11111 Jefferson Boulevard

Hydrology, Hydraulics, and SUSMP Report

September 10th, 2020

Kimley»Horn

TABLE OF CONTENTS

EXHIBITS

APPENDICES

REFERENCES

Hydraulic Design Manual. Los Angeles County Flood Control District, March 1982.

Low Impact Standards Development Manual. County of Los Angeles Department of Public Works, February 2014.

1. HYDROLOGY AND HYDRAULICS

1.1 INTRODUCTION

The project site is comprised of approximately 3.43 acres, located at 11111 Jefferson Boulevard in Culver City, CA; see Exhibit A, Site Map below. The proposed site development includes a 5-story mixed-use building with 230 residential units and approximately 66,500 square feet (SF) of commercial space. The proposed site development will also include various ancillary improvements including 2 levels of at or above grade parking, 1 level of subterranean parking, landscaping, and stormwater conveyance and treatment structures/utilities. This document is provided in support of CEQA documentation, to provide a basis for the project’s stormwater design. It considers pre- and post-development conditions and provides information for the sizing storm drain pipes, catch basins, and stormwater detention structures

EXHIBIT A: SITE MAP

1.2 METHODOLOGY

The Los Angeles County Department of Public Works Hydrology Map was used to determine the approximate rainfall on the project site during a 50-year storm event. This hydrology map contains historical rainfall data from the previous 40 to 80 years at 99 rainfall gauges across the County. Los Angeles County HydroCalc was used to determine the pre- and post-development on-site flows. The calculations are included in Appendix A.

1.3 EXISTING DRAINAGE CONDITIONS

The project site’s elevation ranges from approximately 32 to 35 feet above mean sea level (MSL). Exhibit 1, Existing Drainage Area Map, depicts the 3 existing drainage areas. In the existing condition, stormwater runoff in Drainage Area 1 (DA-1) sheet flows to various inlets located in the site’s western portion at a slope of approximately 1%. The runoff is then routed to various parkway and curb drains and discharged to

Sepulveda Boulevard's public storm drain system. A portion of Drainage Area 2 (DA-2) runoff sheet flows to various inlets located in the site's eastern portion at a slope of approximately 1%. The runoff is then routed to various curb drains and discharged to Jefferson Boulevard's public storm drain system. The remaining portion of Drainage Area 2 runoff sheet flows directly to the Jefferson Boulevard public storm drain system at a slope of approximately 1%. Drainage Area 3 (DA-3) runoff is predominately roof drain runoff that flows through a downspout system and sheet flows to the Machado Road public storm drain system, or connects to a parkway drain and is discharged to Machado Road. See Table 1 below, which summarizes the pre-development conditions drainage areas and flows.

Table 1: Pre-Development Conditions Drainage Areas and Flows

Drainage Area	Area (Acres)	50-year Flow (CFS)
DA-1	2.05	4.38
DA-2	0.97	2.21
DA-3	0.41	1.10

The Sepulveda Boulevard and Machado Road storm drain systems ultimately connect to the Jefferson Boulevard storm drain system and become one 39" RCP storm drain line at the Sepulveda Boulevard and Jefferson Boulevard intersection per Los Angeles County Plan 275-613-D2. Refer to Exhibit 1, Existing Drainage Area Map, for additional information. See Table 2 below which summarizes the pre-development condition impervious and pervious areas.

Table 2: Pre-Development Conditions

Existing site area	**3.43**	**acres**
Percent impervious pre-construction	87	%
Percent pervious pre-construction	13	%

1.4 PROPOSED DRAINAGE CONDITIONS

The proposed site improvements include storm drainage infrastructure, including a roof drainage system and storm drain inlets internal to the site, to convey onsite runoff to an onsite stormwater treatment system with associated overflow structure. The proposed stormwater treatment system will consist of an underground stormwater detention structure, which will capture the stormwater runoff and reuse it onsite for landscaping irrigation. See Table 3 below which summarizes the pre- and post-development conditions impervious and pervious areas.

Table 3: Pre- and Post-Development Conditions

Existing site area	**3.43**	**acres**
Percent impervious before construction	87	%
Percent pervious before construction	13	%
Proposed site area	**3.43**	**acres**
Percent impervious after construction	80	%
Percent pervious after construction	20	%

A summary of the proposed drainage area and its associated flows is presented below. Exhibit 2, Proposed Drainage Area Map & LID Exhibit, depicts the proposed drainage area.

In the proposed condition, Area-1 has a Q50 flow of 6.62 cfs. The onsite runoff will be captured by the proposed roof drain system and proposed catch basins/area drains, and then conveyed via the proposed stormwater pipe network to the proposed stormwater treatment system and associated overflow structure. The overflow from the proposed underground detention structure will connect directly to the existing Jefferson Boulevard public storm drain system.

As compared to pre-development conditions, the proposed development's drainage area is assumed to remain 3.43 acres. The proposed development will however decrease the total Q50 runoff from pre-development to post-development conditions by 1.07 cfs (7.69 cfs pre-development vs 6.62 cfs post-development). This may be attributed to the increase in pervious surface area (i.e., landscape area) and increased flow path length due to the proposed roof area. See Table 4 below, which summarizes the pre- and post-development condition drainage areas and flows. Refer to Exhibits 1 and 2 for additional information.

Table 4: Pre- and Post-Development Drainage Areas and Flows

Drainage Area Number	Drainage Area (Acres)	50-year Flow (CFS)
Pre-Development Condition		
DA-1	2.05	4.38
DA-2	0.97	2.21
DA-3	0.41	1.10
Total Pre-Development	**3.43**	**7.69**
Post-Development Condition		
AREA-1	3.43	6.62
Total Post-Development	**3.43**	**6.62**

1.5 CONCLUSIONS

The project's proposed drainage system is designed to provide stormwater control and quality measures based on the current County of Los Angeles requirements. The site has been analyzed for adherence to stormwater runoff control for the 50-year (Q50) storm event per the Los Angeles County requirements.

The analysis shows that the proposed development will decrease the overall runoff flow rate from 7.69 to 6.62 cfs. Runoff will ultimately discharge to the existing Jefferson Boulevard storm drain system and be conveyed to the south, similar to pre-development conditions. Since the site's total runoff will decrease in the post-development condition, it has been determined that the downstream existing storm drain system has adequate capacity for the proposed development.

2. STANDARD URBAN STORMWATER MITIGATION PLAN (SUSMP)

2.1 SUSMP CALCULATIONS AND DESIGN CRITERIA

Proposed peak mitigated flows and volumes have been calculated using the Los Angeles County HydroCalc Calculator. Per the Los Angeles County Department of Public Works' requirements, the peak mitigated flows and mitigated volumes are based on the 85th Percentile of rainfall or ¾" rainfall, whichever is greater. Our analysis shows the 85th Percentile to be greater, which shows to be 1.1 inches.

The tributary area of the site of 3.43 acres including building roof area and proposed open space landscape and amenity areas. The peak mitigated discharge volume was calculated to be 10,051 cubic feet per the LA County HydroCalc Calculator. Proposed peak mitigated discharge and volume calculations are provided in Appendix B.

2.2 BMP FEASIBILITY ANALYSIS

Infiltration is the first option in Los Angeles County when screening potentially feasible SUSMP BMPs. Per the Report of Geotechnical Engineering Services by GeoDesign, Inc., dated April 26, 2019, a stormwater infiltration system is not recommended at the site due to the historic high groundwater level of 10 feet below grade surface and the site being in a liquefaction zone. See the soils report in Appendix D for more information.

Capture and Reuse is the next option in Los Angeles County when screening potentially feasible SUSMP BMPs. Per the current Landscape Concept, with 0.69 acres of proposed landscaping and a planting factor of 0.2 the Estimated Total Water Usage (ETWU) was calculated to be 81,264 gallons (10,864 CF). A design volume of 10,051 CF is less than the ETWU proving a Rainwater Harvesting Capture & Use System feasible for this project. See Appendix C for calculation details.

2.3 CONCLUSION

The proposed SUSMP BMP will be a Rainwater Harvesting System. The proposed storage volume of the BMP is 10,800 cubic feet, which provides an excess storage of 749 cubic feet. Therefore, 107.5% of the peak discharge will be mitigated onsite. This information is tabulated below in Table 5, SUSMP Summary Table. Stormwater will be pre-treated with a Contech CDS unit prior to entering the Rainwater Harvesting System and then be pumped from the detention structure to irrigation lines throughout the site. The proposed SUSMP BMP will mitigate the peak discharge volume based on the storage volume provided by the Rainwater Harvesting System. See Exhibit 2 and Appendix C for additional SUSMP BMP information.

Table 5: SUSMP Summary Table

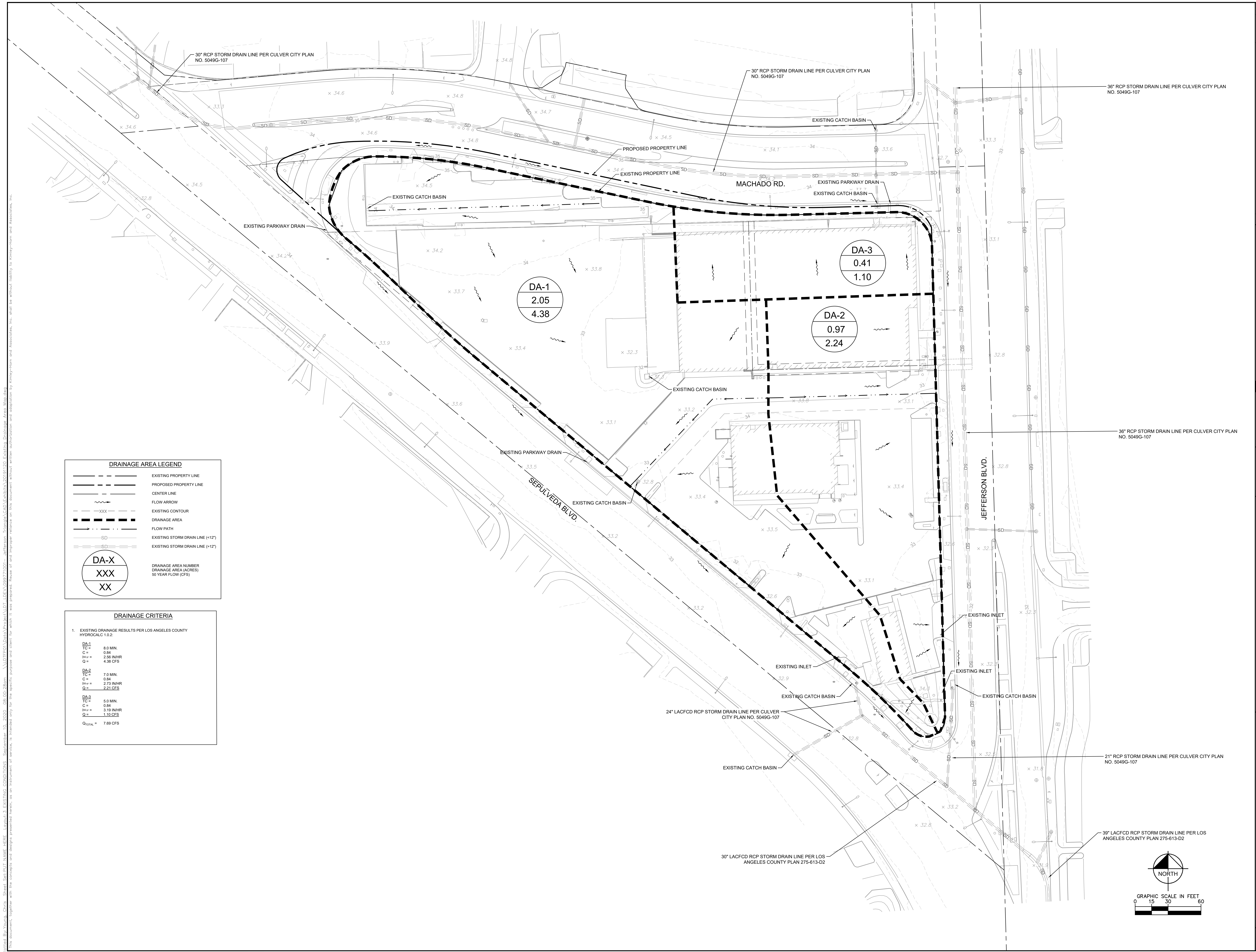
Total Project Area (AC)	BMP Tributary Area (AC)	Design Storm Volume (CF)	Storage Volume Provided (CF)	Volume Mitigated Onsite (%)
3.44	3.44	10,051	10,800	107.5

