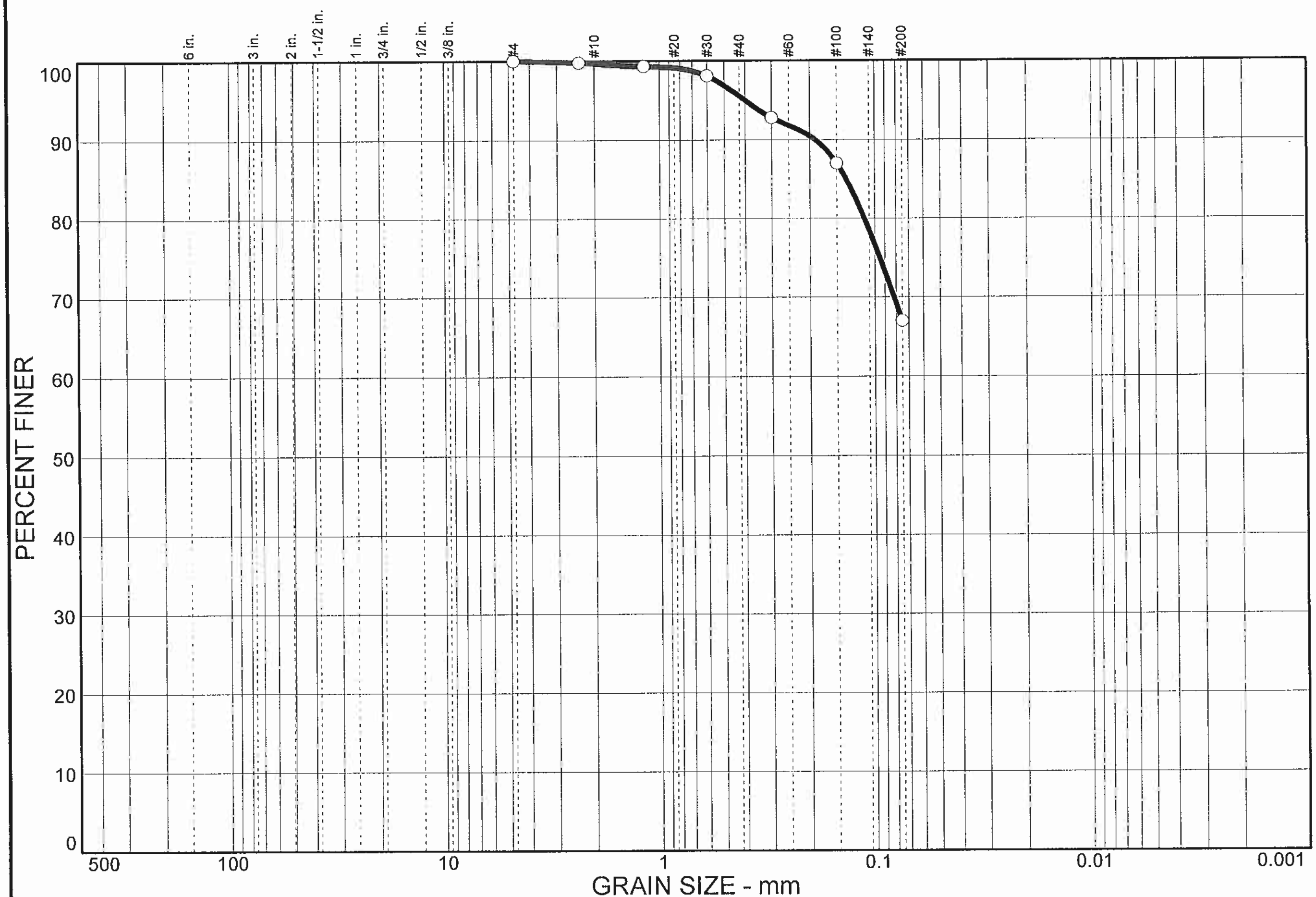
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	33.0	67.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.8		
#16	99.3		
#30	98.1		
#50	92.8		
#100	87.0		
#200	67.0		

* (no specification provided)

Material Description

Sandy lean clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.136 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

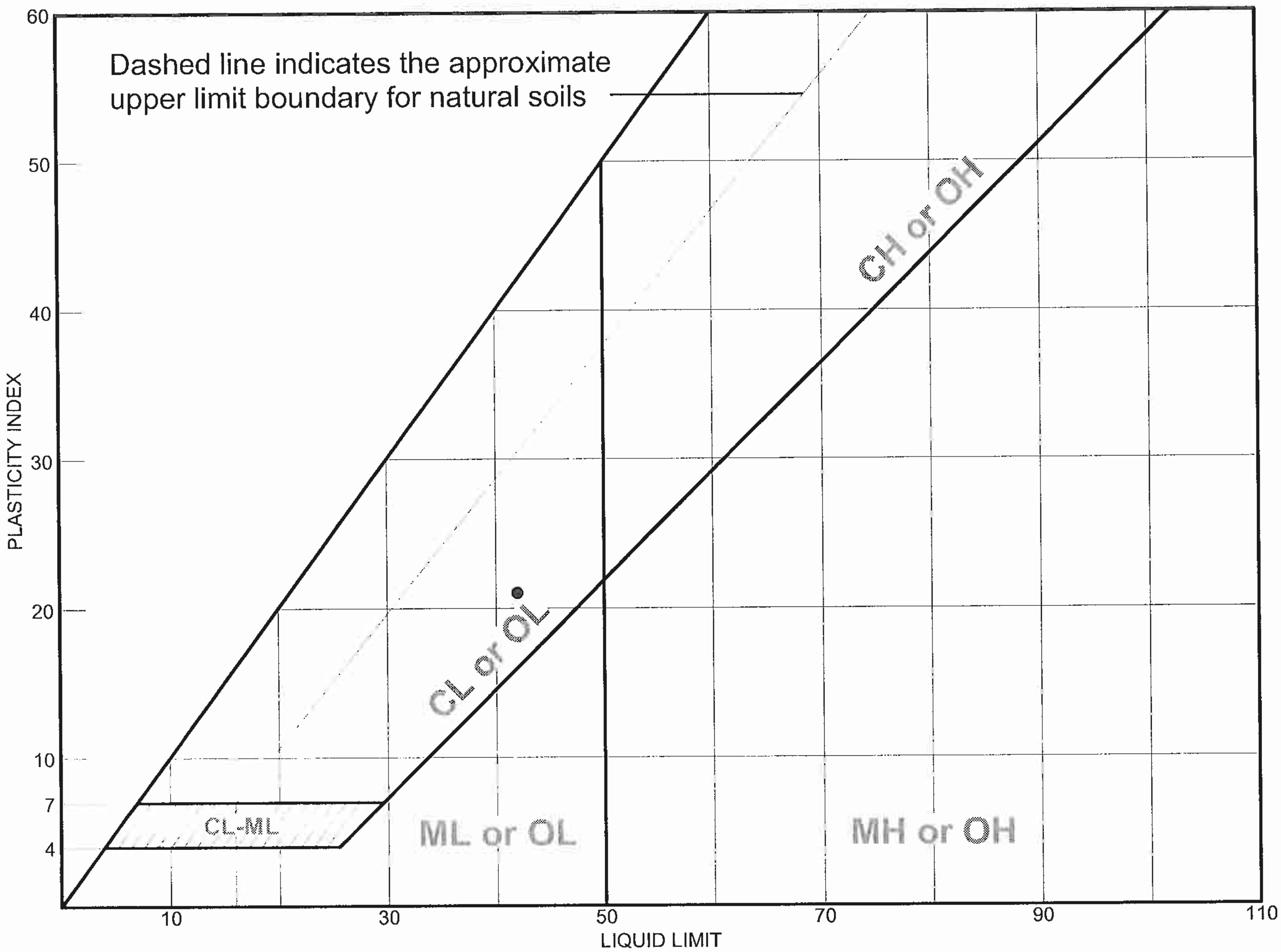
Sample No.: S-7 **Source of Sample:** **Date:** 3/4/19
Location: **Elev./Depth:** 5-6.5'

Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot
Project No: D050R0.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy lean clay	42	21	21	95.5	67.0	CL

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** S-7 **Elev./Depth:** 5-6.5'

Remarks:

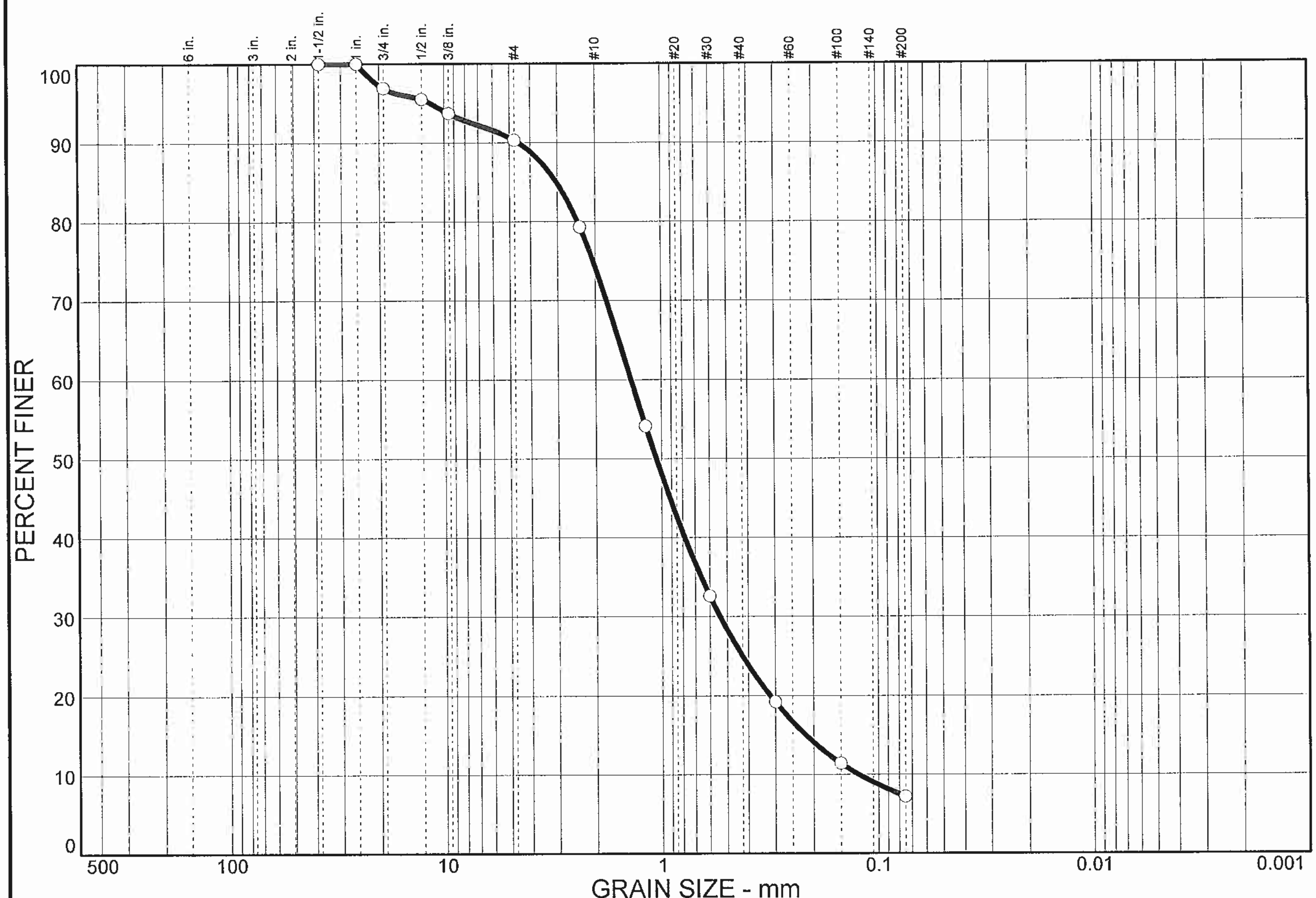
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	9.7	83.1	7.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	100.0		
3/4 in.	96.9		
1/2 in.	95.5		
3/8 in.	93.7		
#4	90.3		
#8	79.3		
#16	54.1		
#30	32.6		
#50	19.2		
#100	11.4		
#200	7.2		

* (no specification provided)

Material Description

Well-graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 3.03 D_{60}= 1.37 D_{50}= 1.06
D_{30}= 0.539 D_{15}= 0.217 D_{10}= 0.124
C_u= 11.10 C_c= 1.71

Classification

USCS= SW-SM AASHTO=

Remarks

Sample No.: S-8 **Source of Sample:** **Date:** 3/4/19
Location: **Elev./Depth:** 40-41.5'

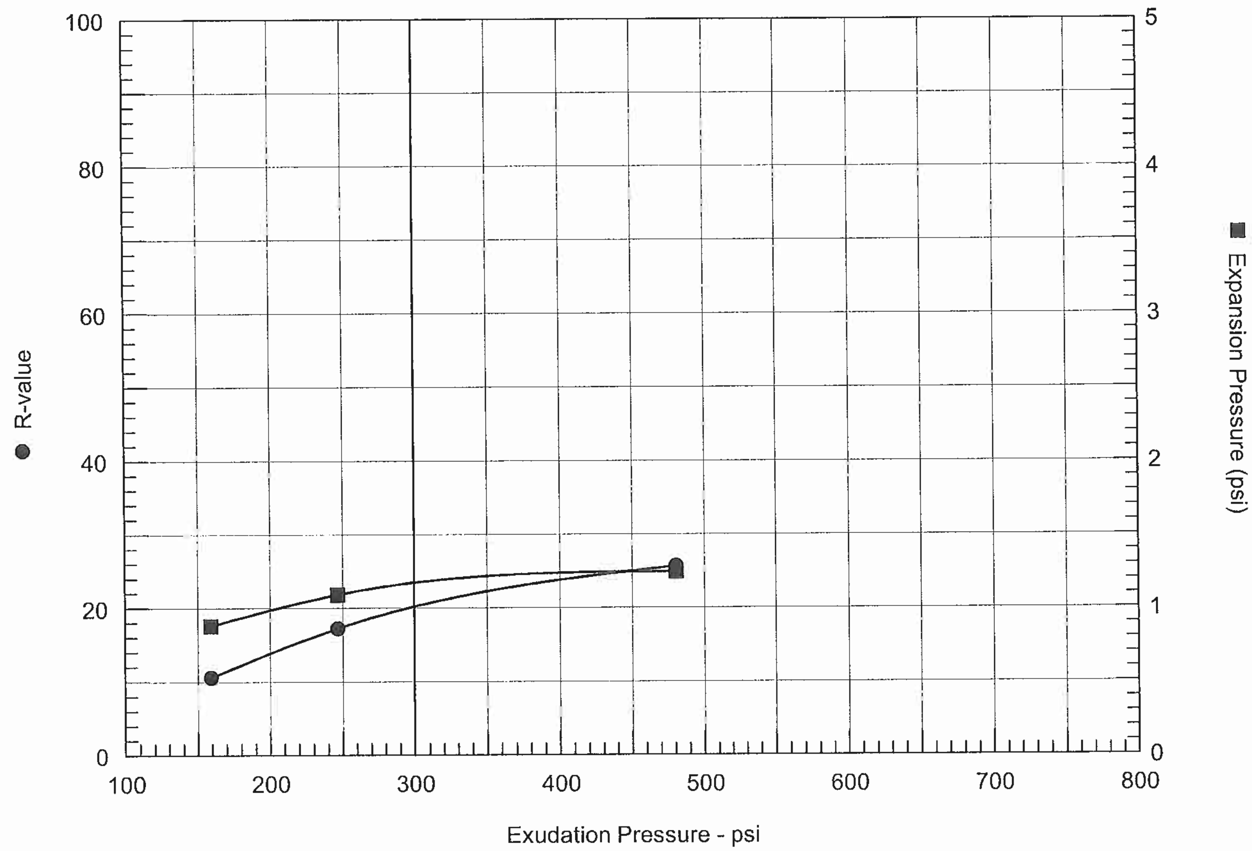
Moore Twining Associates, Inc.