LIMITATIONS

Kimley-Horn was retained to perform a limited preliminary hydrology, hydraulics, and LID analysis and report to support the CEQA documentation, and has performed only those tasks specifically stated in our scope of services. This report may be relied upon only by Kimley-Horn's Client or by others with Client's permission. It is not intended for use by any other party.

The Client may use this report as part of its due diligence, but this report should not be used as the sole basis for the Client's decision making. We endeavored to research site development issues and constraints for the extent practical given the scope, budget, and schedule agreed to by the Client. Our assessment is based on information provided to Kimley-Horn by others (County of Los Angeles, City of Culver City, Caltrans, utility companies, etc.) and, therefore, is only accurate as of this writing, and is based on the Client's desires, which have been specifically disclosed to us. New issues may arise during development because of regulatory and policy changes, changed circumstances, or unforeseen conditions.

EXHIBIT 1 - Existing Drainage Area Map



DRAINAGE AREA LEGEND

━━ ‒ ‒ ━━	EXISTING PROPERTY LINE
━━ ‒ ‒ ━━	PROPOSED PROPERTY LINE
── ‒ ──	CENTER LINE
〰►	FLOW ARROW
‒ ‒ XXX ‒ ‒	EXISTING CONTOUR
■ ■ ■ ■	DRAINAGE AREA
──► · · ── · · ──	FLOW PATH
──SD──	EXISTING STORM DRAIN LINE (<12")
══SD══	EXISTING STORM DRAIN LINE (>12")
DA-X XXX XX	DRAINAGE AREA NUMBER DRAINAGE AREA (ACRES) 50 YEAR FLOW (CFS)

DRAINAGE CRITERIA

1. EXISTING DRAINAGE RESULTS PER LOS ANGELES COUNTY HYDROCALC 1.0.2:

DA-1
TC = 8.0 MIN.
C = 0.84
$I_{50\text{-}yr}$ = 2.56 IN/HR
Q = 4.38 CFS

DA-2
TC = 7.0 MIN.
C = 0.84
$I_{50\text{-}yr}$ = 2.73 IN/HR
Q = 2.21 CFS

DA-3
TC = 5.0 MIN.
C = 0.84
$I_{50\text{-}yr}$ = 3.19 IN/HR
Q = 1.10 CFS

Q_{TOTAL} = 7.69 CFS

No.	REVISIONS	DATE	BY

Kimley»Horn

660 S. FIGUEROA STREET, SUITE 2050, LOS ANGELES, CA 90017
PHONE: 213-261-4040
WWW.KIMLEY-HORN.COM

PRELIMINARY
FOR REVIEW ONLY
NOT FOR
CONSTRUCTION
Kimley»Horn
Kimley-Horn and Associates, Inc.

KHA PROJECT
099771001
DATE
09/10/2020
SCALE AS SHOWN
DESIGNED BY JS
DRAWN BY JS
CHECKED BY RC

EXISTING DRAINAGE AREA MAP

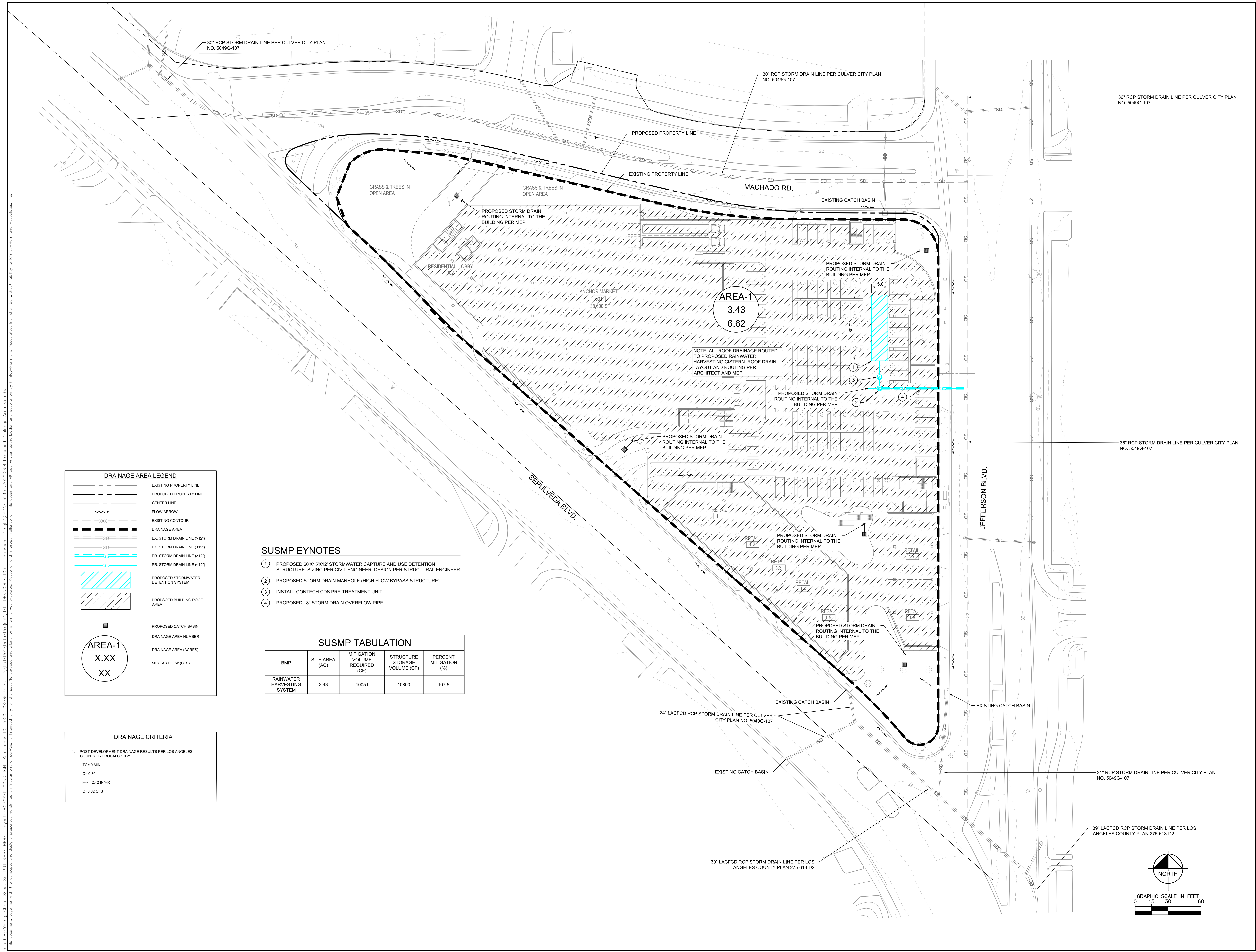
11111 JEFFERSON BOULEVARD

PREPARED FOR
3MR CAPITAL, LLC
CULVER CITY CA

SHEET NUMBER
EX-1

Plotted By: Young, Chris Sheet Set: PUT NAME HERE Layout: 3 EXISTING CONDITIONS September 10, 2020 08:00:28am \\LDTFP01\Data\Project\LDT_LDEV\099771001- Jefferson Triangle\CAD\Exhibits\20191120 Existing Drainage Area Map.dwg

EXHIBIT 2 - Proposed Drainage Area Map & SUSMP Exhibit



DRAINAGE AREA LEGEND

——— — — ———	EXISTING PROPERTY LINE
——— — — ———	PROPOSED PROPERTY LINE
——— — ———	CENTER LINE
〰►	FLOW ARROW
— — XXX — —	EXISTING CONTOUR
■ ■ ■ ■	DRAINAGE AREA
═ SD ═	EX. STORM DRAIN LINE (>12")
— SD —	EX. STORM DRAIN LINE (<12")
═ SD ═	PR. STORM DRAIN LINE (>12")
— SD —	PR. STORM DRAIN LINE (<12")
▨	PROPOSED STORMWATER DETENTION SYSTEM
▨	PROPSOED BUILDING ROOF AREA
▣	PROPOSED CATCH BASIN
AREA-1	DRAINAGE AREA NUMBER
X.XX	DRAINAGE AREA (ACRES)
XX	50 YEAR FLOW (CFS)

SUSMP EYNOTES

① PROPOSED 60'X15'X12' STORMWATER CAPTURE AND USE DETENTION STRUCTURE. SIZING PER CIVIL ENGINEER. DESIGN PER STRUCTURAL ENGINEER

② PROPOSED STORM DRAIN MANHOLE (HIGH FLOW BYPASS STRUCTURE)

③ INSTALL CONTECH CDS PRE-TREATMENT UNIT

④ PROPOSED 18" STORM DRAIN OVERFLOW PIPE

SUSMP TABULATION

BMP	SITE AREA (AC)	MITIGATION VOLUME REQUIRED (CF)	STRUCTURE STORAGE VOLUME (CF)	PERCENT MITIGATION (%)
RAINWATER HARVESTING SYSTEM	3.43	10051	10800	107.5

DRAINAGE CRITERIA

1. POST-DEVELOPMENT DRAINAGE RESULTS PER LOS ANGELES COUNTY HYDROCALC 1.0.2:

   TC= 9 MIN

   C= 0.80

   $I_{50\text{-}yr}$= 2.42 IN/HR

   Q=6.62 CFS

No.	REVISIONS	DATE	BY

Kimley»Horn

660 S. FIGUEROA STREET, SUITE 2050, LOS ANGELES, CA 90017
PHONE: 213-261-4040
WWW.KIMLEY-HORN.COM

PRELIMINARY
FOR REVIEW ONLY
NOT FOR CONSTRUCTION
Kimley»Horn
Kimley-Horn and Associates, Inc.

KHA PROJECT 099771001
DATE 09/04/2020
SCALE AS SHOWN
DESIGNED BY CY
DRAWN BY JS
CHECKED BY RC

PROPOSED DRAINAGE AREA MAP & SUSMP EXHIBIT

11111 JEFFERSON BOULEVARD

PREPARED FOR
3MR CAPITAL, LLC

CULVER CITY CA

SHEET NUMBER
EX-2

APPENDIX A - HydroCalc Calculations – 50-yr Storm

Peak Flow Hydrologic Analysis

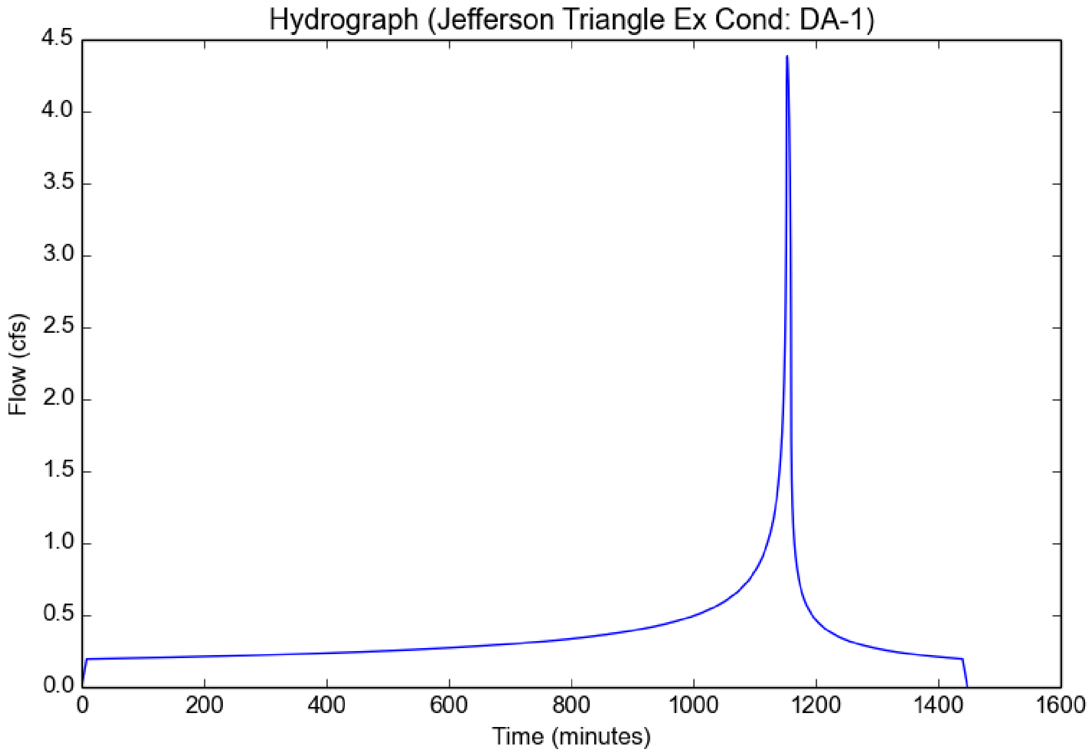
File location: K:/LDT_LDEV/194145001-Jefferson Triangle/Reports/H&H/Calculations/Jefferson Triangle - Existing Conditions - DA-1.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Jefferson Triangle Ex Cond
Subarea ID	DA-1
Area (ac)	2.05
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	5.35
Percent Impervious	0.87
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.35
Peak Intensity (in/hr)	2.5593
Undeveloped Runoff Coefficient (Cu)	0.4038
Developed Runoff Coefficient (Cd)	0.8355
Time of Concentration (min)	8.0
Clear Peak Flow Rate (cfs)	4.3835
Burned Peak Flow Rate (cfs)	4.3835
24-Hr Clear Runoff Volume (ac-ft)	0.7238
24-Hr Clear Runoff Volume (cu-ft)	31528.9568



Peak Flow Hydrologic Analysis

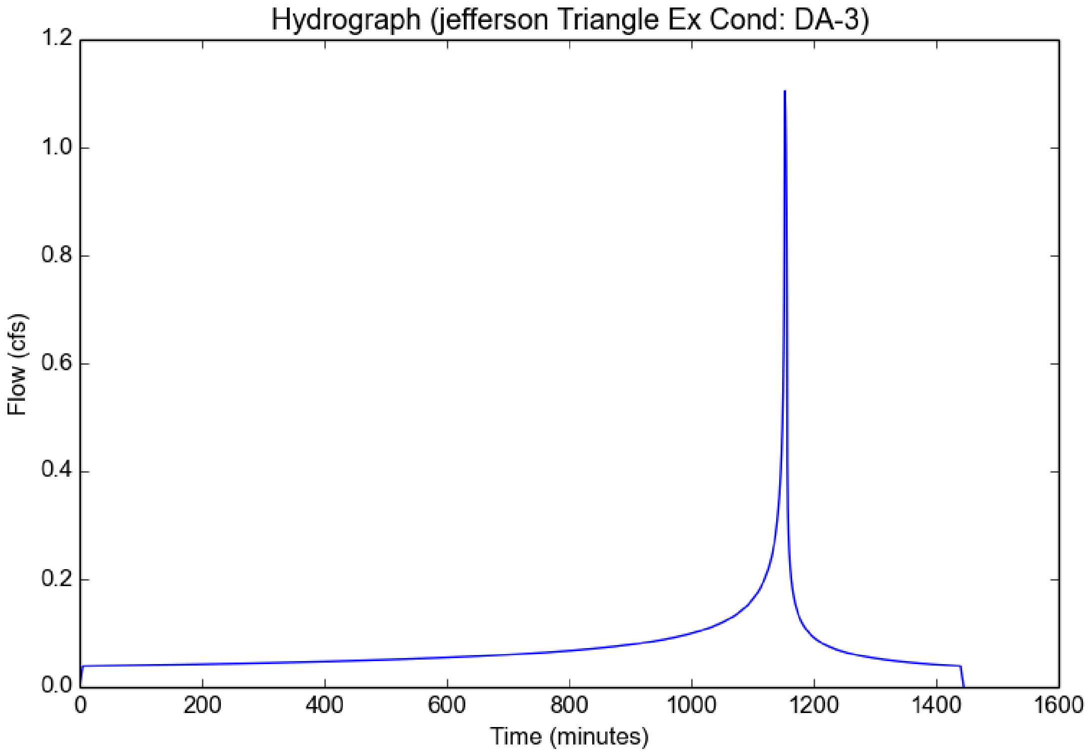
File location: K:/LDT_LDEV/194145001-Jefferson Triangle/Reports/H&H/Calculations/Jefferson Triangle - Existing Conditions - DA-3.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	jefferson Triangle Ex Cond
Subarea ID	DA-3
Area (ac)	0.41
Flow Path Length (ft)	200.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	5.35
Percent Impervious	0.87
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.35
Peak Intensity (in/hr)	3.192
Undeveloped Runoff Coefficient (Cu)	0.4703
Developed Runoff Coefficient (Cd)	0.8441
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	1.1047
Burned Peak Flow Rate (cfs)	1.1047
24-Hr Clear Runoff Volume (ac-ft)	0.1448
24-Hr Clear Runoff Volume (cu-ft)	6307.3961



Peak Flow Hydrologic Analysis

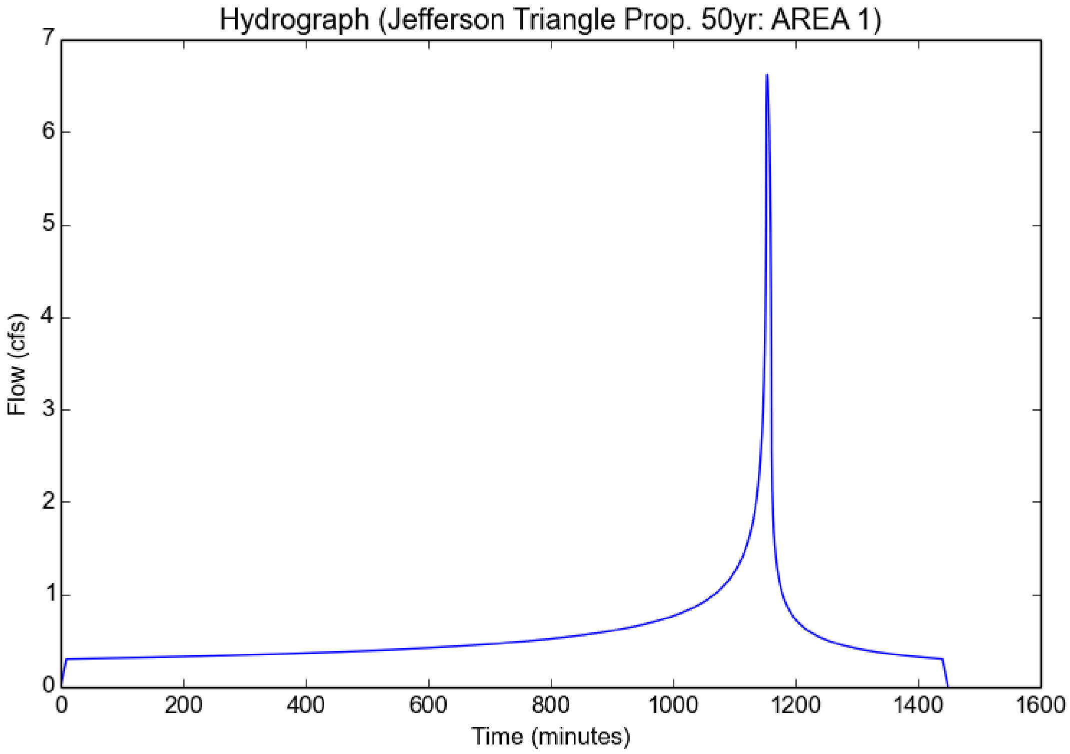
File location: //LDTFP01/Data/Project/LDT_LDEV/099771001- Jefferson Triangle/Reports/EIR Support Memos/H&H Memo/Calculations/Jefferson Triang
Version: HydroCalc 1.0.2