Fresno, CA

Client:
Project: Mission Valley - Home Depot

Project No: D050R0.01 **Figure**

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	150	113.8	14.3	1.24	91	2.56	481	25	26
2	100	112.2	15.3	1.09	107	2.59	247	16	17
3	50	111.5	16.4	0.88	123	2.62	159	10	11

Test Results

R-value at 300 psi exudation pressure = 20

Exp. pressure at 300 psi exudation pressure = 1.17 psi

Material Description

Sandy lean clay

Project No.: D050R0.01

Project: Mission Valley - Home Depot

Sample Number: A-2 **Depth:** 0-5'

Date: 4/17/2019

Tested by: TD

Checked by:

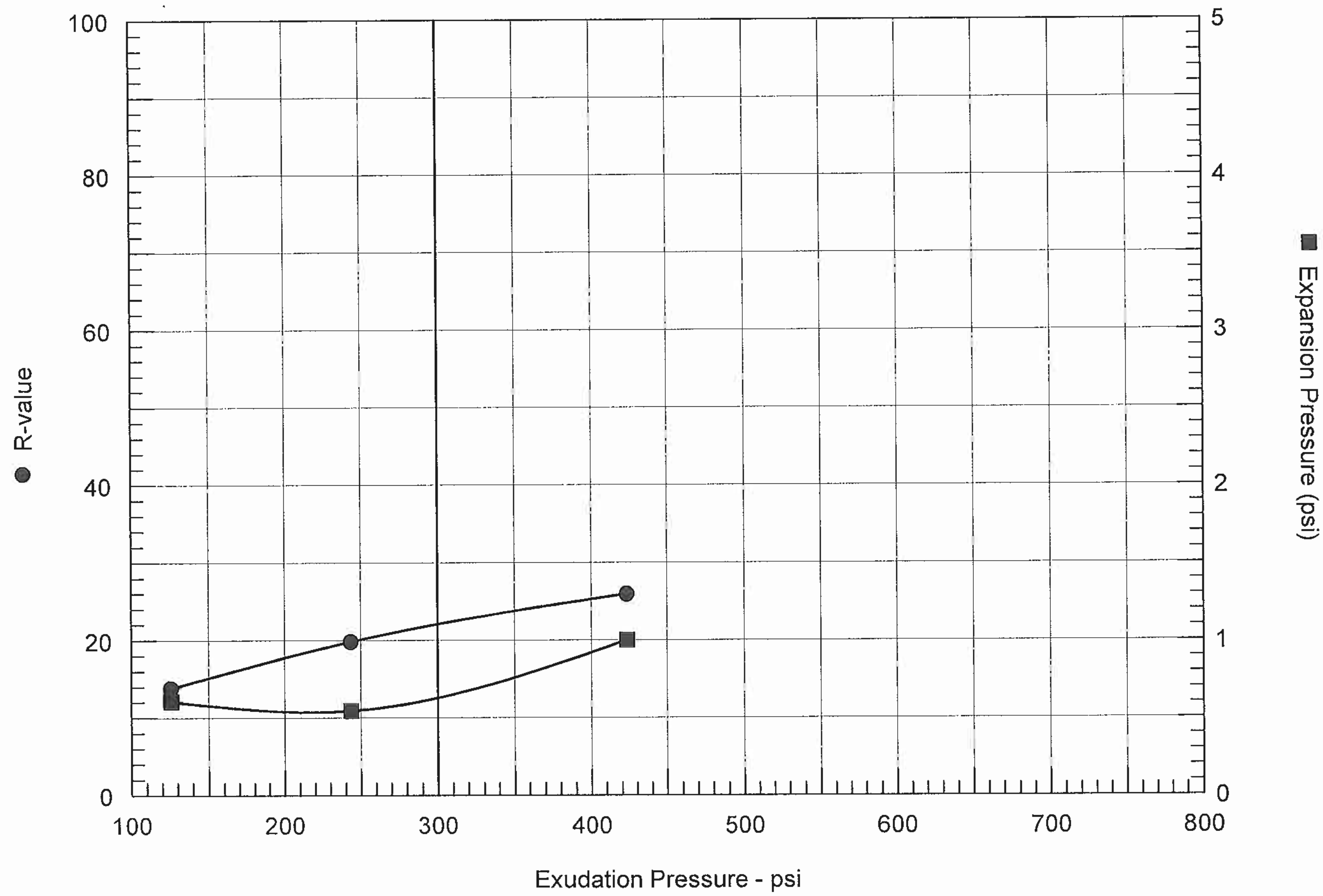
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure ______

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	100	115.3	15.3	0.55	101	2.61	244	18	20
2	50	113.7	16.3	0.61	113	2.63	126	13	14
3	150	116.7	14.3	1.00	89	2.57	423	25	26

Test Results	Material Description
R-value at 300 psi exudation pressure = 22 **Exp. pressure at 300 psi exudation pressure =** 0.63 psi	Sandy lean clay

Project No.: D050R0.01

Project: Mission Valley - Home Depot

Sample Number: D-1 **Depth:** 1.2-5'

Date: 4/11/2019

Tested by:

Checked by:

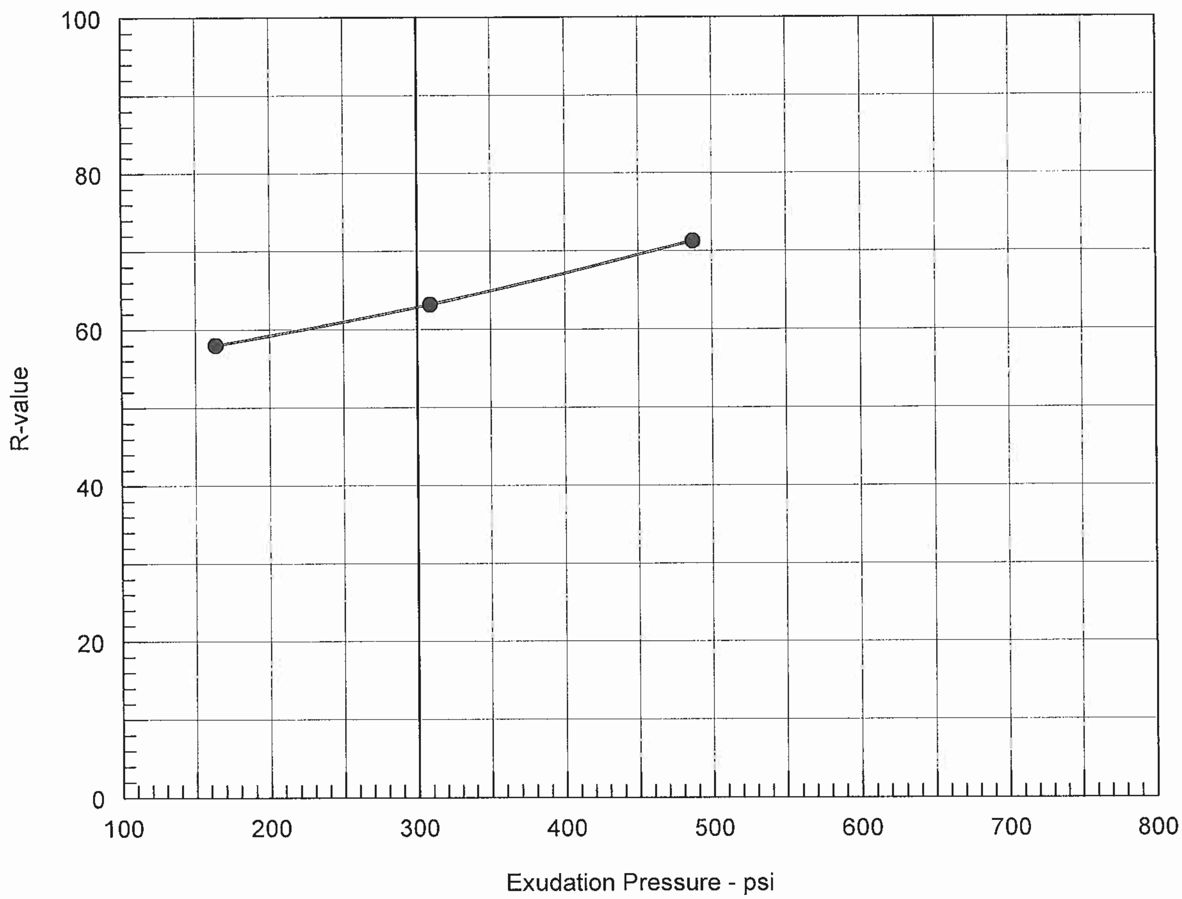
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure ______

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	300	123.6	9.5	0.00	27	2.51	487	71	71
2	250	120.8	11.0	0.00	36	2.56	309	62	63
3	250	121.0	11.9	0.00	41	2.56	164	56	58

Test Results	Material Description
R-value at 300 psi exudation pressure = 63	Silty sand

Project No.: D050R0.01

Project: Mission Valley - Home Depot

Sample Number: E-6 **Depth:** 0.9-5'

Date: 4/11/2019

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

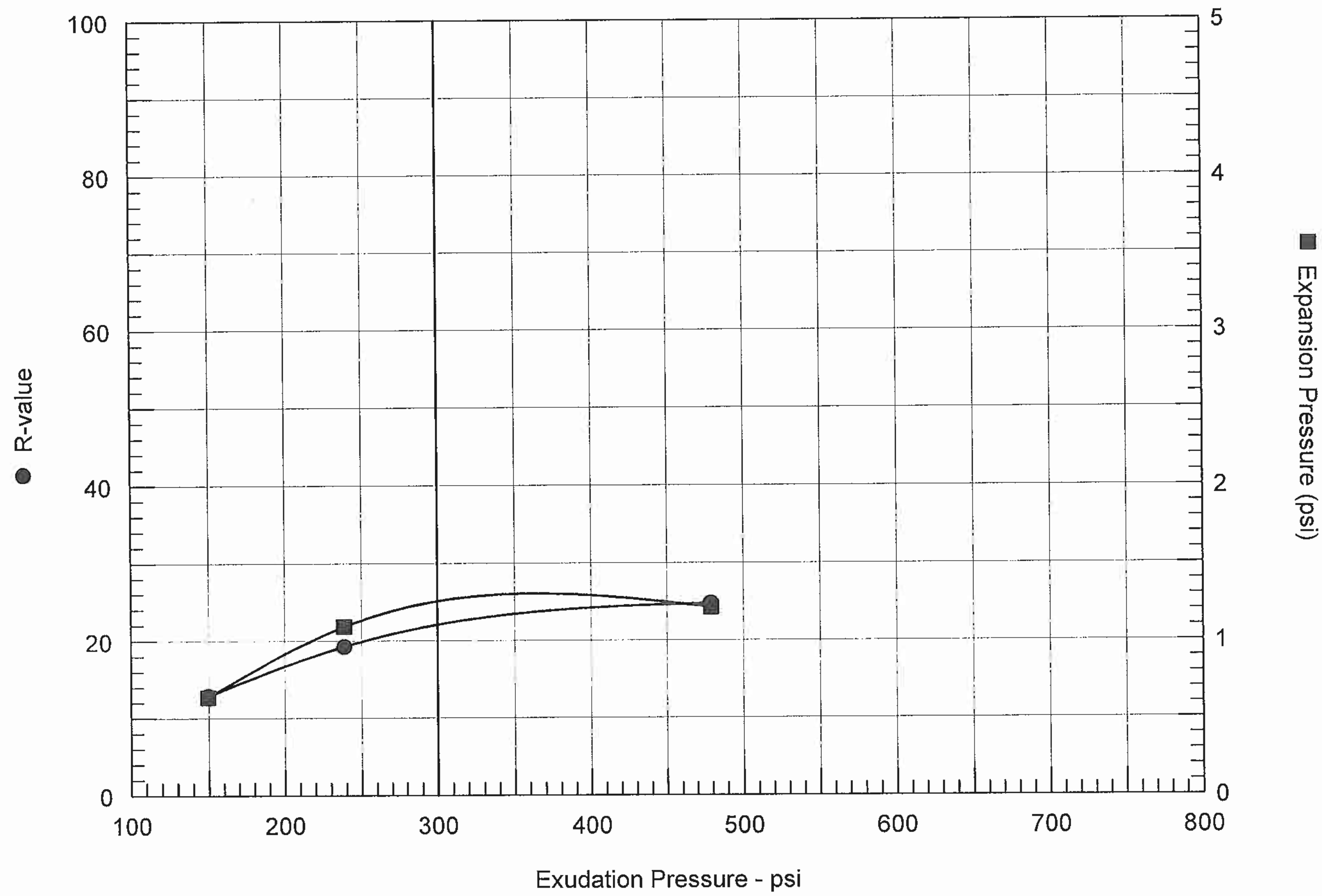
Tested by:

Checked by:

Remarks:

Figure ______

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	100	116.7	13.0	1.21	92	2.58	479	23	25
2	100	115.0	14.0	1.09	101	2.61	239	18	19
3	50	113.1	15.5	0.64	116	2.65	150	12	13

Test Results	Material Description
R-value at 300 psi exudation pressure = 22 **Exp. pressure at 300 psi exudation pressure =** 1.26 psi	Clayey sand

Project No.: D050R0.01

Project: Mission Valley - Home Depot

Sample Number: J-8 **Depth:** 0.9-5'

Date: 4/11/2019

Tested by:

Checked by:

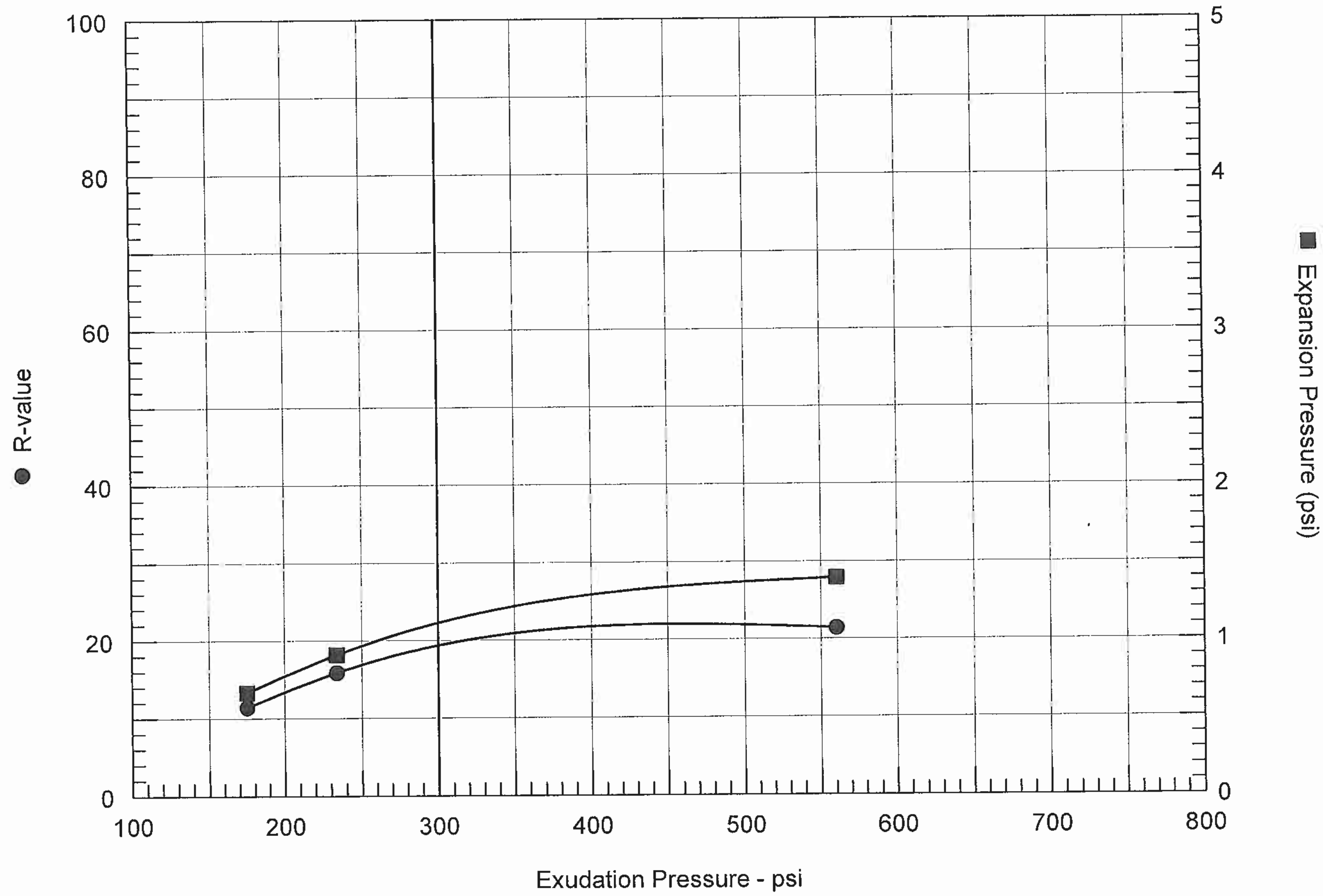
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure ______

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	200	116.7	11.8	1.39	100	2.60	560	20	21
2	100	116.0	13.7	0.91	111	2.62	234	15	16
3	50	115.8	15.2	0.67	120	2.63	175	11	11

Test Results	Material Description
R-value at 300 psi exudation pressure = 19 **Exp. pressure at 300 psi exudation pressure =** 1.11 psi	Sandy lean clay

Project No.: D050R0.01

Project: Mission Valley - Home Depot

Sample Number: H-8 **Depth:** 10-11.5'

Date: 4/11/2019

Tested by:

Checked by:

Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure ______

COMPACTION TEST REPORT

Test specification: ASTM D 1557-12 Method A Modified
Oversize correction applied to each point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0.8-5.0'	SC				31	16	8.0	34.2

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 131.3 pcf	129.3 pcf	Clayey sand
Optimum moisture = 7.8 %	8.3 %	

Project No. D050R0.01 **Client:**
Project: Mission Valley - Home Depot

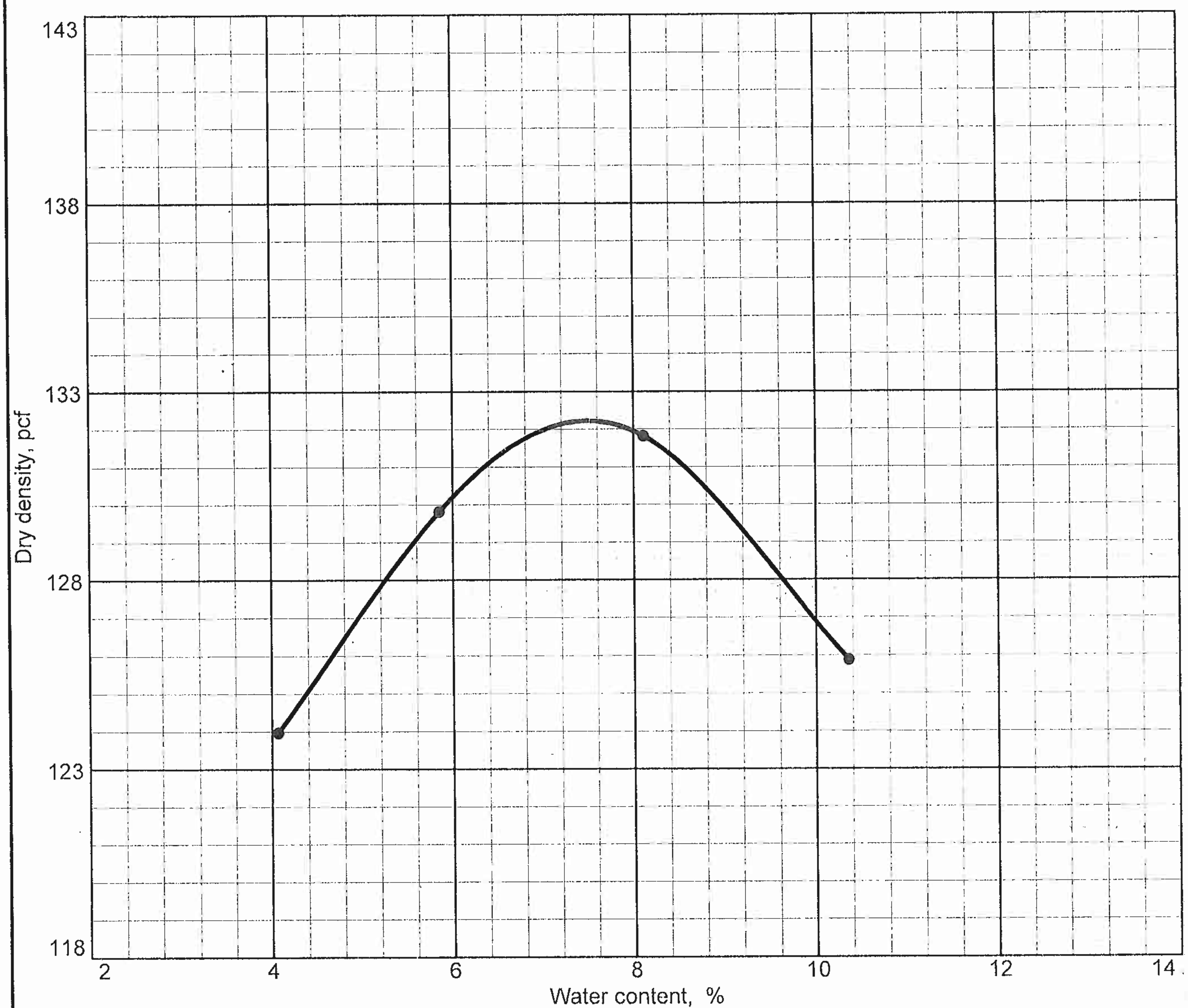
● **Source:** **Sample No.:** B-7 **Elev./Depth:** 0.8-5.0'

Moore Twining Associates, Inc.
Fresno, CA

Remarks:

Figure

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
1-3'	SC				27	12	0.0	37.1

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 132.2 pcf Optimum moisture = 7.5 %	Clayey sand

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** E-8 **Elev./Depth:** 1-3'

Remarks:

Moore Twining Associates, Inc.

Fresno, CA

Figure

COMPACTION TEST REPORT

Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
1-4.5'	CL							

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 126.8 pcf Optimum moisture = 10.4 %	Sandy lean clay

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** F-1A **Elev./Depth:** 1-4.5'

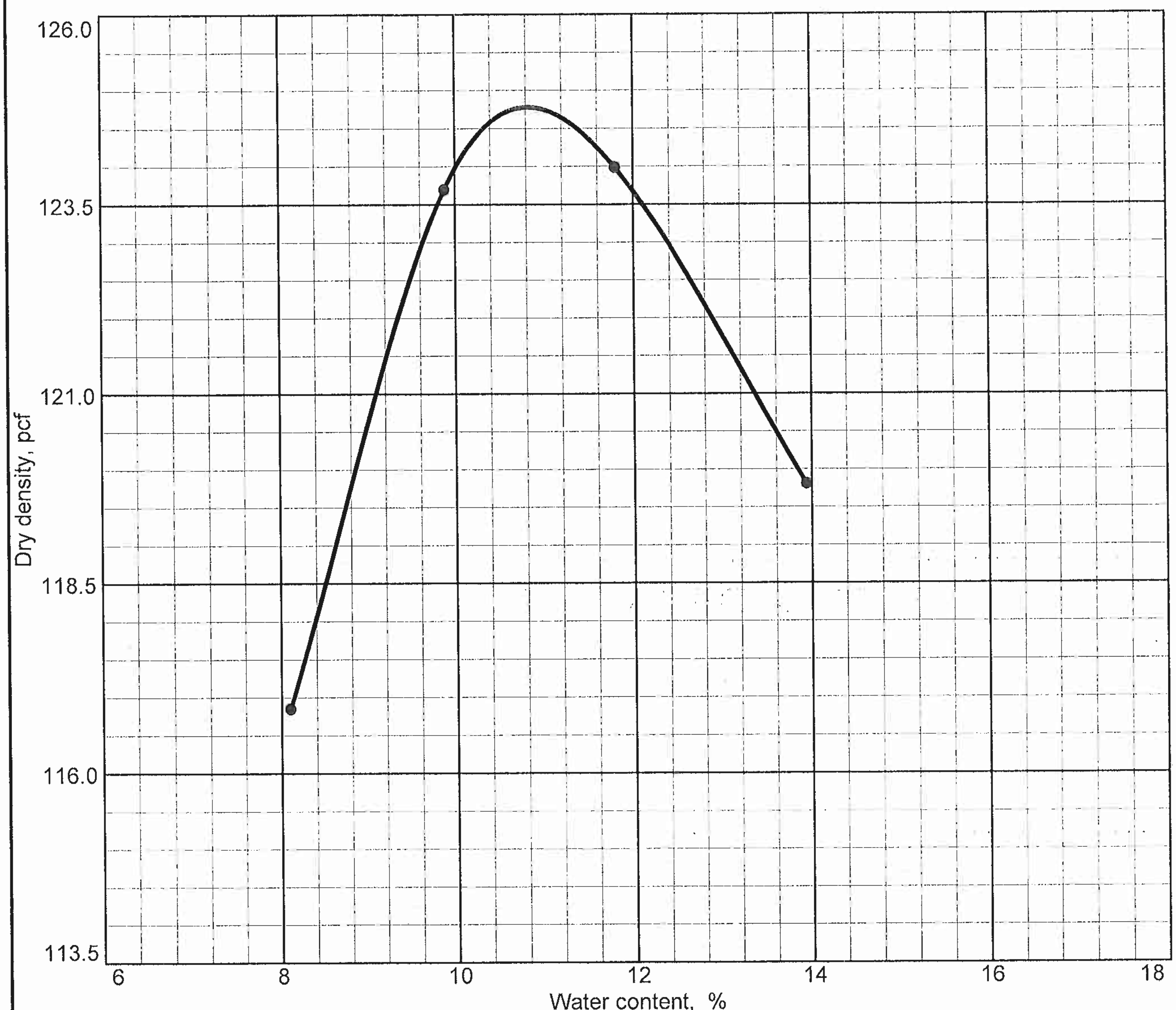
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
1-5'	CL				30	12		68.4

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 124.8 pcf Optimum moisture = 10.8 %	Sandy lean clay

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** G-7.6 **Elev./Depth:** 1-5'