Input Parameters

Project Name	Jefferson Triangle Prop. 50yr
Subarea ID	AREA 1
Area (ac)	3.43
Flow Path Length (ft)	600.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	5.35
Percent Impervious	0.8
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.35
Peak Intensity (in/hr)	2.4215
Undeveloped Runoff Coefficient (Cu)	0.3857
Developed Runoff Coefficient (Cd)	0.7971
Time of Concentration (min)	9.0
Clear Peak Flow Rate (cfs)	6.6207
Burned Peak Flow Rate (cfs)	6.6207
24-Hr Clear Runoff Volume (ac-ft)	1.128
24-Hr Clear Runoff Volume (cu-ft)	49136.6898



APPENDIX B - SUSMP Calculations

Peak Flow Hydrologic Analysis

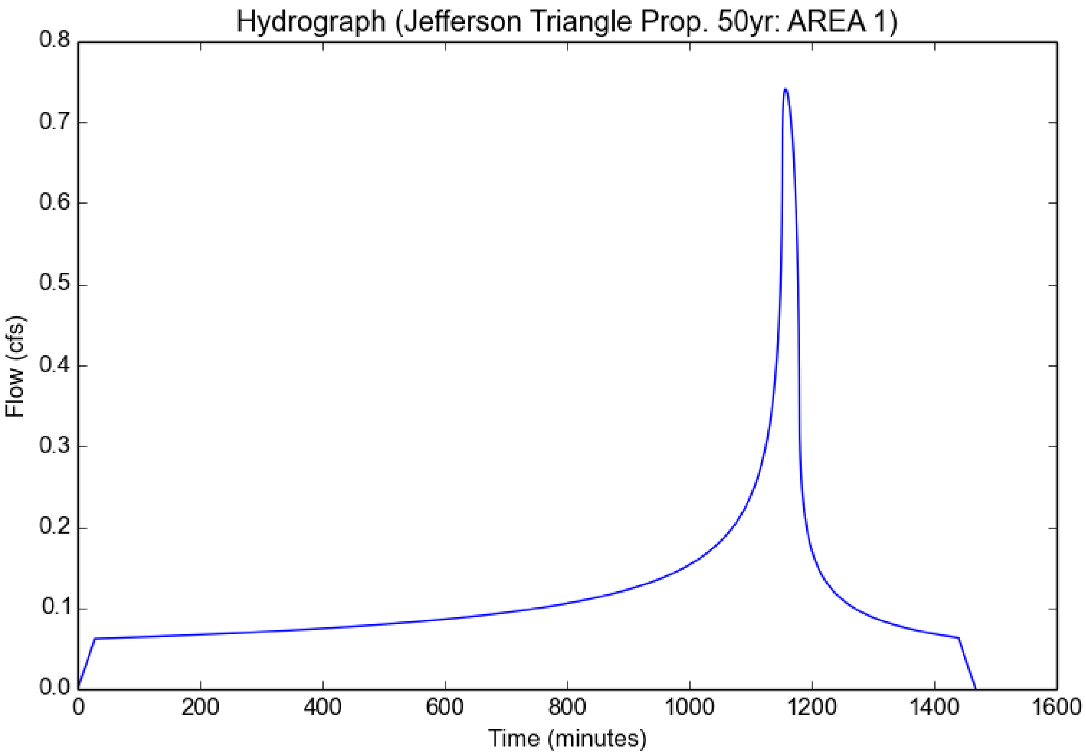
File location: //LDTFP01/Data/Project/LDT_LDEV/099771001- Jefferson Triangle/Reports/EIR Support Memos/H&H Memo/Calculations/Jefferson Triang
Version: HydroCalc 1.0.2

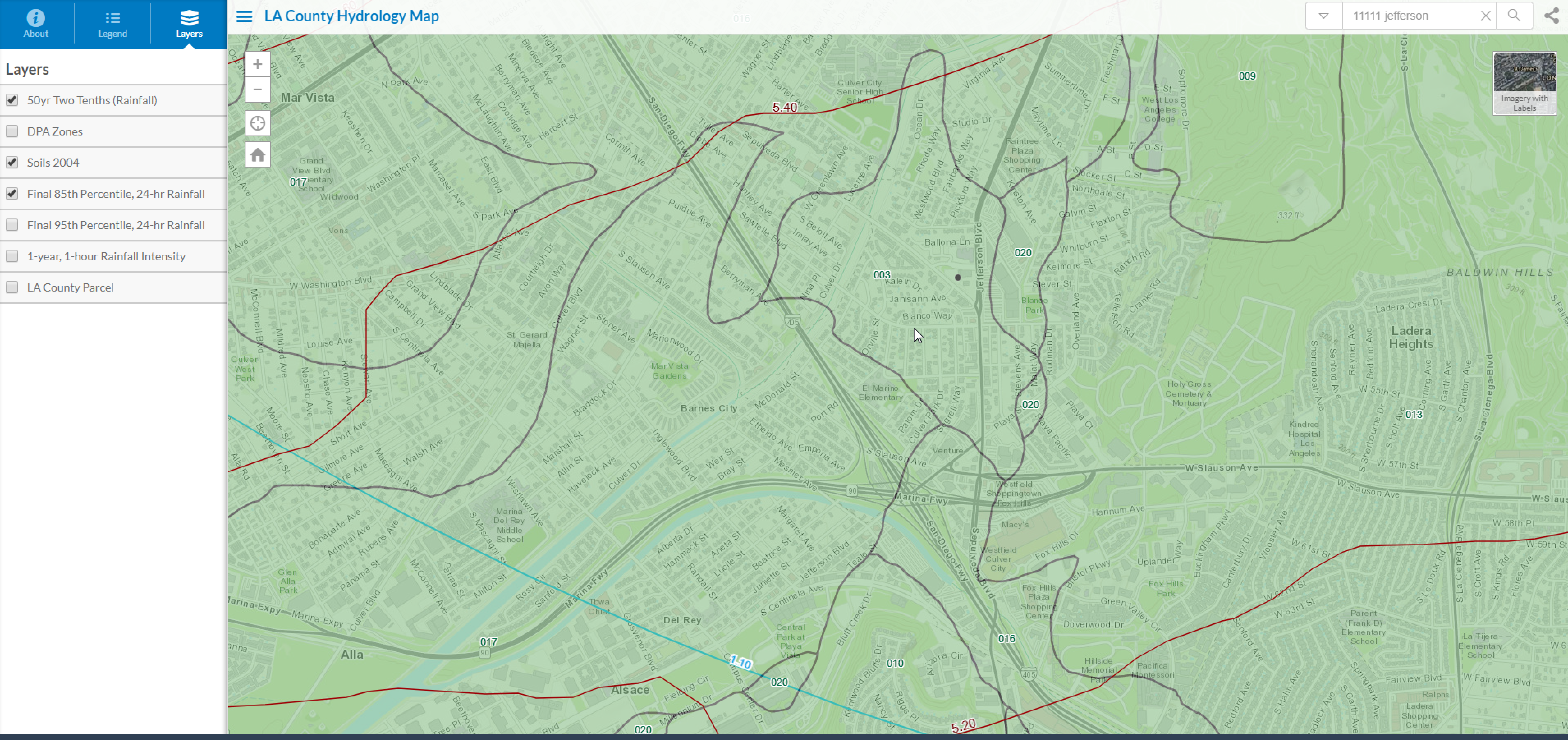
Input Parameters

Project Name	Jefferson Triangle Prop. 50yr
Subarea ID	AREA 1
Area (ac)	3.43
Flow Path Length (ft)	600.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.1
Percent Impervious	0.8
Soil Type	3
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.1
Peak Intensity (in/hr)	0.292
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.74
Time of Concentration (min)	28.0
Clear Peak Flow Rate (cfs)	0.7413
Burned Peak Flow Rate (cfs)	0.7413
24-Hr Clear Runoff Volume (ac-ft)	0.2307
24-Hr Clear Runoff Volume (cu-ft)	10051.3724





Layers

- ☑ 50yr Two Tenths (Rainfall)
- ☐ DPA Zones
- ☑ Soils 2004
- ☑ Final 85th Percentile, 24-hr Rainfall
- ☐ Final 95th Percentile, 24-hr Rainfall
- ☐ 1-year, 1-hour Rainfall Intensity
- ☐ LA County Parcel

APPENDIX C - BMP Calculations

Capture & Use Sizing

Note:

Red values to be changed by user.

Black values are automatically calculated.

[1]	Total Area (SF)		149846
[2]	Impervious Area (SF)		119646
[3]	Pervious Area (SF)	**[1]-[2] =**	29969
[4]	Catchment Area (SF)	**([2]*0.9)+([3]*0.1) =**	110678
[5]	Design Rainfall Depth (in)	**Greater of 0.75", 85th percentile**	1.10
[6]	V_{design} (gal)	**[5]/12*7.48*[4] =**	**61674**
[7]	Planting Area (SF)		29969
[8]	Plant Factor*		0.2
[9]	$ETWU_{(7\text{-month})}$	**21.7*0.62*[8]*[7] =**	80641
[10]	Is $ETWU_{(7\text{-month})} \geq V_{design}$?		**YES**

*The plant factor used shall be from WUCOLS. The plant factor ranges from 0 to 0.3 for low water use plants, from 0.4 to 0.6 for moderate water use plants, and from 0.7 to 1.0 for high water use plants.

Source: Low Impact Standards Development Manual, County of Los Angeles, Public Works, (Feb 2014)

Rectangular Detention Structure Sizing

[10]	V_{design} (CF)	**0.133681*[5] =**	**10081**
[11]	Number of Detention Structures		1
[12]	Length of Detention Structure (FT)		15
[13]	Width of Detention Structure (FT)		55
[14]	Depth of Detention Structure (FT)		12
[15]	$V_{provided}$ (CF)	**[11]*[12]*[13]*[14] =**	**10800**
[16]	Is $V_{provided} \geq V_{design}$?		**YES**
[17]	Percent Full		93.3%

APPENDIX D - Soils Information

GeoDesign INC

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Mixed-Use Development
11111 Jefferson Boulevard
Culver City, California

For
Buck Development LLC
April 26, 2019

GeoDesign Project: 3MRCap-1-01

GEODESIGN INC

April 26, 2019

Buck Development LLC
1749 Axenty Way
Redondo Beach, CA 90278

Attention: Kyle Faulkner

Report of Geotechnical Engineering Services
Proposed Mixed-Use Development
11111 Jefferson Boulevard
Culver City, California
GeoDesign Project: 3MRCap-1-01

GeoDesign, Inc. is pleased to submit our geotechnical engineering report for the proposed mixed-use development at 11111 Jefferson Boulevard in Culver City, California. The approximately 3.5-acre, triangular property includes four parcels encompassed by Sepulveda Boulevard, Jefferson Boulevard, and Machado Road. Our services for this project were conducted in accordance with our proposal dated March 11, 2019.

We appreciate the opportunity to be of service to you. Please contact us if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Shawn M. Dimke, P.E.
Principal Engineer

Brett A. Shipton, P.E., G.E. (Oregon)
Principal Engineer

cc: Dominic Adducci, Buck Development LLC (via email only)

SMD:kt
Attachments
One copy submitted (via email only)
Document ID: 3MRCap-1-01-042619-geor.docx

EXECUTIVE SUMMARY

The primary geotechnical considerations for the project are summarized as follows:

- The mixed-use building with the assumed loads can be supported on spread footings or on spread footings on top of rammed aggregate piers. We observed an approximately 5- to 7-foot-thick layer of high plasticity clay at depths between 21.5 and 40.5 feet in our borings. The high plasticity clay is soft to medium stiff in boring B-1. We also observed an approximately 4-foot-thick zone of loose, silty sand from 29.0 to 33.0 feet BGS in boring B-4. Soft, loose, or high plasticity soil will require over-excavation if encountered at shallow foundation subgrades bearing on native soil. Rammed aggregate piers may be preferred, particularly if the building will include one or two levels of below-grade parking, to increase allowable bearing pressure; reduce required footing sizes, and reduce the potential to over-excavate soft, loose, or high plasticity soil for shallow foundation subgrades.

- Based on subsurface conditions, laboratory testing, and our analysis, we estimate up to 1 inch of liquefaction-induced settlement is possible at the existing ground surface in sand zones when using the HHGWL for our analysis. Liquefaction potential reduces to negligible amounts for the considerably lower groundwater level at 38 to 43 feet BGS observed at the time of our explorations.

TABLE OF CONTENTS **PAGE NO.**

TABLE OF CONTENTS **PAGE NO.**

ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
AOS	apparent opening size
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CBC	California Building Code
CGS	California Geological Survey
CPT	cone penetrometer test
g	gravitational acceleration (32.2 feet/second^2)
HHGWL	historical high groundwater level
H:V	horizontal to vertical
ksf	kips per square foot
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
pci	pounds per cubic inch
PGA_M	maximum considered earthquake geometric mean peak ground acceleration adjusted for site affects
psf	pounds per square foot
psi	pounds per square inch
SPT	standard penetration test
TI	traffic index
USGS	U.S. Geological Survey

1.0 INTRODUCTION

This report presents the results of GeoDesign's geotechnical engineering evaluation for the proposed mixed-use development at 11111 Jefferson Boulevard in Culver City, California. The approximately 3.5-acre, triangular property includes four parcels encompassed by Sepulveda Boulevard, Jefferson Boulevard, and Machado Road. There are currently three buildings and associated parking and landscaping areas at the site.

The site is shown relative to surrounding features on Figure 1. A site plan showing the locations of our explorations and approximate boundaries of the site is presented on Figure 2. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

We understand current plans are for a five-story building with parking areas on the first three floors and no basement levels. However, plans could change to a five- or six-story building with one or two levels of below-grade parking. The building will occupy most of the site. Foundation loads were unknown at the time of this report; however, we anticipate maximum column and wall loads may range up to 500 kips and 15 kips per lineal foot, respectively. We have assumed maximum floor slab loads of 150 psf with cuts and fills of less than a few feet each, except cuts required for below-grade parking.

3.0 PURPOSE AND SCOPE

The purpose of our geotechnical exploration was to provide an understanding of the subsurface conditions and develop engineering recommendations for use in design and construction of the proposed development. Our specific scope of services is summarized as follows:

- Contacted the one call utility notification center and subcontracted a private subcontractor to locate subsurface utilities before beginning our subsurface exploration program.
- Explored subsurface soil and groundwater conditions for the proposed development by conducting the following explorations:
  - Four borings to depths between 51.3 and 71.5 feet BGS.
  - Three CPT probes to practical refusal at depths between 29.9 and 53.6 feet BGS.
  - A pore-pressure dissipation test was conducted in the deepest CPT probe.
- Maintained a detailed log of each boring and classified the material encountered in the borings in general accordance with ASTM D2488.
- Provided measured and estimated groundwater elevations.
- Conducted a laboratory testing program consisting of the following:
  - Thirty-seven moisture content determinations in general accordance with ASTM D2216
  - Six dry density determinations in general accordance with ASTM D2937
  - Six particle-size analyses in general accordance with ASTM D1140
  - Two Atterberg limits tests in general accordance with ASTM D4318
- Provided recommendations for temporary shoring to support excavation, if necessary, for below-grade parking.

- Provided recommendations for site preparation, including grading, temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, and subgrade preparation.
- Provided recommendations for wet weather construction.
- Provided foundation support recommendations for the proposed structure, including preferred foundation type, allowable bearing capacity, estimated settlement, and lateral resistance parameters.
- Provided recommendations for use in design of conventional retaining walls, including backfill and drainage requirements and lateral earth pressures.
- Evaluated groundwater conditions at the site and provided general recommendations for dewatering during construction and subsurface drainage (if required).
- Provided recommendations for construction of AC pavement for parking and driveway areas, including subbase, base course, and AC paving thickness.
- Provided seismic design recommendations based on the 2016 CBC.
- Evaluated the potential for liquefaction-induced settlement.
- Prepared this geotechnical engineering report summarizing the results of our geotechnical evaluation.

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The approximately 3.5-acre, triangular site is bordered by Machado Road on the north, Jefferson Boulevard on the east, and Sepulveda Boulevard on the southwest in Culver City, California. The site is currently occupied by three single-story commercial buildings and associated parking and landscaping areas. The southern building is an oil-change facility, the middle building is a restaurant, and the northern building is the Culver City Post Office. The site is relatively flat with elevations ranging from approximately 33 to 35 feet based on Google Earth.

4.2 SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by drilling four borings (B-1 through B-4) to depths between 51.3 and 71.5 feet BGS and advancing three CPT probes (CPT-1 through CPT-3) to practical refusal at depths between 29.9 and 52.6 feet BGS. The locations of the explorations are shown on Figure 2. Details of our field exploration and laboratory testing programs, the boring logs, and the results of laboratory testing are presented in Appendix A. The CPT probe logs are presented in Appendix B.

In general, subsurface conditions consist of 12.0 to 16.0 feet of stiff clay with variable sand content underlain by alternating layers and/or lenses of medium dense to very dense sand with variable fines content and medium stiff to very stiff clay with variable sand content. The sand generally becomes dense to very dense and there are fewer and thinner clay layers/lenses with increasing depth. Soft to medium stiff, high plasticity clay was also encountered from 22.0 to 29.0 feet BGS in boring B-1 and loose, silty sand was encountered from 29.0 to 33.0 feet BGS in boring B-4. We encountered pavement sections consisting of 2.5 to 5.5 inches of AC with no underlaying base rock to base rock sections of up to 4 inches thick.

4.2.1 Clay

The clay with variable sand content encountered at the site is generally stiff to very stiff, except for a soft to medium stiff zone from 22.0 to 29.0 feet BGS in boring B-1. Most of the clay encountered was observed to exhibit low plasticity except for a 5- to 7-foot-thick layer encountered at depths between 21.5 and 40.5 feet which exhibits high plasticity with tested plasticity indices of 38 to 41. The tested moisture contents of all the clay ranged from 12 to 47 percent at the time of our explorations. The tested moisture contents of the clay encountered in the upper 10 feet ranged from 12 to 23 percent at the time of our explorations.

4.2.2 Sand

The sand with variable silt and clay content encountered at the site is generally medium dense to very dense, except for a zone of loose, silty sand from 29.0 to 33.0 feet BGS in boring B-4. Variable amounts of gravel were also encountered in some of the sand. The sand generally becomes dense to very dense and is more contiguous with increasing depth. The tested moisture content of the sand ranged from 3 to 33 percent at the time of our explorations.

4.2.3 Groundwater

We observed groundwater at depths of 38.0 to 43.0 feet in the borings during our drilling. An increase in the measured pore pressure in CPT-3 at 40.0 to 42.0 feet BGS also suggests groundwater is present at this depth. Groundwater has been measured at elevations of -5.8 feet up to 10.3 feet (depths of 23.7 to 39.8 feet BGS) from years 2000 through 2015 in a groundwater well (Well I.D. 1290P) approximately ½ mile west of the site. Groundwater levels at the site are expected to fluctuate similar to those measured at the nearby well. Perched water may also be present at higher depths or where more permeable soil is underlain by less permeable soil.

Based on our review of the groundwater map presented in the *Seismic Hazard Zone Report for the Venice 7.5-Minutes Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 036 (Plate 1.2)* (CDMG, 1998), the HHGWL at the site is approximately 9 feet BGS, corresponding to an approximate elevation of 25 feet. The HHGWL presented in the State's Seismic Hazard Zone reports is intended for use in liquefaction analysis and not necessarily for other design purposes or considerations.

5.0 SITE DEVELOPMENT RECOMMENDATIONS

The following sections present general recommendations based on evaluation of results from geotechnical evaluations at the site and our understanding of the proposed site development alternatives.

5.1 SITE PREPARATION

5.1.1 Demolition

Demolition should include removal of existing pavements, concrete curbs, abandoned utilities, foundations, and any subsurface elements from the existing and any previous on-site structures. Demolition material should be transported off site for disposal or recycled and used on site if the material is acceptable for structural fill. Excavations from removing buried foundations, utilities, and other subsurface elements should be backfilled with structural fill. The sides and bottom of excavations should be cut into firm material and sloped at an inclination no steeper than 1½H:1V prior to installing structural fill.

Utility lines to be abandoned and left in place should be grouted full to reduce the potential for differential settlement resulting from collapsed pipes or erosion. The existing backfill for abandoned utility lines should be replaced with structural fill in building and pavement areas, unless inspection records or testing show that it has been compacted in accordance with the recommendations in this report.

5.1.2 Stripping and Grubbing

The existing root zone, where present, should be stripped and removed from all fill areas. The actual stripping depth should be based on field observations at the time the site is stripped. Stripped material should be transported off site for disposal or used in landscaped areas.

Trees and shrubs should be removed before mass grading begins on the site. The root balls of trees and shrubs should be grubbed out to the depth of the roots, which could exceed 3 feet. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade can occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

5.2 SUBGRADE EVALUATION

Prior to placing fill or base rock for slab-on-grade floors and pavement, the subgrade should be proof rolled with a fully loaded dump truck or similar heavy, rubber tire construction equipment to identify any soft, loose, or unsuitable areas. Proof rolling should be observed by a qualified geotechnical engineer or geotechnical field technician who should evaluate the suitability of the subgrade and identify any areas of yielding, which are indicative of soft or loose soil. If soft or loose zones are identified during proof rolling, these areas should be excavated to the extent indicated by the engineer and replaced with structural fill.

5.3 SUBGRADE PROTECTION

The fine-grained soil at the site is easily disturbed during the wet season or when the moisture content of the soil is more than a few percentage points above optimum. If not carefully executed, site preparation and utility trench work can create extensive soft areas and significant subgrade repair costs. If construction is planned when the surficial soil is wet or may become wet, appropriate construction methods should be selected to protect the subgrade and reduce the need for over-excavation of disturbed and softened soil.