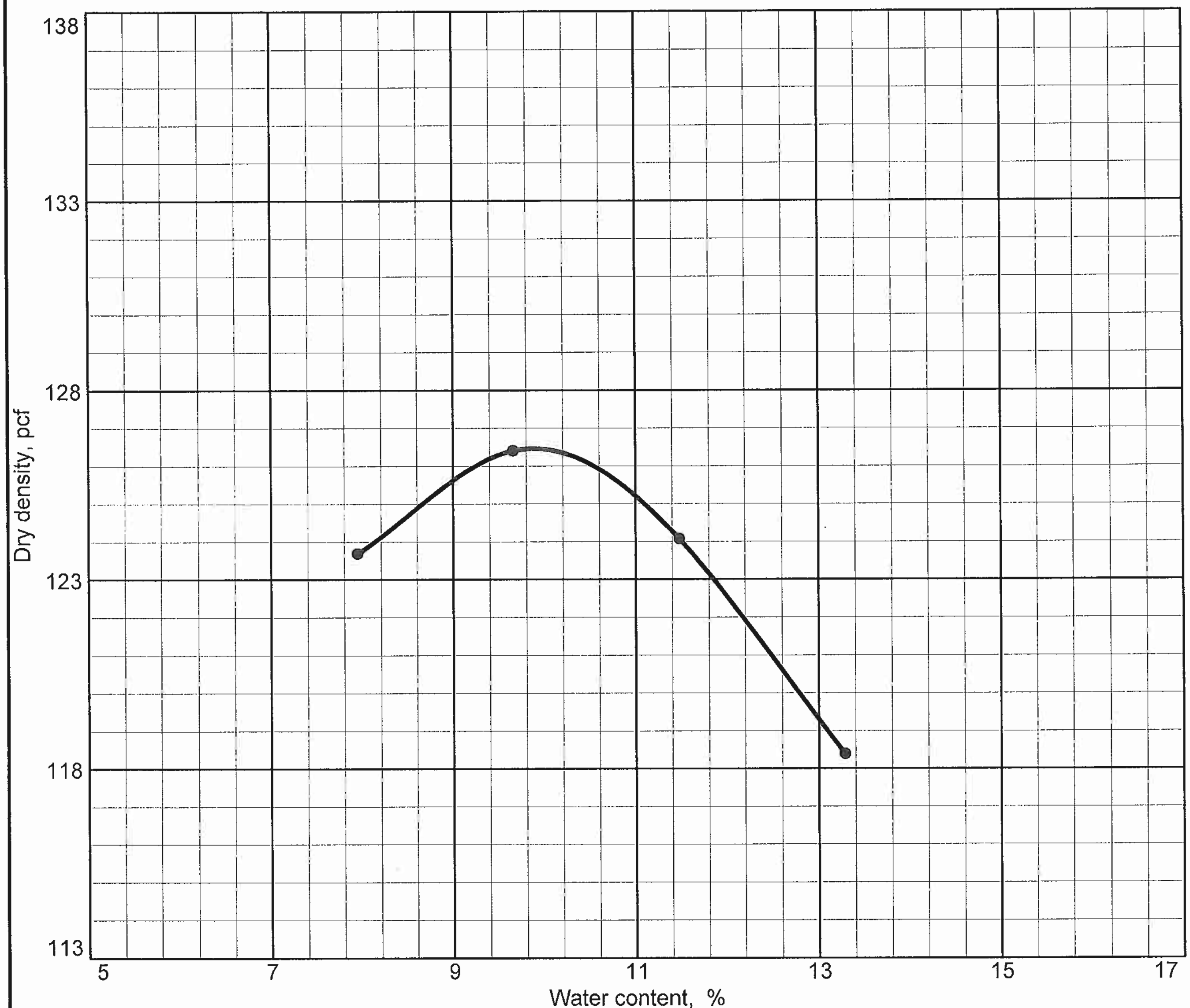
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
1-5'	SC							

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 126.5 pcf Optimum moisture = 9.9 %	Clayey sand

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** H-5 **Elev./Depth:** 1-5'

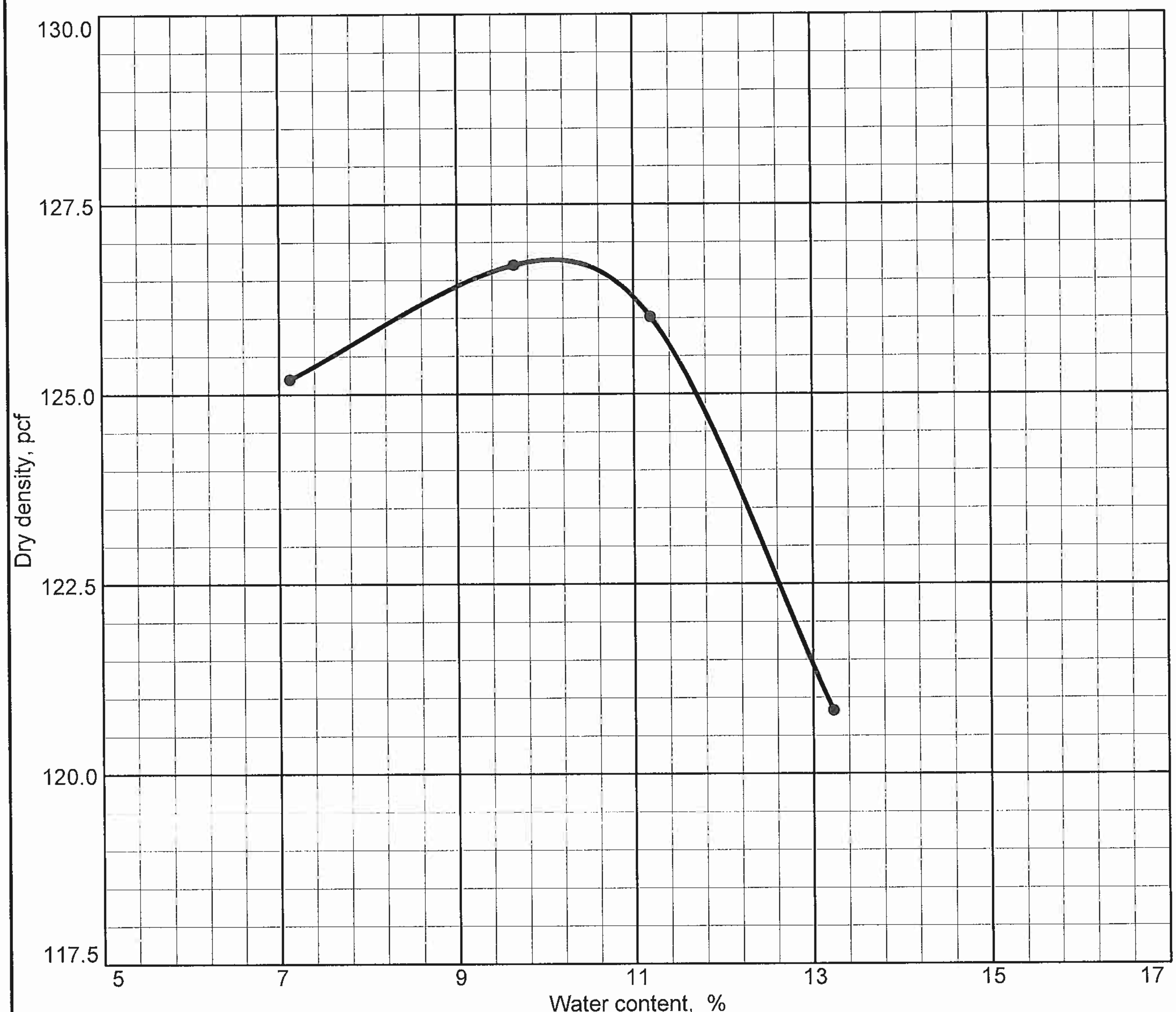
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
1-5'	CL						0.0	54.6

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 126.8 pcf Optimum moisture = 10.1 %	Sandy lean clay

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** H-8 **Elev./Depth:** 1-5'

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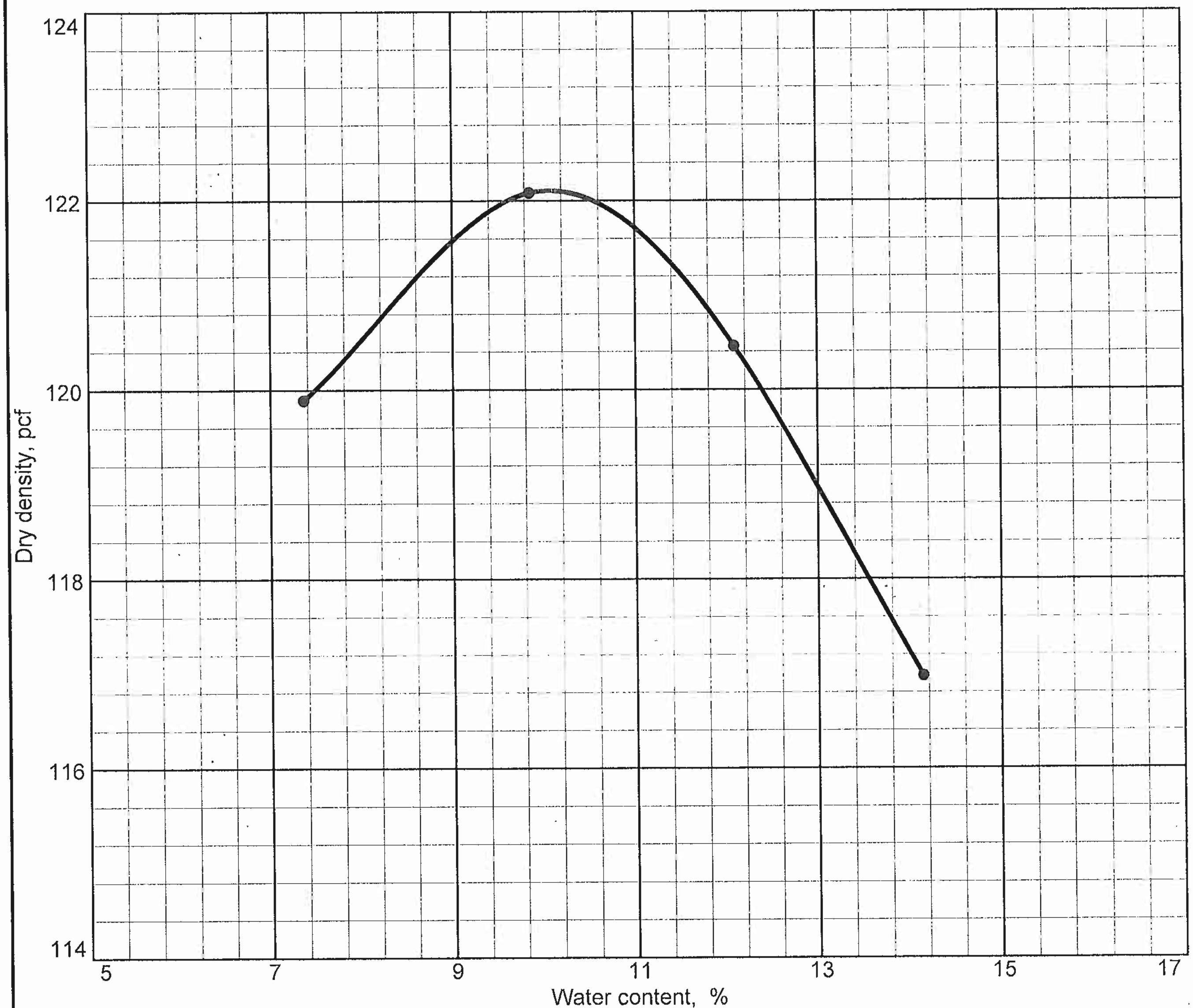
Fresno, CA

Remarks:

Figure

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
	CL				31	14	0.0	57.4

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 122.1 pcf Optimum moisture = 10.1 %	Sandy lean clay

Project No. D050R0.01 **Client:**

Project: Mission Valley - Home Depot

● **Source:** **Sample No.:** Top Cut Slope (CST)

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Remarks:

Figure

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EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Mission Valley - Home Depot	REPORT DATE:	2/29/19
		TEST DATE:	3/25/2019
MTA PROJECT NO.:	D050R0.01		
SAMPLE I.D.:	F-1A @ 1-4.5'		
SAMPLED BY:	VB / JC		
SAMPLE DATE:	2/25/2019	TESTED BY:	TD

MATERIALS DESCRIPTION: Sandy lean clay

% PASSING # 4 SIEVE: 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9198
Pan + Dry Soil Wt., gm	223.8	Dry Soil Wt., lbs	0.7478
Pan Wt., gm	0.0		
Initial % Moisture Content	11.7	Final % Moisture Content	23.0

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8353	Ring + Sample Wt., lbs	0.9198
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8353	Remolded Wt., lbs	0.9198
Remolded Wet Density, pcf	114.9	Remolded Wet Density, pcf	117.5
Remolded Dry Density, pcf	102.8	Remolded Dry Density, pcf	95.5

Initial Volume	Final Volume
0.00727222	0.007829

Expansion Data:

Initial Gage Reading, in:	0.0500
Final Gage Reading, in:	0.1265
Expansion, in:	0.0765
Expansion Index	77

Comments: **Medium Expansion Potential**

Classification of Expansive Soils. (Table No.1 From ASTM D4829)

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Mission Valley - Home Depot	REPORT DATE:	2/29/19
		TEST DATE:	3/25/2019
MTA PROJECT NO.:	D050R0.01		
SAMPLE I.D.:	H-5 @ 1-5'		
SAMPLED BY:	VB / JC		
SAMPLE DATE:	2/25/2019	TESTED BY:	TD

MATERIALS DESCRIPTION: Sandy lean clay

% PASSING # 4 SIEVE: 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9251
Pan + Dry Soil Wt., gm	224.2	Dry Soil Wt., lbs	0.7539
Pan Wt., gm	0.0		
Initial % Moisture Content	11.5	Final % Moisture Content	22.7

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8406	Ring + Sample Wt., lbs	0.9251
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8406	Remolded Wt., lbs	0.9251
Remolded Wet Density, pcf	115.6	Remolded Wet Density, pcf	117.7
Remolded Dry Density, pcf	103.7	Remolded Dry Density, pcf	95.9

Initial Volume	Final Volume
0.00727222	0.007859

Expansion Data:

Initial Gage Reading, in:	0.0500
Final Gage Reading, in:	0.1307
Expansion, in:	0.0807
Expansion Index	81

Comments: **Medium Expansion Potential**

Classification of Expansive Soils. (Table No.1 From ASTM D4829)

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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Project Name:	Mission Valley - Home Depot	Report Date:	3/29/2019
		Sample Date:	2/25/2019
Project Number:	D050R0.01		
		Sampled By:	VB / JC
Subject:	Minimum Resistivity, ASTM G187	Tested By:	TD
Material Description:	Silty sand	Test Date:	3/27/2019
Location:	E-6 @ 0.9-5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	12,006
100 mls	10,005
150 mls	8,004
200 mls	5,336
250 mls	4,602
300 mls	4,869

Remarks: Min. Resistivity is 4,602 Ohm-cm

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Project Name:	Mission Valley - Home Depot	Report Date:	3/29/2019
		Sample Date:	2/25/2019
Project Number:	D050R0.01		
		Sampled By:	VB / JC
Subject:	Minimum Resistivity, ASTM G187	Tested By:	TD
Material Description:	Sandy lean clay	Test Date:	3/27/2019
Location:	H-5 @ 1-5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	8,671
100 mls	6,670
150 mls	4,002
200 mls	2,268
250 mls	1,801
300 mls	1,868
350 mls	2,068

Remarks: Min. Resistivity is 1,801 Ohm-cm

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Project Name:	Mission Valley - Home Depot	Report Date:	3/29/2019
		Sample Date:	2/25/2019
Project Number:	D050R0.01		
		Sampled By:	VB / JC
Subject:	Minimum Resistivity, ASTM G187	Tested By:	TD
Material Description:	Sandy lean clay	Test Date:	3/27/2019
Location:	D-1 @ 1.2-5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	8,004
100 mls	6,337
150 mls	4,736
200 mls	2,668
250 mls	2,201
300 mls	2,535

Remarks: Min. Resistivity is 2,201 Ohm-cm

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MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

March 15, 2019

Work Order #: **FC06030**

Scott Krauter
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: Soil Investigation in San Diego

Enclosed are the analytical results for samples received by our laboratory on **03/06/19** . For your reference, these analyses have been assigned laboratory work order number **FC06030**.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.

Susan Federico

Susan Federico
Client Services Representative

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California ELAP Certificate #1371

2527 Fresno Street
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(559) 268-0740 Fax

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Soil Investigation in San Diego **Project Number:** D050R0.01 **Project Manager:** Scott Krauter	**Reported:** 03/15/2019

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
D1@1.2-5		FC06030-01	Soil	03/06/19 00:00	03/06/19 14:00
E6@0.9-5		FC06030-02	Soil	03/06/19 00:00	03/06/19 14:00
H5@1-5		FC06030-03	Soil	03/06/19 00:00	03/06/19 14:00

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(559) 268-0740 Fax

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Soil Investigation in San Diego **Project Number:** D050R0.01 **Project Manager:** Scott Krauter	**Reported:** 03/15/2019

D1@1.2-5

FC06030-01 (Soil) Sampled: 03/06/19 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**17**	6.0	mg/kg	3	B9C0616	03/06/19	03/07/19	ASTM D4327-84
Chloride		**0.0017**	0.00060	% by Weight	3	[CALC]	03/07/19	03/07/19	ASTM D4327-84
Sulfate as SO4		**0.0042**	0.00060	% by Weight	3	[CALC]	03/07/19	03/07/19	ASTM D4327-84
pH		**8.8**	0.10	pH Units	1	B9C0616	03/06/19	03/07/19	ASTM D4972-89 Mod
Sulfate as SO4		**42**	6.0	mg/kg	3	B9C0616	03/06/19	03/07/19	ASTM D4327

E6@0.9-5

FC06030-02 (Soil) Sampled: 03/06/19 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		ND	6.0	mg/kg	3	B9C0616	03/06/19	03/07/19	ASTM D4327-84
Chloride		ND	0.00060	% by Weight	3	[CALC]	03/07/19	03/07/19	ASTM D4327-84
Sulfate as SO4		**0.0021**	0.00060	% by Weight	3	[CALC]	03/07/19	03/07/19	ASTM D4327-84
pH		**8.5**	0.10	pH Units	1	B9C0616	03/06/19	03/07/19	ASTM D4972-89 Mod
Sulfate as SO4		**21**	6.0	mg/kg	3	B9C0616	03/06/19	03/07/19	ASTM D4327

H5@1-5

FC06030-03 (Soil) Sampled: 03/06/19 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		ND	6.0	mg/kg	3	B9C0616	03/06/19	03/07/19	ASTM D4327-84
Chloride		ND	0.00060	% by Weight	3	[CALC]	03/07/19	03/07/19	ASTM D4327-84
Sulfate as SO4		**0.0033**	0.00060	% by Weight	3	[CALC]	03/07/19	03/07/19	ASTM D4327-84
pH		**8.9**	0.10	pH Units	1	B9C0616	03/06/19	03/07/19	ASTM D4972-89 Mod
Sulfate as SO4		**33**	6.0	mg/kg	3	B9C0616	03/06/19	03/07/19	ASTM D4327

Notes and Definitions

DUP1	A high RPD was observed between a sample and this sample's duplicate.
µg/L	micrograms per liter (parts per billion concentration units)
mg/L	milligrams per liter (parts per million concentration units)
mg/kg	milligrams per kilogram (parts per million concentration units)
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference

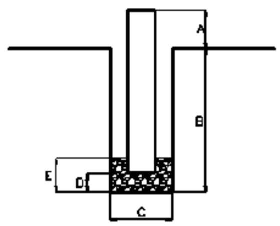
Analysis of pH, filtration, and residual chlorine is to take place immediately after sampling in the field.
If the test was performed in the laboratory, the hold time was exceeded. **(for aqueous matrices only)**

APPENDIX D

RESULTS OF PERCOLATION TESTS

PERCOLATION TEST
No. P-1

Project:	New Home Depot Store - Mission Valley	**Project No.**	D050R0.01
Location:	San Diego	**Test Date:**	3/13/2019
Boring Location	J-8		



A. Top of Pipe Above Ground	11 Inches
B. Depth of Hole	73 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	30 Inches
F. Pipe Length	80 Inches
G. Pipe Diameter	2 Inches

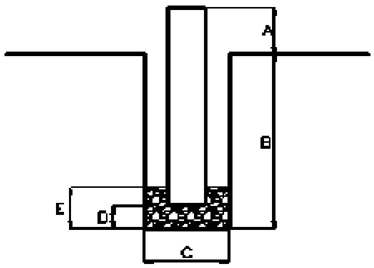
Pre-saturated:	3/12/2019	to 32 inches from bottom at 2:45pm
Checked	3/13/2019	no water at 7:58 am

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/13/2019	10:45:00	5.21				
	3/13/2019	11:15:00	5.63	30	5.04	15.2	0.4
2	3/13/2019	11:15:00	5.63				
	3/13/2019	11:47:00	5.81	32	2.16	37.9	0.2
3	3/13/2019	11:52:00	5.73				
	3/13/2019	12:02:00	5.78	10	0.6	42.7	0.2
4	3/13/2019	12:02:00	5.78				
	3/13/2019	12:12:00	5.87	10	1.08	23.7	0.3
5	3/13/2019	12:12:00	5.87				
	3/13/2019	12:24:00	5.95	12	0.96	32.0	0.2
6	3/13/2019	12:29:00	5.66				
	3/13/2019	12:39:00	5.71	10	0.6	42.7	0.2
7	3/13/2019	12:39:00	5.71				
	3/13/2019	12:49:00	5.76	10	0.6	42.7	0.2
8	3/13/2019	12:49:00	5.76				
	3/13/2019	12:59:00	5.80	10	0.48	53.3	0.1
9	3/13/2019	13:01:00	5.68				
	3/13/2019	13:31:00	5.80	30	1.44	53.3	0.1
10	3/13/2019	13:31:00	5.80				
	3/13/2019	14:01:00	5.91	30	1.32	58.2	0.1
11	3/13/2019	14:03:00	5.68				
	3/13/2019	14:33:00	5.81	30	1.56	49.2	0.1
12	3/13/2019	14:33:00	5.81				
	3/13/2019	15:03:00	5.94	30	1.56	49.2	0.2

* Depth to water measured from top of pipe

Project:	New Home Depot Store - Mission Valley	**Project No.**	D050R0.01
Location:	San Diego	**Test Date:**	3/13/2019
Boring Location	I-8		



A. Top of Pipe Above Ground	23.5	Inches
B. Depth of Hole	123	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	2	Inches
E. Total Gravel Layer Depth	30	Inches
F. Pipe Length	144.5	Inches
G. Pipe Diameter	2	Inches

Pre-saturated:	3/12/2019	to 34 inches from bottom at 3:30pm
Checked	3/13/2019	no water at 7:54 am

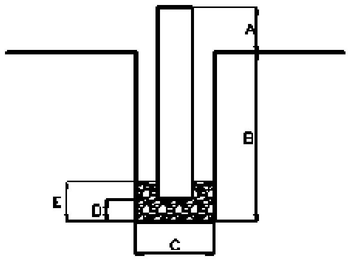
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/13/2019	8:09:00	11.06				
	3/13/2019	8:19:00	11.33	10.00	3.24	7.9	1.1
2	3/13/2019	8:19:00	11.33				
	3/13/2019	8:29:00	11.50	10.00	2.04	12.5	0.8
3	3/13/2019	8:29:00	11.50				
	3/13/2019	8:39:00	11.62	10.00	1.44	17.8	0.7
4	3/13/2019	9:06:00	11.07				
	3/13/2019	9:16:00	11.26	10.00	2.28	11.2	0.7
5	3/13/2019	9:26:00	11.07				
	3/13/2019	9:36:00	11.26	10.00	2.28	11.2	0.7
6	3/13/2019	9:38:00	11.02				
	3/13/2019	9:48:00	11.18	10.00	1.92	13.3	0.6
7	3/13/2019	10:00:00	11.05				
	3/13/2019	10:10:00	11.21	10.00	1.92	13.3	0.6
8	3/13/2019	10:14:00	11.04				
	3/13/2019	10:24:00	11.20	10.00	1.92	13.3	0.6
9	3/13/2019	10:27:00	11.04				
	3/13/2019	10:37:00	11.20	10.00	1.92	13.3	0.6
10	3/13/2019	10:50:00	11.04				
	3/13/2019	11:00:00	11.22	10.00	2.16	11.8	0.7
11	3/13/2019	11:03:00	11.04				
	3/13/2019	11:13:00	11.19	10.00	1.8	14.2	0.6
12	3/13/2019	11:27:00	10.99				
	3/13/2019	11:37:00	11.14	10.00	1.8	14.2	0.5

PERCOLATION TEST
No. P-3

Project: New Home Depot Store - Mission Valley
Location: San Diego
Boring Location K-8

Project No. D050R0.01
Test Date: 3/13/2019



A. Top of Pipe Above Ground	11 Inches
B. Depth of Hole	184 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	31 Inches
F. Pipe Length	193 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: 3/12/2019 to 35 inches from bottom at 4:05pm
Checked 3/13/2019 no water at 8:10 am

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	3/13/2019	8:28:00	14.51				
	3/13/2019	8:38:00	14.75	10.00	2.88	8.9	0.6
2	3/13/2019	8:38:00	14.75				
	3/13/2019	8:48:00	14.89	10.00	1.68	15.2	0.4
3	3/13/2019	8:48:00	14.89				
	3/13/2019	8:58:00	15	10.00	1.32	19.4	0.3
4	3/13/2019	8:58:00	15				
	3/13/2019	9:08:00	15.22	10.00	2.64	9.7	0.8
5	3/13/2019	9:08:00	15.22				
	3/13/2019	9:18:00	15.31	10.00	1.08	23.7	0.4
6	3/13/2019	9:18:00	15.31				
	3/13/2019	9:28:00	15.38	10.00	0.84	30.5	0.3
7	3/13/2019	9:42:00	15.02				
	3/13/2019	9:52:00	15.14	10.00	1.44	17.8	0.4
8	3/13/2019	9:55:00	15.03				
	3/13/2019	10:05:00	15.14	10.00	1.32	19.4	0.4
9	3/13/2019	10:07:00	14.99				
	3/13/2019	10:17:00	15.1	10.00	1.32	19.4	0.4
10	3/13/2019	10:20:00	15.04				
	3/13/2019	10:30:00	15.15	10.00	1.32	19.4	0.4
11	3/13/2019	10:54:00	15.06				
	3/13/2019	11:04:00	15.17	10.00	1.32	19.4	0.4
12	3/13/2019	11:10:00	15.01				
	3/13/2019	11:20:00	15.12	10.00	1.32	19.4	0.4

APPENDIX E

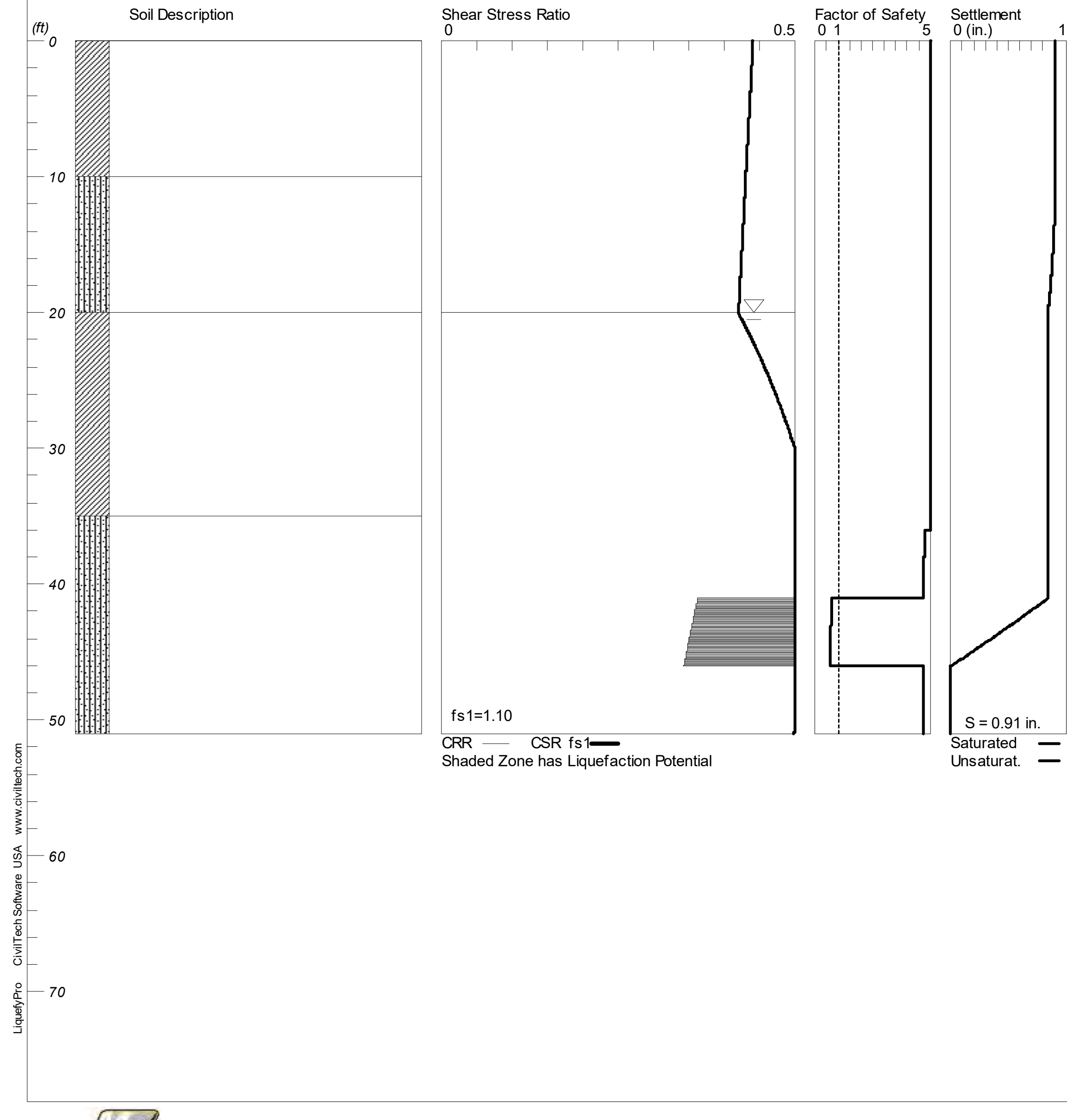
RESULTS OF LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

Mission Valley Home Depot

Hole No.=A-2 Water Depth=20 ft Surface Elev.=54

Magnitude=6.89
Acceleration=0.617g



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LIQUEFACTION ANALYSIS SUMMARY

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Font: Courier New, Regular, Size 8 is recommended for this report.