When the soil is wet of optimum, the contractor should protect the surface from construction traffic using thickened granular working mats. Generally, at least 12 inches of imported granular material is required for light staging areas, but this thickness is not adequate to support heavy equipment or truck traffic. The granular mat for haul roads and areas with repeated heavy construction traffic typically should be between 18 and 24 inches thick. These sections are intended to be guidelines, and the actual thickness of haul roads and staging areas should be based on the contractor's approach to site development and the frequency and type of construction traffic. The contractor should be responsible for selecting the location and granular thickness of haul roads and staging areas.

The imported granular material for haul roads and staging areas should be placed in one lift over the prepared, undisturbed subgrade and compacted using a smooth-drum roller without the use of vibratory action. The granular material should meet the specifications for imported granular material in the "Structural Fill" section. In addition, a geotextile fabric can be placed as a barrier between the subgrade and granular material in areas of repeated construction traffic. The geotextile should have a minimum Mullen burst strength of 250 psi and an AOS between U.S. Standard No. 70 and No. 100 sieves.

5.4 TEMPORARY SLOPES

Excavation side slopes less than 15 feet high should be no steeper than 1½H:1V, provided groundwater seepage does not occur. If slopes greater than 10 feet high are required, GeoDesign should be contacted to make additional recommendations. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of the temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If sloughing or instability is observed, the slope should be flattened or supported by shoring.

5.5 SHORING

5.5.1 General

Cantilever and conventional soldier pile shoring with tieback anchors may be required for the project. Tieback anchors should be considered where excavation is near settlement-sensitive objects.

5.5.2 Cantilever Shoring

Soldier pile shoring can be designed using the values presented on Figure 3. These values do not include surcharged-induced earth pressures. Figure 4 should be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways.

5.5.3 Anchored Shoring

Anchored soldier pile shoring can be designed using the values presented on Figure 3. These values do not include surcharged-induced earth pressures. Figure 4 should be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways.

Structural design of the soldier piles should consider the lateral earth pressures discussed above. In addition to lateral earth pressures, the soldier piles will be subject to compressive forces as a result of the downward component of the tieback anchor loads. We recommend the tips of soldier piles are embedded at least 10 feet below the base of the excavation. An allowable bearing pressure of 4 ksf may be used for the base of the soldier piles resting on firm soil. Skin friction along the sides of the solider piles will also be able to resist downward forces. An allowable skin friction of between 1 and 2 ksf may be used for subsurface soil at the site.

The bonded zone for tieback anchors should be maintained outside of the "no load zone" show on Figure 3. We anticipate that the tieback anchors can achieve allowable bond strength of between 1 and 2 ksf in the subsurface soil at the site. A variety of methods are available for construction of tieback anchors. Therefore, we recommend that the contractor be responsible for selecting the appropriate bonded length and installation methods to achieve the required anchor capacity. Tieback anchors should be locked off at 100 percent of the design load.

Prior to installing production anchors, we recommend that verification testing be conducted on a minimum of two anchors. The purpose of this testing is to verify the installation procedure selected by the contractor before a large number of anchors are installed. We recommend that proof testing be conducted on all production anchors. Performance and proof testing should be performed in accordance with the guidelines provided in *Recommendations for Prestressed Rock and Soil Anchors* (Post Tensioning Institute, 2014).

We anticipate that wood lagging will be used for the shoring. To maintain the integrity of the excavation, prompt and careful installation of lagging, particularly in areas of seepage and loose soil, is recommended. All voids behind the lagging should be completely backfilled with grout slurry.

5.6 EXCAVATION

Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage does not occur in the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1H:1V or flatter and groundwater seepage does not occur. Excavations should be flattened to 1.5H:1V or flatter if excessive sloughing occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation. All excavations should be made in accordance with applicable OSHA and state regulations.

5.7 GROUNDWATER

5.7.1 Construction Considerations

Depending on the groundwater levels during construction and if plans change to include one or two levels of below-grade parking, groundwater could be encountered and provisions to lower the groundwater level temporarily during construction could be required. Dewatering systems are best designed by the contractor. It may be possible to use a sump located within excavations to dewater isolated zones of perched water or shallow limited excavations below the water table. Flow rates for dewatering are likely to vary depending on location, soil type, and the season during which the excavation occurs. Dewatering systems should be capable of adapting to variable flows.

5.7.2 Permanent Design Considerations

Based on our review of the HHGWL data, we recommend a design groundwater level of elevation 25 feet. Provisions to provide relief for potential hydrostatic pressure should be implemented for structures extending below this elevation or structures should be designed to resist hydrostatic pressures up to this elevation.

5.8 STRUCTURAL FILL

Structural fill should be free of organic matter and other deleterious material and, in general, should consist of particles no larger than 3 inches in diameter. Existing concrete debris or remnant concrete structural elements, AC pavement, and base rock can be used as structural fill, provided it is adequately processed as described below for recycled concrete or broken into particles no greater than 3 inches in greatest dimension and can be incorporated into well-graded structural fill and adequately compacted.

5.8.1 On-Site Soil

The near-surface fine-grained soil will be difficult to moisture condition and likely will not be practical for use as structural fill. The on-site soil can be used as structural fill provided it does not exhibit high plasticity, does not contain deleterious material, and can be moisture conditioned to within a few percentage points of optimum.

On-site material, to be used as structural fill, should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Compaction of all fills should be tested at least every 2 feet vertically by a minimum of two tests and at least one test per 5,000 square feet.

5.8.2 Imported Granular Material

Imported granular material should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is fairly well graded between coarse and fine and has less than 12 percent by dry weight passing the U.S. Standard No. 200 sieve. The percentage of fines may need to be decreased depending on weather conditions during construction. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Compaction of all fills should be tested at least every 2 feet vertically by a minimum of two tests and at least one test per 5,000 square feet.

5.8.3 Recycled Concrete

Recycled concrete can be used for structural fill, provided the concrete is broken to a maximum particle size of 3 inches. Recycled concrete should be processed so that it is fairly well graded between coarse and fine particle sizes. This material can be used as trench backfill and pavement base rock if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Compaction of all fills should be tested at least every 2 feet vertically by a minimum of two tests and at least one test per 5,000 square feet.

5.8.4 Utility Trenches

Pipe bedding shall consist of imported or free-draining material with a sand equivalent of not less than 30 or as specified by the pipe manufacturer or local agency. Pipe bedding is defined as the material surrounding the pipe and extending to a minimum of 12 inches above the pipe.

Trench backfill above the pipe zone should consist of durable, well-graded, granular material containing no organic or other deleterious material, should have a maximum particle size of ¾ inch, and should have less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve.

Pipe bedding should be placed in maximum 12-inch-thick lifts and compacted to not less than 90 percent of the maximum dry density, as determined by ASTM D1557, or as recommended by the pipe manufacturer. Backfill above the pipe bedding should be placed in maximum 12-inch-thick lifts and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D1557. Trench backfill located within 2 feet of finish subgrade elevation should be placed in maximum 12-inch-thick lifts and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

Utility trench backfill compaction should be tested every 2 feet vertically and every 100 feet horizontally with a minimum of two horizontal tests per pipe run.

5.9 SITE DRAINAGE

During grading at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During grading and excavation on site, the contractor should keep all footing excavations and floor slab subgrades free of water.

5.10 EROSION CONTROL

The on-site soil is moderately susceptible to erosion. We recommend that slopes be covered with an appropriate erosion control product if construction occurs during periods of wet weather. We recommend that all slope surfaces be planted as soon as practical to minimize erosion. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with state and local ordinances.

6.0 FOUNDATION SUPPORT RECOMMENDATIONS

6.1 GENERAL

Based on the anticipated loads, it is our opinion the mixed-use building can be supported on spread footings or on spread footings on top of rammed aggregate piers. We observed an approximately 5- to 7-foot-thick layer of high plasticity clay at depths between 21.5 and 40.5 feet BGS in our borings. The high plasticity clay is soft to medium stiff in boring B-1. We also observed an approximately 4-foot-thick zone of loose, silty sand between 29.0 and 33.0 feet BGS in boring B-4. Soft, loose, or high plasticity soil will require over-excavation if encountered at shallow foundation subgrades bearing on native soil. Rammed aggregate piers may be preferred, particularly if the building will include one or two levels of below-grade parking, to increase allowable bearing pressure; reduce required footing sizes; and reduce the potential to over-excavate soft, loose, or high plasticity soil for shallow foundation subgrades.

6.2 SHALLOW FOUNDATIONS

Shallow foundations bearing on firm, undisturbed native soil or structural fill overlaying firm, undisturbed native soil can be used to support the proposed building loads and can be proportioned for a maximum allowable soil bearing pressure of 4,000 psf. This bearing pressure is a net bearing pressure and applies to the total of dead and long-term live loads and may be increased by one-third when considering seismic or wind loads. The weight of the footing and any overlying backfill can be ignored in calculating footing loads.

Continuous wall and isolated spread footings should be at least 18 and 24 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

We recommend all footing subgrades be evaluated by the project geotechnical engineer or their representative to confirm suitable bearing conditions. Observations should also confirm all loose or soft material, organics, undocumented fill, high plasticity soil, and softened subgrades (if present) have been removed as discussed above. Localized deepening of footing excavations and backfill with crushed rock may be required to penetrate unsuitable materials in isolated areas. Foundation-bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, the water and any disturbed subgrade should be removed before placing reinforcing steel or concrete.

We estimate total post-construction consolidation-induced settlement under static conditions should be less than 1 inch, with differential settlement of less than ½ inch between footings.

6.3 SPREAD FOOTINGS BEARING ON AGGREGATE PIERS

The building can be supported on spread footings underlain by rammed aggregate piers. Rammed aggregate pier foundation systems should consist of compacted aggregate piers that reinforce and improve the soil. These systems are proprietary and designed and constructed by a specialty contractor. Conventional spread footings are placed over the completed rammed aggregate piers. An allowable bearing pressure of 8,000 psf can typically be achieved for footings bearing on rammed aggregate piers. We estimate the allowable bearing pressure can be increased by one-half for short-term loading conditions. We anticipate that the rammed aggregate piers will extend at least 35 feet below the current ground surface. Design-build contractors should be contacted and provided with a copy of this report to confirm the recommended allowable bearing pressures, estimated settlements, and aggregate pier depths and configurations.

Sloughing or caving should be anticipated during drilling and compaction of the crushed rock in the rammed aggregate piers because of the potential for sand below the groundwater table. It may be necessary to use casing to prevent excessive sloughing and caving during installation of the rammed aggregate piers.

6.4 LATERAL RESISTANCE

Resistance to lateral loads can be developed by passive pressure on the face of footings, grade beams, tie beams, and other buried foundation elements. Assuming a minimum translation of 1.0 inch, the allowable passive resistance on the face of buried foundation elements may be computed using an equivalent fluid pressure of 350 pcf (triangular distribution) for foundation elements cast neatly against the existing soil or backfilled with structural fill. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. Coefficients of friction equal to 0.35 and 0.45 can be used for footings bearing on native soil and rammed aggregate piers, respectively.