Licensed to , 1/8/2020 10:37:24 AM

Input File Name: F:\ENG\Geotech\D050R0.01 - New Store - Mission Valley - San Diego\Computations\Seismic Calcs\A-2 update to 7-16.liq

Title: Mission Valley Home Depot

Subtitle: 1895 Camino del Rio S - San Diego

Surface Elev.=54

Hole No.=A-2

Depth of Hole= 51.00 ft

Water Table during Earthquake= 20.00 ft

Water Table during In-Situ Testing= 29.50 ft

Max. Acceleration= 0.62 g

Earthquake Magnitude= 6.89

Input Data:

Surface Elev.=54

Hole No.=A-2

Depth of Hole=51.00 ft

Water Table during Earthquake= 20.00 ft

Water Table during In-Situ Testing= 29.50 ft

Max. Acceleration=0.62 g

Earthquake Magnitude=6.89

No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.

2. Settlement Analysis Method: Ishihara / Yoshimine

3. Fines Correction for Liquefaction: Idriss/Seed

4. Fine Correction for Settlement: During Liquefaction*

5. Settlement Calculation in: All zones*

6. Hammer Energy Ratio, Ce = 1.4

7. Borehole Diameter, Cb= 1.15

8. Sampling Method, Cs= 1.2

9. User request factor of safety (apply to CSR) , User= 1.1

Plot one CSR curve (fs1=User)

10. Use Curve Smoothing: No

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	13.00	115.00	NoLiq
6.00	10.00	115.00	NoLiq
11.00	18.00	115.00	30.00
16.00	16.00	115.00	30.00
21.00	14.00	115.00	NoLiq
26.00	61.00	115.00	NoLiq
31.00	15.00	115.00	NoLiq
36.00	24.00	115.00	15.00
41.00	16.00	115.00	15.00
46.00	50.00	115.00	15.00
51.00	50.00	115.00	15.00

Output Results:

Settlement of Saturated Sands=0.84 in.

Settlement of Unsaturated Sands=0.07 in.

Total Settlement of Saturated and Unsaturated Sands=0.91 in.

Differential Settlement=0.455 to 0.600 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.44	5.00	0.84	0.07	0.91
1.00	2.00	0.44	5.00	0.84	0.07	0.91
2.00	2.00	0.44	5.00	0.84	0.07	0.91
3.00	2.00	0.44	5.00	0.84	0.07	0.91
4.00	2.00	0.44	5.00	0.84	0.07	0.91
5.00	2.00	0.44	5.00	0.84	0.07	0.91
6.00	2.00	0.43	5.00	0.84	0.07	0.91
7.00	2.00	0.43	5.00	0.84	0.07	0.91
8.00	2.00	0.43	5.00	0.84	0.07	0.91
9.00	2.00	0.43	5.00	0.84	0.07	0.91
10.00	2.00	0.43	5.00	0.84	0.07	0.91
11.00	2.48	0.43	5.00	0.84	0.07	0.91
12.00	2.48	0.43	5.00	0.84	0.06	0.91

13.00	2.48	0.43	5.00	0.84	0.06	0.90
14.00	2.48	0.43	5.00	0.84	0.05	0.90
15.00	2.48	0.43	5.00	0.84	0.05	0.89
16.00	2.48	0.42	5.00	0.84	0.04	0.88
17.00	2.48	0.42	5.00	0.84	0.03	0.88
18.00	2.48	0.42	5.00	0.84	0.03	0.87
19.00	2.48	0.42	5.00	0.84	0.01	0.86
20.00	2.48	0.42	5.00	0.84	0.00	0.84
21.00	2.00	0.43	5.00	0.84	0.00	0.84
22.00	2.00	0.44	5.00	0.84	0.00	0.84
23.00	2.00	0.45	5.00	0.84	0.00	0.84
24.00	2.00	0.46	5.00	0.84	0.00	0.84
25.00	2.00	0.47	5.00	0.84	0.00	0.84
26.00	2.00	0.47	5.00	0.84	0.00	0.84
27.00	2.00	0.48	5.00	0.84	0.00	0.84
28.00	2.00	0.49	5.00	0.84	0.00	0.84
29.00	2.00	0.49	5.00	0.84	0.00	0.84
30.00	2.00	0.50	5.00	0.84	0.00	0.84
31.00	2.00	0.50	5.00	0.84	0.00	0.84
32.00	2.00	0.51	5.00	0.84	0.00	0.84
33.00	2.00	0.51	5.00	0.84	0.00	0.84
34.00	2.00	0.51	5.00	0.84	0.00	0.84
35.00	2.00	0.51	5.00	0.84	0.00	0.84
36.00	2.00	0.51	5.00	0.84	0.00	0.84
37.00	2.44	0.51	4.75	0.84	0.00	0.84
38.00	2.43	0.51	4.74	0.84	0.00	0.84
39.00	2.42	0.51	4.72	0.84	0.00	0.84
40.00	2.42	0.51	4.71	0.84	0.00	0.84
41.00	2.41	0.51	4.70	0.84	0.00	0.84
42.00	0.36	0.51	0.70*	0.69	0.00	0.69
43.00	0.35	0.51	0.69*	0.52	0.00	0.52
44.00	0.35	0.51	0.69*	0.36	0.00	0.36
45.00	0.35	0.51	0.68*	0.18	0.00	0.18
46.00	0.34	0.51	0.67*	0.01	0.00	0.01
47.00	2.38	0.51	4.69	0.00	0.00	0.00
48.00	2.37	0.51	4.69	0.00	0.00	0.00
49.00	2.37	0.50	4.70	0.00	0.00	0.00
50.00	2.36	0.50	4.71	0.00	0.00	0.00
51.00	2.35	0.50	4.72	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone

(F.S. is limited to 5,   CRR is limited to 2,   CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)

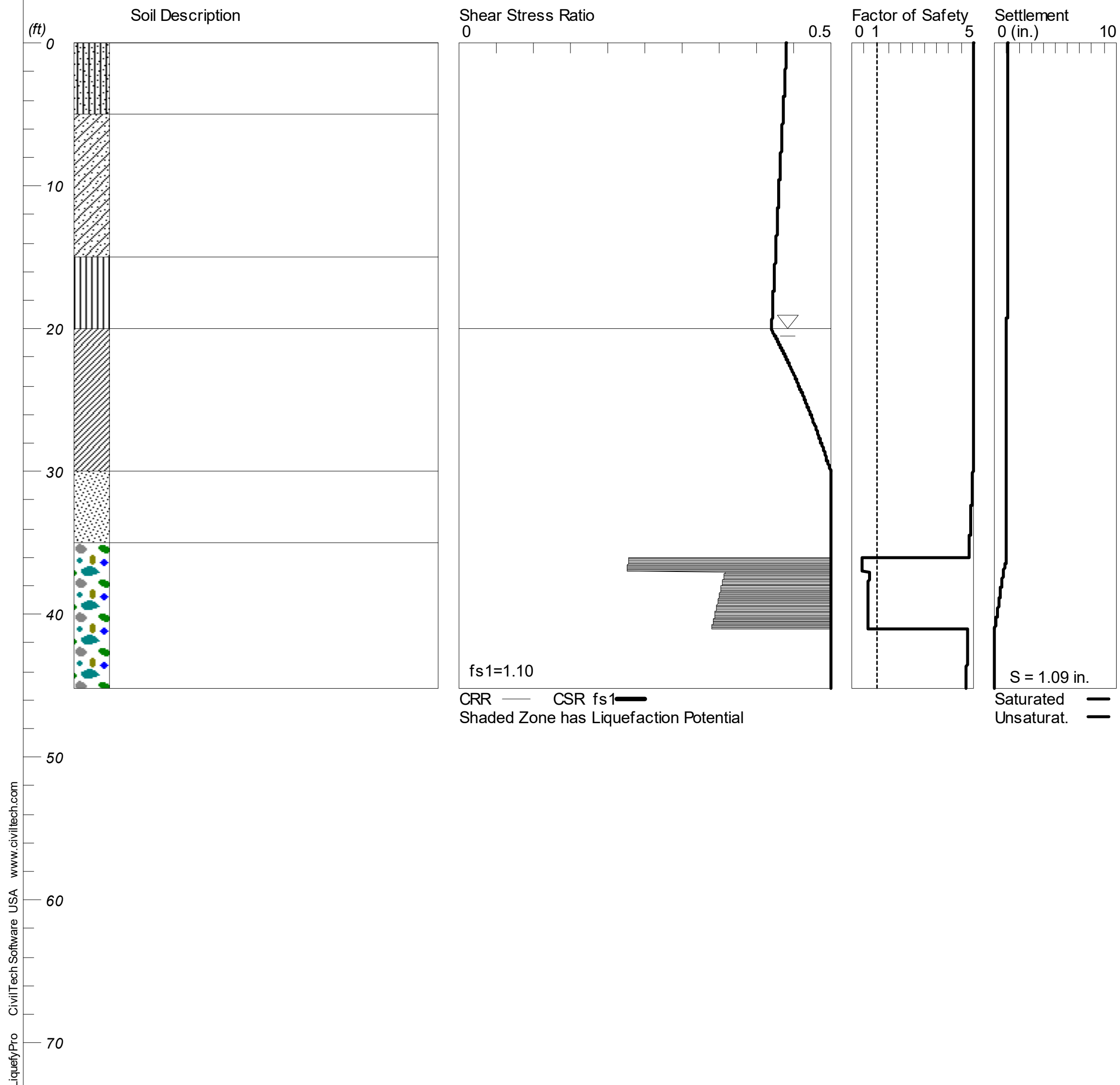
CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

Mission Valley Home Depot

Hole No.=M-8C   Water Depth=20 ft   Surface Elev.=51 ft

Magnitude=6.89
Acceleration=0.617g



Moore Twining Associates, Inc. 1895 Camino del Rio S - San Diego

Plate A-1

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LIQUEFACTION ANALYSIS SUMMARY

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Font: Courier New, Regular, Size 8 is recommended for this report.

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Input File Name: F:\ENG\Geotech\D050R0.01 - New Store - Mission Valley - San Diego\Computations\Seismic Calcs\M-8C.liq

Title: Mission Valley Home Depot

Subtitle: 1895 Camino del Rio S - San Diego

Surface Elev.=51 ft

Hole No.=M-8C

Depth of Hole= 45.20 ft

Water Table during Earthquake= 20.00 ft

Water Table during In-Situ Testing= 26.00 ft

Max. Acceleration= 0.62 g

Earthquake Magnitude= 6.89

Input Data:

Surface Elev.=51 ft

Hole No.=M-8C

Depth of Hole=45.20 ft

Water Table during Earthquake= 20.00 ft

Water Table during In-Situ Testing= 26.00 ft

Max. Acceleration=0.62 g

Earthquake Magnitude=6.89

No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.

2. Settlement Analysis Method: Ishihara / Yoshimine

3. Fines Correction for Liquefaction: Idriss/Seed

4. Fine Correction for Settlement: During Liquefaction*

5. Settlement Calculation in: All zones*

6. Hammer Energy Ratio, Ce = 1.6

7. Borehole Diameter, Cb= 1.15

8. Sampling Method, Cs= 1.2

9. User request factor of safety (apply to CSR) , User= 1.1

Plot one CSR curve (fs1=User)

10. Use Curve Smoothing: No

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	15.00	115.00	30.00
6.00	15.00	115.00	30.00
11.00	14.00	115.00	60.00
16.00	18.00	115.00	60.00
21.00	18.00	115.00	NoLiq
26.00	22.00	115.00	NoLiq
28.00	37.00	115.00	NoLiq
30.00	35.00	115.00	4.00
33.00	18.00	115.00	4.00
36.00	10.00	115.00	4.00
37.00	15.00	115.00	4.00
41.00	50.00	115.00	4.00
45.00	50.00	115.00	4.00

Output Results:

Settlement of Saturated Sands=1.01 in.

Settlement of Unsaturated Sands=0.08 in.

Total Settlement of Saturated and Unsaturated Sands=1.09 in.

Differential Settlement=0.545 to 0.720 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.48	0.44	5.00	1.01	0.08	1.09
1.00	2.48	0.44	5.00	1.01	0.08	1.09
2.00	2.48	0.44	5.00	1.01	0.08	1.09
3.00	2.48	0.44	5.00	1.01	0.08	1.09
4.00	2.48	0.44	5.00	1.01	0.08	1.09
5.00	2.48	0.44	5.00	1.01	0.07	1.08
6.00	2.48	0.43	5.00	1.01	0.07	1.08
7.00	2.48	0.43	5.00	1.01	0.07	1.08
8.00	2.48	0.43	5.00	1.01	0.07	1.08
9.00	2.48	0.43	5.00	1.01	0.06	1.07
10.00	2.48	0.43	5.00	1.01	0.06	1.07

11.00	2.48	0.43	5.00	1.01	0.06	1.07
12.00	2.48	0.43	5.00	1.01	0.05	1.06
13.00	2.48	0.43	5.00	1.01	0.05	1.06
14.00	2.48	0.43	5.00	1.01	0.04	1.05
15.00	2.48	0.43	5.00	1.01	0.03	1.05
16.00	2.48	0.42	5.00	1.01	0.03	1.04
17.00	2.48	0.42	5.00	1.01	0.02	1.03
18.00	2.48	0.42	5.00	1.01	0.02	1.03
19.00	2.48	0.42	5.00	1.01	0.01	1.02
20.00	2.48	0.42	5.00	1.01	0.00	1.01
21.00	2.00	0.43	5.00	1.01	0.00	1.01
22.00	2.00	0.44	5.00	1.01	0.00	1.01
23.00	2.00	0.45	5.00	1.01	0.00	1.01
24.00	2.00	0.46	5.00	1.01	0.00	1.01
25.00	2.00	0.47	5.00	1.01	0.00	1.01
26.00	2.00	0.47	5.00	1.01	0.00	1.01
27.00	2.00	0.48	5.00	1.01	0.00	1.01
28.00	2.00	0.49	5.00	1.01	0.00	1.01
29.00	2.00	0.49	5.00	1.01	0.00	1.01
30.00	2.00	0.50	5.00	1.01	0.00	1.01
31.00	2.50	0.50	4.97	1.01	0.00	1.01
32.00	2.49	0.51	4.93	1.01	0.00	1.01
33.00	2.49	0.51	4.90	1.01	0.00	1.01
34.00	2.48	0.51	4.87	1.01	0.00	1.01
35.00	2.47	0.51	4.85	1.01	0.00	1.01
36.00	2.47	0.51	4.82	1.01	0.00	1.01
37.00	0.23	0.51	0.44*	0.73	0.00	0.73
38.00	0.35	0.51	0.69*	0.55	0.00	0.55
39.00	0.35	0.51	0.68*	0.37	0.00	0.37
40.00	0.34	0.51	0.67*	0.19	0.00	0.19
41.00	0.34	0.51	0.66*	0.01	0.00	0.01
42.00	2.43	0.51	4.74	0.00	0.00	0.00
43.00	2.42	0.51	4.74	0.00	0.00	0.00
44.00	2.42	0.51	4.74	0.00	0.00	0.00
45.00	2.41	0.51	4.73	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone

(F.S. is limited to 5,   CRR is limited to 2,   CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)

CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

Job No. 198075-2

Report on:
Energy Measurement for Dynamic Penetrometers – Standard Penetration Test
Truck 75192H1 CME 75 Drill Rig Calibration
Lemoore, CA

Prepared for Moore Twining Associates, Inc.

By Camilo Alvarez, PE & Diego A. Campos

July 10, 2019

GRL engineers, inc.
Dynamic Measurements and Analyses

GRL Dynamic Measurements and Analyses
engineers, inc.

July 11, 2019

Allen Bushey
Moore Twining Associates, Inc

Re: Energy Measurement for Dynamic Penetrometers
Standard Penetration Test (SPT) on Truck 75192H1 CME 75 drill rig
Lemoore, CA. GRL Job No. 198075-2

Dear Mr. Allen Bushey:

This report transmits our findings from energy measurements and related data analysis conducted by GRL Engineers, Inc. (GRL) for your Truck 75192H1 mounted CME 75 drill rig located in Lemoore, CA. One automatic hammer and penetrometer system was monitored during Standard Penetration Test (SPT) of the test borehole. Dynamic testing summarized in this report was conducted on July 10, 2019.

The purpose in collecting the SPT energy measurements was to compute the energy transfer efficiency for a single SPT hammer. To meet this objective, an 8G Model, Pile Driving Analyzer® (PDA) utilizing the SPT Analyzer feature was used to acquire and process the dynamic test data. Additional information regarding the testing equipment and analytical procedures is provided in Appendix A.

Test Sequence

Using an instrumented AW-J rod for a Truck 75192H1 mounted CME 75 drill rig at test borehole, energy measurements were made at five sample depths for the drill rig. From BH1, the dynamic measurements were obtained from sample depths of 2.0, 5.0, 10.0, 15.0 and 20.0 ft. Each sample depth consisted of energy measurements of 18 inches of driving.

516 Crane Boulevard • Los Angeles, CA 90065 USA
323.441.0965 • fax 323.441.0975 • www.GRLengineers.com

Energy Transfer Measurements

A Model 8G Pile Driving Analyzer was used to take measurements of strain and acceleration. The strain and acceleration signals were conditioned and converted to forces and velocities by the PDA. The PDA interprets the measured dynamic data according to the Case Method equations. Force and velocity records from the PDA were also viewed graphically on an LCD screen to evaluate data quality. All force and velocity records were also digitally stored for subsequent analysis.

The maximum energy transferred to the rod (EMX) was calculated by integrating both the force and velocity records over time as follows:

$$EMX = \int F(t)V(t)dt$$

Where: $F(t)$ = the force at time t
$V(t)$ = the velocity at time t

The energy transfer ratio or efficiency is computed by dividing EMX by the theoretical SPT hammer energy of 350 lb-ft (computed from the product of the hammer weight, assumed to be the standard 140 lbs, and the fall height, assumed to be 2.5 ft). The SPT N values can then be corrected for a nominal 60% transfer efficiency, $N_{60,}$ as follows:

$$N_{60} = (e_m / 60)\ N_m$$

Where: e_m = the measured transfer ratio (ETR)
N_m = the measured SPT "N" value

Conclusions

Table 1 in Appendix B presents a summary of the average transferred energy and the energy transfer ratio for the single drill rig at each sample depth calculated using the *EMX* equation. Included in Table 1 are also average values of the hammer operating rate, maximum impact force and maximum velocity of the rod. The overall performance, which represents the average of data from all sample depths for each rig/rod type is also shown. Complete data, including the maximum, minimum and standard deviation for each sampling depth, is included in Appendix B.

For the Truck 75192H1 mounted CME 75 drill rig-RIG 156, the average energy transfer ratio from individual sample depths ranged from 81.3 to 96.9%.

The average, overall transfer ratio (for all sampling depths weighted by N-values for each sample) were as follows:

SPT Rig (*Serial Number*)	Overall Transfer Efficiency	Hammer Operating Rate (BPM)
Truck 75192H1 CME 75 drill rig 156	88.9%	40.4

Presented N_{60} values, provided in the Table 1 in Appendix B, does not account for any required corrections such as those for overburden or sampling spoon.

We appreciate the opportunity to be of assistance to you. Please do not hesitate to contact us if you have any questions regarding this report, or if we may be of further service.

Respectfully,
GRL Engineers, Inc.

REGISTERED PROFESSIONAL ENGINEER
Camilo A Alvarez
No. 67938
Exp. 6/30/21
Civil
STATE OF CALIFORNIA

Camilo Alvarez, P.E.
Senior Engineer

Diego Campos, EIT
Engineer

APPENDIX A
AN INTRODUCTION INTO SPT DYNAMIC PILE TESTING

1. BACKGROUND

The Standard Penetration Test is frequently conducted as an in-situ assessment of soil strength. This test requires that a 140 lb weight is dropped 30 inches onto a drive rod at whose bottom a sampler is usually installed. The sampler is driven for 18 inches; the number of blows required for the last 12 inches of driving is the so-called N-value. The N-value may be used as a strength indicator for foundation design or as a means of assessing the liquefaction potential of soils.

Obviously, the SPT hammer efficiency is an important consideration when using the N-values for design purposes. Measurements have indicated that the energy in the drive rod is sometimes only 30% and and may reach 90% of the potential or rated energy of the SPT hammer (E-rated = 0.35 kip-ft or 0.475 kJ). The type of hammer used to drive the rod is the main reason for these variations. On the average, the energy in the drive rod is 60% of the standard rated energy.

Because of the variability of energy, methods based on N-values are considered unreliable. However, measurements during SPT testing using the Case Method can be done on a routine basis and these measurements yield the transferred energy values. With measured energy, EMX, known, an adjustment of the measured N-value, N_m, can be made as follows.

$$N_{60} = N_m \left[E_m / (0.6E_r)\right] \qquad (1)$$

Thus, if the measured energy value is equal to the normally expected transferred energy of 60% of E-rated then the adjusted and measured N-values are identical. On the other hand, if the measured energy is only 30% then the adjusted blow count will be reduced by 50%.

2. DYNAMIC TESTING AND ANALYSIS METHODS APPLIED TO SPT

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (e.g. a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. Thus, the method is also referred to as a “High Strain Method”. The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then a calculation of various quantities. Conveniently, for SPT applications, the measurements and analyses are done by a single piece of equipment: the SPT Analyzer. The Pile Driving Analyzer® (PDA) is also suitable to perform these measurements and data processing.

A related analysis method is the “Wave Equation Analysis” which calculates a relationship between bearing capacity, pile stresses, transferred energy and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data. This program can be used very effectively to simulate the SPT driving process.

3. MEASUREMENTS

GRL uses equipment manufactured by Pile Dynamics, Inc. The system includes either an SPT-Analyzer™ (SPTA) or a Pile Driving Analyzer® (PDA), an instrumented rod section and two accelerometers. SPT energy testing is very closely related to and borrows procedures from dynamic pile testing. Those interested in the basis of the SPT energy testing method may obtain extensive literature on dynamic pile testing from GRL Engineers, Inc.

3.1 SPT Analyzer or Pile Driving Analyzer

The basis for the results calculated by the SPTA or PDA are strain and acceleration measured in an instrumented rod section. These signals are converted to rod top force, F(t), and rod top velocity, v(t). The SPTA or PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. The product of these two measurements is then integrated over time which yields the energy transferred to the instrumented section as a function of time (see Section 4.1).

For convenience and accuracy, strain measurements are usually taken on an instrumented section of SPT drive rod. Ideally, the section properties of the instrumented rod and those of the drive rod are the same, however, using subs, other sections can also be utilized.

For the instrumented section, PDI provides a force calibration in such a way that the output of the instrumented rod is directly calculated without the need for an accurate elastic modulus or cross sectional area of the rod section.

The acceleration measurements are often demanding in the SPT environment, because of high frequency and high acceleration motion components. An experienced measurement engineer, therefore, has to evaluate the quality of this data before final conclusions are drawn from the numerical results calculated by SPTA or PDA.

SPTA or PDA records are taken while the standard N-value is acquired in the conventional manner. This then allows a direct correlation between N-value and average transferred energy.

3.2 HPA

The SPT hammer's ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. The impact velocity results can be automatically processed with a PC or recorded on a strip chart. HPA measurements yield a hammer kinetic energy, but not the energy transferred to the drive rod.

4 RECORD EVALUATION BY SPTA OR PDA

4.1 HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = \int_o^t F(\tau)v(\tau)\, d\tau \qquad (2)$$

The maximum of the E(t) curve is often called **ENTHRU or EMX;** it is the most important quantity for an overall evaluation of the performance of a hammer and driving system. **EMX** allows for a classification of the hammer's performance when presented as, e_T, the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency.

$$e_T = EMX/E_R \qquad (3)$$

where E_R is the hammer manufacturer's rated energy value or 0.35 kip-ft (0.475 kJ) in the case of the SPT hammer.

Often in the SPT literature one finds also reference to the EF2 energy. This evaluation is based on assumed proportionality between force and velocity (see also Section 5):

$$v(t) = F(t) / Z \qquad (4)$$

where Z = EA/c is the pile impedance, E is the elastic modulus, A is the cross sectional area and c is the speed of the stress wave in the pile material..

Combining equations 2 and 4 leads to

$$EF(t) = \int_o^t F(\tau)^2 / Z\, d\tau \qquad (5)$$

The EF2 transferred energy value is the EF-value at the time t = 2L/c, where L is the drive rod length and c is the stress wave speed in steel (16,800 ft/s or 5,124 m/s). Since the force is easier to measure than both force and velocity, Equation 5 is preferred by some test engineers. However, the EF method is fraught with errors and certain correction factors have to be applied to make it approximately correct. Among the error sources are the following:

- Proportionality is often violated prior to time 2L/c. The proportionality between force and velocity in a downward traveling wave only holds if the wave does not encounter a disturbance prior to reflecting off the pile toe. Such disturbances include a change in cross sectional area, an open or loose splice or joint, or resistance along the shaft.

- Using only one force measurement precludes a data quality check based on the proportionality between force and velocity. Thus, a force measurement that is for some reason in error may not be detectable, which will lead to errors in the EF2 value. Data quality checks will be discussed further in Section 5.

The use if EF2 is therefore not recommended but it is often included in result presentations for the sake of completeness.

4.2 STRESSES

During SPT monitoring, it is also of interest to monitor compressive stresses at both the top of the drive rod and at its bottom.

At the pile top (location of sensors) the maximum compression stress averaged over the rod's cross section, **CSX**, is directly obtained from the measurements. Note that this stress value refers to the instrumented section. If the rod has a different cross sectional area then the stress in the rod will be different from CSX.

The SPTA or PDA can also calculate, in an approximate manner, the force at the rod bottom, **CFB**. To obtain the corresponding stress, this force value should be divided by the appropriate cross sectional area, e.g. by the rod area just above the sampler or by the sampler area itself. Of course, non-uniform stress components as they might occur at the sampler tip due to a sloping rock are not considered in this calculation.

5. DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing SPTA or PDA tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to the so-called proportionality relationship.

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the rod, force and velocity measured at its top are proportional

$$F = v\,Z \tag{5}$$

where Z is again the pile impedance, Z = EA/c. This relationship can also be expressed in terms of stress

$$\sigma = F/A = v\,(E/c) \tag{6}$$

or strain

$$\epsilon = \sigma/E = v\,/\,c \tag{7}$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements but is only truly meaningful for perfectly uniform rods. Open or loose splices, for example, will lead to a non-proportionality. For SPT rods it is fortunate that usually no soil resistance acts along the shaft and for that reason, proportionality can exist until the stress wave returns from sampler top or rod bottom unless connectors are not sufficiently tightened or have a significant mass.

Velocity data quality can also be checked by looking at the final displacement, DFN, which is calculated from the acceleration by double integration. If the calculated final displacement is much higher or lower than indicated by the N-value, the accelerometer attachment may be loose or the sensor may be faulty. If major drift in the velocity is observed, the EMX value may be in error, even though proportionality from impact to time 2L/c exists. In this case, it may be useful to evaluate the energy transferred to the drill rod at time 2L/c, which is calculated by the PDA or SPTA as the E2E quantity.

Appendix B

SPT Analyses Results

Summary of SPT Test Results

Project: CME 75 - RIG 156 - 75192H1, Test Date: 7/10/2019

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
8.00	**4-6-8**	**14**	**20**	29	17.6	44.4	320	91.5
9.00	**3-3-3**	**6**	**8**	27	18.2	37.2	284	81.3
12.50	**1-3-7**	**10**	**14**	28	18.0	40.0	339	96.9
18.50	**3-1-10**	**11**	**16**	30	17.5	40.0	307	87.6
23.50	**2-8-13**	**21**	**31**	29	17.0	39.1	302	86.2
			Overall Average Values:	29	17.5	40.4	311	88.9
			Standard Deviation:	1	0.6	2.3	20	5.7
			Overall Maximum Value:	31	19.1	44.7	373	106.6
			Overall Minimum Value:	27	16.6	37.1	271	77.6

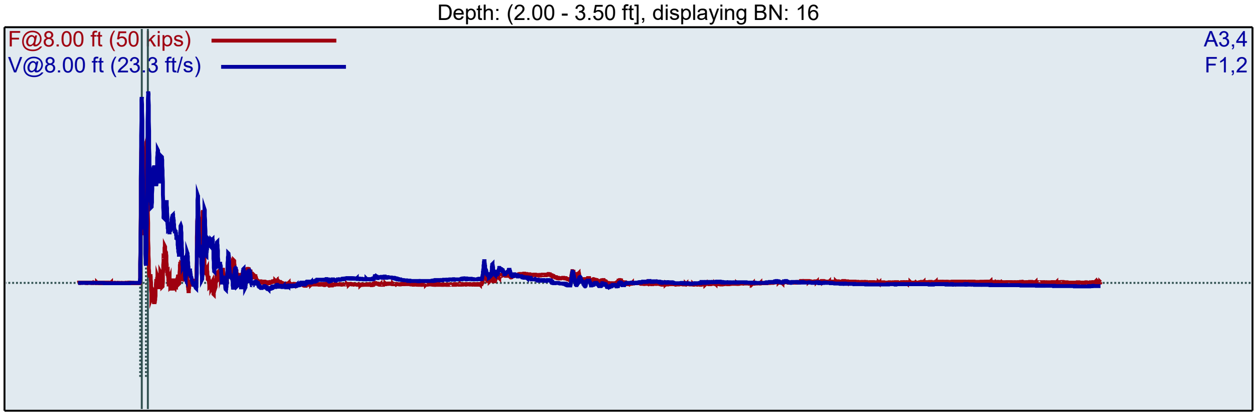
CME 75 - RIG 156 - 75192H1
DC

At 2 feet
Test date: 7/10/2019

AR: 1.20 in^2
LE: 8.00 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (2.00 - 3.50 ft], displaying BN: 16



F1 : [217AWJ2] 214.53 PDICAL (1.03) FF6
F2 : [217AWJ1] 214 PDICAL (1.03) FF6

A3 (PR): [K4695] 378 mv/6.4v/5000g (0.97) VF6
A4 (PR): [K1388] 384 mv/6.4v/5000g (0.97) VF6

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
1	4	28	17.1	1.9	298	85.1
2	4	29	17.6	44.3	308	88.0
3	4	29	17.5	44.6	323	92.3
4	4	28	17.3	44.0	325	92.9
5	6	28	17.3	44.6	316	90.3
6	6	29	17.7	44.0	337	96.3
7	6	28	17.3	44.7	310	88.6
8	6	28	17.1	44.6	310	88.7
9	6	29	17.5	44.3	331	94.6
10	6	29	17.6	44.6	320	91.4
11	8	29	17.4	44.4	330	94.2
12	8	29	17.3	44.5	325	92.8
13	8	29	17.9	44.4	335	95.7
14	8	29	17.8	44.4	315	90.1
15	8	30	17.5	44.4	316	90.2
16	8	28	17.5	44.4	309	88.3
17	8	28	17.9	44.5	311	88.8
18	8	29	18.0	44.4	321	91.6
	Average	29	17.6	44.4	320	91.5
	Std Dev	1	0.3	0.2	9	2.6
	Maximum	30	18.0	44.7	337	96.3
	Minimum	28	17.1	44.0	309	88.3

N-value: 14

BN: 18 4-6-8

Sample Interval Time: 22.97 seconds.