6.5 SLABS ON GRADE

A modulus of subgrade reaction of 125 pci should be used for design of the floor slab supporting up to 150 psf areal loading, provided the subgrade is prepared in accordance with the recommendations presented in this section.

We recommend that the floor slab be supported on at least 6 inches of imported aggregate base to aid as a capillary break and to provide uniform support. The imported granular material should be placed and compacted as previously recommended for aggregate base.

Floor slab performance can also be affected by poor subgrade performance. All slab subgrades should be evaluated by appropriate personnel to confirm suitable bearing conditions. Observations should also confirm that loose or soft material, organics, unsuitable fill, high plasticity soil, and softened subgrades (if present) have been removed.

Vapor barriers beneath floor slabs are typically required by flooring manufactures to maintain the warranty on their products. In our experience, adequate performance of floor adhesives can be achieved by using a clean base rock (less than 5 percent fines) beneath the floor slab with no vapor barrier. In fact, vapor barriers can frequently cause moisture problems by trapping water beneath the floor slab that is introduced during construction. If a vapor barrier is used, water should not be applied to the base rock prior to pouring the slab and the work should be completed during extended dry weather so that rainfall is not trapped on top of the vapor barrier.

Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

6.6 EXPANSIVE SOIL

Expansive soil exhibits an appreciable volume change in response to changes in moisture content. Material that is susceptible to expansion is typically high plasticity clay. Expansive soil can potentially impact the proposed site flatwork by causing cracks in pavement and/or vertical offsets at expansion joints.

Expansive soil was not encountered in the borings within close proximity to the existing ground surface; however, an approximately 5- to 7-foot-thick layer of high plasticity clay, which could be expansive, was encountered at depths between 21.5 and 40.5 feet BGS and other discontinuous zones of high plasticity clay may also be present at the site.

High-plasticity clay, if identified within the upper few feet at the site during construction, may require removal and replacement with non-expansive soil beneath foundations, building floor slabs, and site flatwork. High-plasticity clay is not suitable for re-use in compacted fills.

7.0 PERMANENT RETAINING STRUCTURES

Permanent retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit pressure of 35 pcf. If retaining walls are restrained against rotation during backfilling, they should be designed for an at-rest earth pressure of 55 pcf. These values are based on the assumptions that (1) the retained soil is level, (2) the retained soil is drained, and (3) the wall is less than 20 feet in height. Provisions to provide relief for potential hydrostatic pressure should be implemented if structures will extend below the HHGWL elevation of approximately 25 feet or we should be contacted to provide recommendations for structures to resist hydrostatic pressures up to this elevation. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions. Lateral pressures induced by surcharge loads can be computed using the methods presented on Figure 4. Seismic lateral forces can be calculated using a dynamic force equal to $8.5H^2$ pounds per linear foot of wall for active conditions and $16H^2$ for at-rest conditions, where H is the wall height. The seismic force should be applied as a distributed load with the centroid located at 0.6H from the wall base. Footings for retaining walls should be designed as recommended for shallow foundations.

Drains consisting of a perforated drainpipe wrapped in a geotextile filter should be installed behind retaining walls. The pipe should be embedded in a zone of coarse sand or gravel containing less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve and should outlet to a suitable discharge.

8.0 SEISMIC CONSIDERATIONS

8.1 SEISMIC HAZARDS

8.1.1 Liquefaction

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking.

The site is in a liquefaction hazard zone according to the *Earthquake Zones of Required Investigation Venice Quadrangle*. Based on the soil conditions encountered at the site and the earthquake hazard mapping, we completed a liquefaction analysis at the site. We conducted our analysis using the computer program CLIQ and the data from the CPTs. Ground shaking was modeled using the ASCE 7-10 design-level crustal earthquake event with a magnitude of 7.25 and a PGA_m of 0.66 g as described in ASCE 7-10 Section 11.8.3.

Based on subsurface conditions, laboratory testing, and our analysis, we estimate up to 1 inch of liquefaction-induced settlement is possible at the existing ground surface in sand zones when using the HHGWL for our analysis. We estimate differential settlement will be less than one-half the total between adjacent footings or over distances of 50 feet. Liquefaction potential reduces to negligible amounts for the considerably lower groundwater level at 38.0 to 43.0 feet BGS observed at the time of our explorations.

8.1.2 Seismically Induced Dry Settlement

Seismically induced ground settlement can occur in the soil above the groundwater table during strong shaking (dry settlement). We completed an analysis to determine the dry settlement potential at the site. Based on analysis, significant dry settlement is not anticipated for the stiff clay soil above the HHGWL of approximately 9 feet BGS. We estimate less than 0.15 inch of dry settlement for our analysis using a lower groundwater depth of 38 feet BGS.

8.1.3 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. There are no major open faces close to the site, and the liquefaction potential at the site is low. Accordingly, the potential for lateral spreading at the site is not a design consideration for the project.

8.1.4 Fault Rupture

Faults in Southern California are considered active, potentially active, and inactive based on criteria developed by CGS for the Alquist-Priolo Earthquake Fault Zoning Program (Hart, 1999). By definition, an active fault is one that has had surface displacement within Holocene time (approximately the last 11,000 years). A potentially active fault is one that has demonstrated surface displacement of Quaternary Age deposits (last 1.6 million years). Inactive faults have not moved in the last 1.6 million years.

The primary purpose of the Alquist-Priolo Earthquake Fault Zoning Program is to identify sites that have a potential for surface rupture due to active faults that are in close proximity to the site. In such cases, a building setback zone is established to mitigate the potential for surface rupture.

The site is not located within an Alquist-Priolo Special Study zone. Based on our review of the Fault Activity Map of California (Hart, 1999), the site is not located within an active fault zone. Therefore, the potential for surface fault rupture at the site is considered to be very low.

8.1.5 Other Hazards

Due to the distance from the ocean and elevation of the site, seiche and tsunami hazards are not a consideration at the site. Based on the flat nature of the site, landslide hazards are not a design consideration at the site.

8.2 SEISMIC DESIGN PARAMETERS

Seismic design parameters were determined in accordance with Chapter 16, Section 1613 of the 2016 CBC and ASCE 7-10. A Site Class F is applicable to the site since the soil is vulnerable to potential failure or collapse under seismic loading (liquefaction). The structure at the site can be designed for a Site Class D if the subsurface conditions below foundations are improved with aggregate piers mitigating the potential for liquefaction-induced settlement. Alternately, ASCE 7-10 indicates structures with periods of vibrations less than or equal to 0.5 second can be designed for the site class determined without regard to liquefaction. Assuming the period of the structure will be less than 0.5 second or subsurface conditions are improved with aggregate piers, we recommend the following seismic parameters for Site Class D can be used for design. If the period of the structure exceeds 0.5 second and subsurface conditions will not be improved with aggregate piers, GeoDesign should be contacted to conduct a site-specific seismic response analysis. The seismic design criteria in accordance with ASCE 7-10 are summarized in Table 1.

Table 1. Seismic Design Parameters

Parameter	0.2 Second (Short Period)	1 Second (Long Period)
MCE Spectral Acceleration	S_s = 1.835 g	S_1 = 0.668 g
Site Class	(D or F)*	
Site Coefficient	F_a = 1.00	F_v = 1.50
Adjusted Spectral Acceleration	S_{MS} = 1.835 g	S_{M1} = 1.003 g
Design Spectral Response Acceleration Parameters	S_{DS} = 1.223 g	S_{D1} = 0.668 g

* Site is defined as Site Class F due to potential for liquefaction; however, Site Class D is appropriate if the subsurface will be improved with aggregate piers. Site Class D parameters can also be used for a Site Class F if structures have a fundamental period of 0.5 second or less per ASCE 7-10 Section 20.3.1

9.0 PAVEMENT DESIGN

9.1 GENERAL

The required pavement and base thicknesses will depend on the expected wheel loads and volume of traffic (TI). Recommendations for various TIs for AC pavement are presented below.

The preparation of the paving area subgrade should be completed immediately prior to placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the pavement.

For design of the various paving materials, the pavement thickness is based on our observations of the on-site soil conditions. Based on the nature of the upper soil, we assumed an R-value of 15 for use in our pavement design calculations.

9.2 AC DESIGN SECTIONS

Table 2 summarizes our AC pavement recommendations for assumed TIs of 4, 5, and 7.

Table 2. Paving Design Sections

Traffic Use	TI	AC (inches)	Base Course (inches)
Parking Areas	4	3	7
Drive Lanes	5	3	9
Delivery Access Lanes and Loading Docks	7	4.5	14

The AC pavement sections were determined using the State of California Department of Transportation design method. We can determine the recommended pavement and base course thickness for other TIs if required. Careful observation is recommended to confirm that the recommended thickness or greater is achieved and that proper construction procedures are followed.

The base course should conform to requirements of Section 26 of State of California Standard Specifications for Public Works Construction (the "Greenbook"). The base course should be compacted to at least 95 percent as determined in accordance with ASTM D1557.

10.0 OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated. In addition, sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications.

11.0 LIMITATIONS

We have prepared this report for use by Buck Development LLC and members of the design and construction teams for the proposed development. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil explorations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were not finalized at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

◆ ◆ ◆

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Shawn M. Dimke, P.E.
Principal Engineer

Brett A. Shipton, P.E., G.E. (Oregon)
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
SHAWN MICHAEL DIMKE
C68997
CIVIL
STATE OF CALIFORNIA

Signed 04/26/2019

REFERENCES

American Society of Civil Engineers (ASCE), 2010. Minimum Design Loads for Buildings and Other Structures. ASCE Standard ASCE/SEI 7-010.

American Concrete Institute, 2006. Publication 302.2R-06, Guide for Concrete Slabs That Receive Moisture Sensitive Flooring Materials.

California Building Code (CBC), 2016. 2016 California Building Code, California Code of Regulations, Title 24, Part 2, 3 volumes.

California Department of Conservation, Division of Mines and Geology (CDMG), 1998. *Seismic Hazard Zone Report for the Venice 7.5-Minute Quadrangle.*

California Department of Transportation, 2018. The "Greenbook," Standard Specifications for Public Works Construction.

California Geological Survey (CGS), 2008. Special Publication 117A, "Guidelines for Evaluating and Mitigating Seismic Hazards in California."

Hart, E. W., Bryant, W. A., California Division of Mines and Geology, 1999. *Fault-rupture hazard zones in California: Alquist-Priolo Earthquake Fault Zoning Act with index to earthquake fault zones maps*. Rev. 1997. Sacramento, Calif.: California Dept. of Conservation, Division of Mines and Geology.