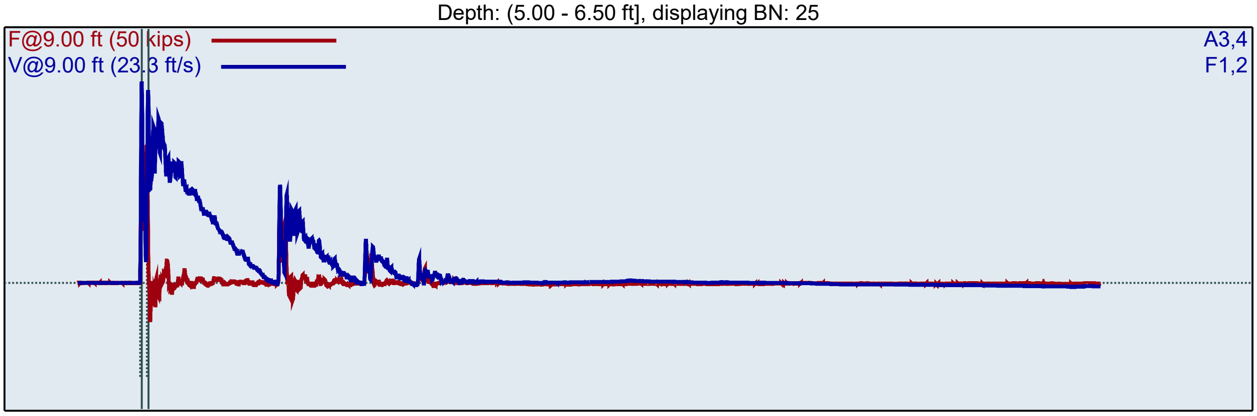
CME 75 - RIG 156 - 75192H1
DC

At 2 feet
Test date: 7/10/2019

AR: 1.20 in^2
LE: 9.00 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (5.00 - 6.50 ft], displaying BN: 25



F1 : [217AWJ2] 214.53 PDICAL (1) FF6
F2 : [217AWJ1] 214 PDICAL (1) FF6

A3 (PR): [K4695] 378 mv/6.4v/5000g (1) VF6
A4 (PR): [K1388] 384 mv/6.4v/5000g (1) VF6

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
19	3	26	17.8	5.2	270	77.3
20	3	26	17.9	36.8	252	72.0
21	3	27	17.7	37.1	262	74.9
22	3	27	17.9	37.1	275	78.5
23	3	27	18.2	37.2	288	82.3
24	3	27	18.5	37.2	274	78.2
25	3	27	18.4	37.1	285	81.3
26	3	28	18.2	37.2	291	83.1
27	3	27	18.2	37.2	295	84.3
	Average	27	18.2	37.2	284	81.3
	Std Dev	0	0.2	0.1	8	2.3
	Maximum	28	18.5	37.2	295	84.3
	Minimum	27	17.9	37.1	274	78.2

N-value: 6

BN: 27 3-3-3

Sample Interval Time: 12.91 seconds.

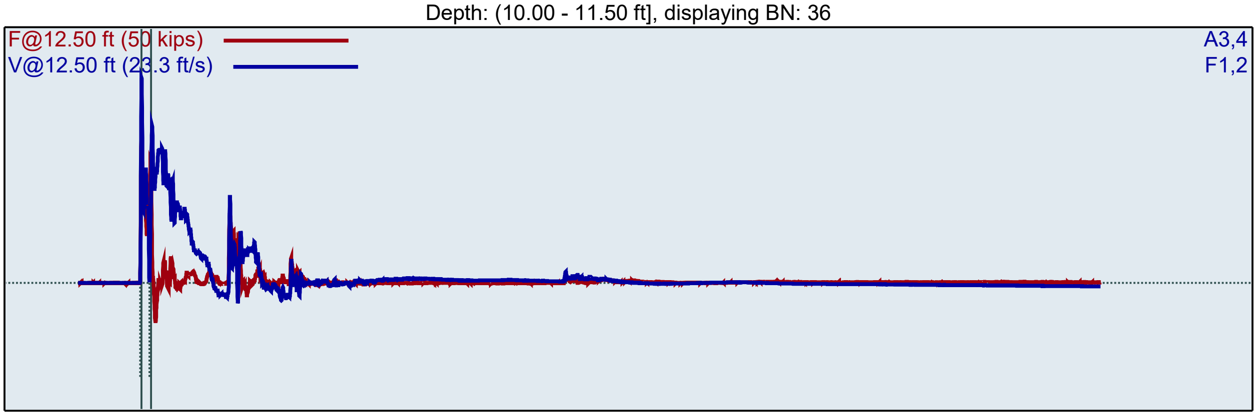
CME 75 - RIG 156 - 75192H1
DC

At 2 feet
Test date: 7/10/2019

AR: 1.20 in^2
LE: 12.50 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (10.00 - 11.50 ft], displaying BN: 36



F1 : [217AWJ2] 214.53 PDICAL (1) FF6
F2 : [217AWJ1] 214 PDICAL (1) FF6

A3 (PR): [K4695] 378 mv/6.4v/5000g (1) VF6
A4 (PR): [K1388] 384 mv/6.4v/5000g (1) VF6

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
28	1	27	17.7	1.9	308	87.9
29	3	28	17.5	39.6	314	89.8
30	3	29	17.8	40.1	320	91.4
31	3	28	17.1	40.2	329	94.1
32	7	28	17.3	40.2	322	91.9
33	7	28	17.4	40.2	321	91.7
34	7	28	17.9	40.3	350	100.0
35	7	29	18.0	40.0	349	99.7
36	7	28	18.7	39.8	349	99.8
37	7	29	18.9	40.0	365	104.3
38	7	29	19.1	39.9	373	106.6
	Average	28	18.0	40.0	339	96.9
	Std Dev	0	0.7	0.2	20	5.6
	Maximum	29	19.1	40.3	373	106.6
	Minimum	28	17.1	39.6	314	89.8

N-value: 10

BN: 38 1-3-7

Sample Interval Time: 14.99 seconds.

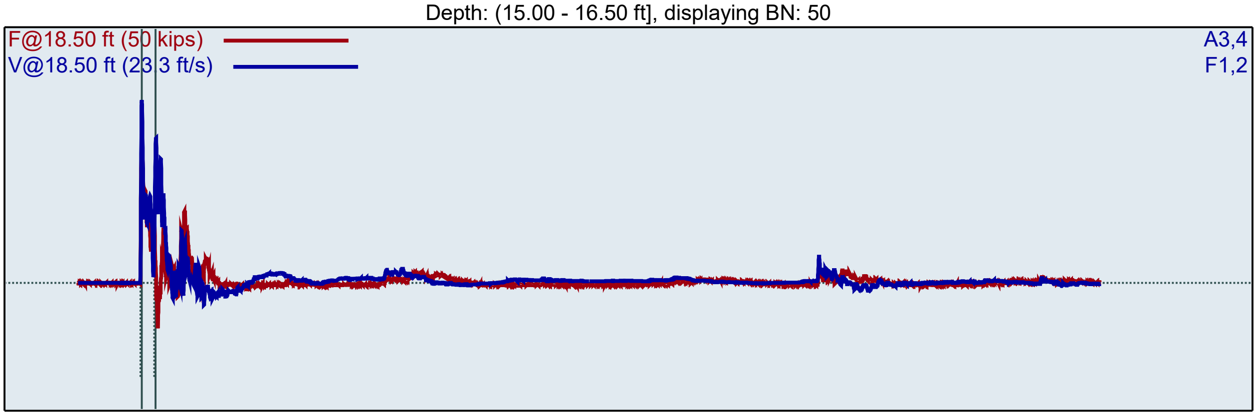
CME 75 - RIG 156 - 75192H1
DC

At 2 feet
Test date: 7/10/2019

AR: 1.20 in^2
LE: 18.50 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (15.00 - 16.50 ft], displaying BN: 50



F1 : [217AWJ2] 214.53 PDICAL (1) FF1
F2 : [217AWJ1] 214 PDICAL (1) FF1

A3 (PR): [K4695] 378 mv/6.4v/5000g (1) VF1
A4 (PR): [K1388] 384 mv/6.4v/5000g (1) VF1

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
39	3	28	18.2	13.7	264	75.6
40	3	31	18.7	40.4	278	79.5
41	3	31	18.5	40.5	279	79.6
42	1	29	18.5	40.5	271	77.6
43	10	30	18.1	40.2	310	88.6
44	10	29	17.1	40.2	313	89.5
45	10	29	17.2	40.1	313	89.4
46	10	29	17.1	40.2	309	88.3
47	10	31	18.5	39.9	321	91.8
48	10	29	17.0	39.9	304	86.7
49	10	31	18.0	39.8	310	88.6
50	10	29	16.7	39.8	302	86.3
51	10	29	16.9	39.7	314	89.8
52	10	29	17.1	39.7	306	87.3
	Average	30	17.5	40.0	307	87.6
	Std Dev	1	0.6	0.2	12	3.5
	Maximum	31	18.5	40.5	321	91.8
	Minimum	29	16.7	39.7	271	77.6

N-value: 11

BN: 52 3-1-10

Sample Interval Time: 19.45 seconds.

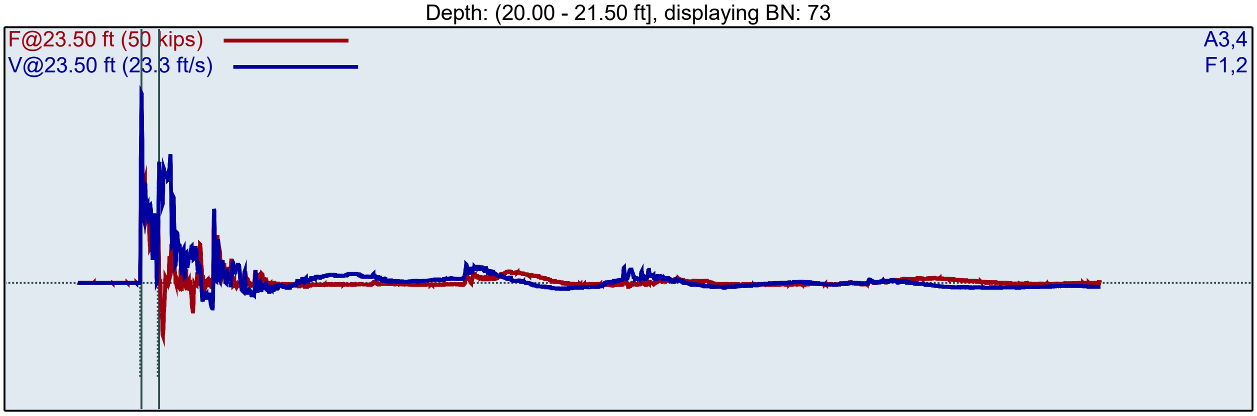
CME 75 - RIG 156 - 75192H1
DC

At 2 feet
Test date: 7/10/2019

AR: 1.20 in^2
LE: 23.50 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (20.00 - 21.50 ft], displaying BN: 73



F1 : [217AWJ2] 214.53 PDICAL (1) FF6
F2 : [217AWJ1] 214 PDICAL (1) FF6

A3 (PR): [K4695] 378 mv/6.4v/5000g (1) VF6
A4 (PR): [K1388] 384 mv/6.4v/5000g (1) VF6

BL#	BC	FMX	VMX	BPM	EFV	ETR
	/6"	kips	ft/s	bpm	ft-lb	%
53	2	28	17.3	4.6	277	79.1
54	2	29	16.8	38.3	273	78.0
55	8	29	17.0	38.8	291	83.2
56	8	28	17.0	38.8	281	80.4
57	8	28	16.6	39.1	283	80.8
58	8	29	16.7	39.0	308	88.0
59	8	29	17.0	39.1	296	84.4
60	8	29	16.8	39.1	305	87.1
61	8	28	16.7	39.1	289	82.5
62	8	29	17.4	39.2	307	87.7
63	13	28	16.7	39.3	306	87.3
64	13	29	17.2	39.3	308	87.9
65	13	29	17.2	39.1	304	86.7
66	13	28	16.8	39.2	304	86.8
67	13	29	17.0	39.2	304	86.7
68	13	29	16.8	39.2	305	87.2
69	13	29	17.4	39.2	304	86.7
70	13	29	17.5	39.2	310	88.7
71	13	29	17.2	39.2	303	86.4
72	13	29	17.6	39.2	306	87.3
73	13	29	17.3	39.2	312	89.1
74	13	29	17.0	39.1	318	90.7
75	13	28	16.9	39.2	296	84.5
	Average	29	17.0	39.1	302	86.2
	Std Dev	0	0.3	0.1	9	2.6
	Maximum	29	17.6	39.3	318	90.7
	Minimum	28	16.6	38.8	281	80.4

N-value: 21

BN: 75 2-8-13

Sample Interval Time: 33.74 seconds.