Post Tensioning Institute, 2014. *Recommendations for Prestressed Rock and Soil Anchors.*

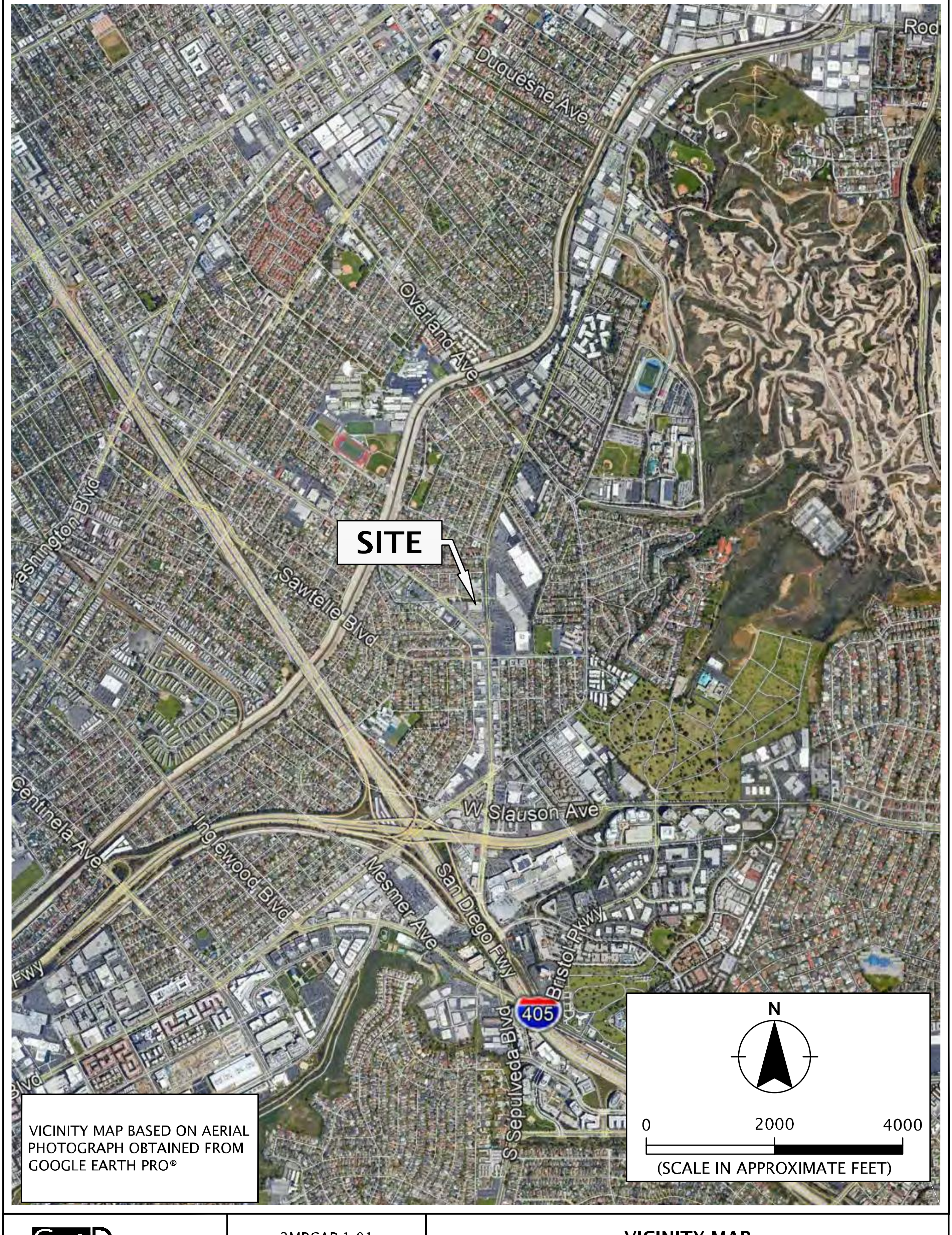
Seismic Design Values for Buildings.
http://geohazards.usgs.gov/designmaps/us/application.php

Thomas, Kate, and Thompson, Jim, 1999. Venice Quadrangle Seismic Hazard Zones, California Geological Survey: Special Publication 42.

USGS, 2017. www.usgs.gov

USGS, 2017. Earthquake Hazards Program, *US Earthquake Information by State*, U.S. Geological Survey. http://earthquake.usgs.gov/earthquakes/states/

FIGURES



VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

Printed By: aday | Print Date: 4/26/2019 8:22:14 AM
File Name: J:\0-9\3MRCap\3MRCap-1\3MRCap-1-01\Figures\CAD\3MRCap-1-01-VM01.dwg | Layout: FIGURE 1

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

VICINITY MAP

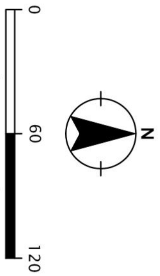
APRIL 2019

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE 1

LEGEND:

- – – SITE BOUNDARY
- B-1◐ BORING
- CPT-1△ CPT



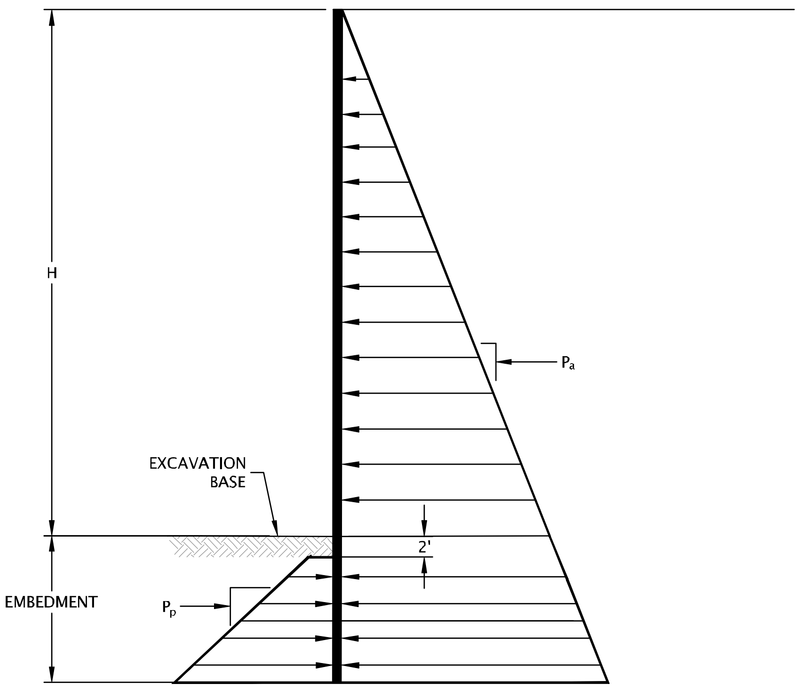
SITE PLAN BASED ON AERIAL PHOTOGRAPH
OBTAINED FROM GOOGLE EARTH PRO®,
FEBRUARY 7, 2019

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01	**SITE PLAN**	
APRIL 2019	PROPOSED MIXED-USE DEVELOPMENT CULVER CITY, CA	**FIGURE 2**

Printed By: aday | Print Date: 4/26/2019 8:22:17 AM
File Name: J:\0-9\3MRCap\3MRCap-1\3MRCap-1-01\Figures\CAD\3MRCap-1-01-det-cbwdc.dwg | Layout: FIGURE 3

RECOMMENDED DESIGN PARAMETERS FOR CANTILEVERED WALL

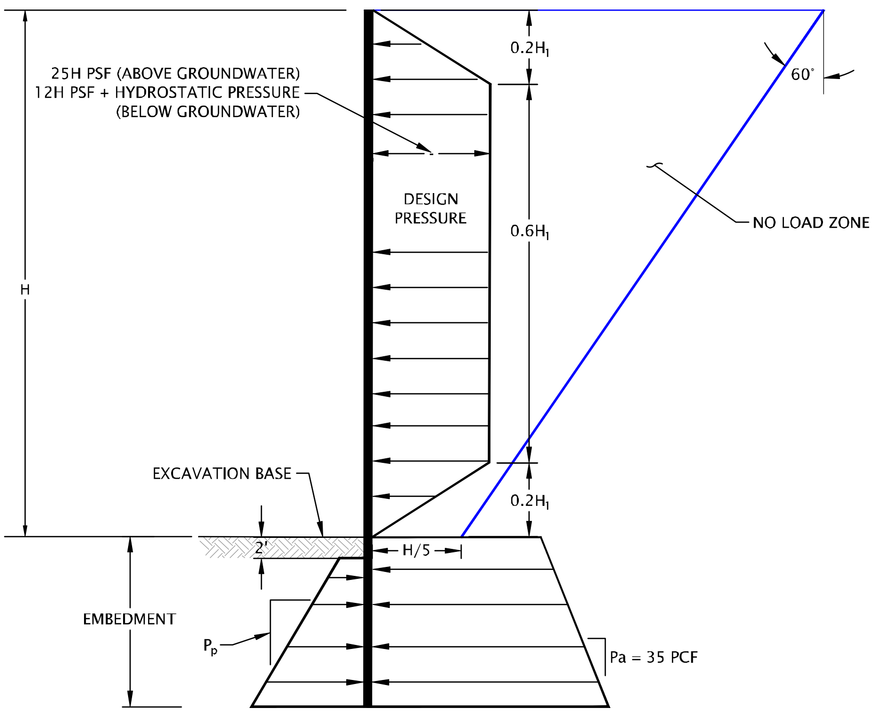


EXPLANATION:
H = HEIGHT OF SOLDIER PILE EXPOSED (IN FEET)

EQUIVALENT FLUID PRESSURES FOR DESIGN
P_a = 35 PCF (WALL FREE TO ROTATE, ABOVE GROUNDWATER)
P_a = 55 PCF (WALL NOT FREE TO ROTATE, ABOVE GROUNDWATER)
P_a = 85 PCF (WALL FREE TO ROTATE, BELOW GROUNDWATER)
P_a = 95 PCF (WALL NOT FREE TO ROTATE, BELOW GROUNDWATER)
P_p = 350 PCF (ABOVE GROUNDWATER)
P_p = 250 PCF (BELOW GROUNDWATER)

NOTES:
1. FIGURE DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. PASSIVE EARTH PRESSURES DO NOT INCLUDE A FACTOR OF SAFETY.
3. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
4. PASSIVE PRESSURE ACTS OVER 2X THE PILE WIDTH.
5. ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW THE EXCAVATION.

RECOMMENDED DESIGN PARAMETERS FOR BRACED WALLS



EXPLANATION:
H = HEIGHT OF SOLDIER PILE EXPOSED (IN FEET)

EQUIVALENT FLUID PRESSURES FOR DESIGN
P_p = 350 PCF (ABOVE GROUNDWATER)
P_p = 250 PCF (BELOW GROUNDWATER)

NOTES:
1. FIGURE DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. PASSIVE EARTH PRESSURES DO NOT INCLUDE A FACTOR OF SAFETY.
3. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
4. PASSIVE PRESSURE ACTS OVER 2X THE PILE WIDTH.
5. ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW THE EXCAVATION.
6. TIEBACKS SHOULD BE LOCKED OFF AT 100 PERCENT OF THE DESIGN LOAD.

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

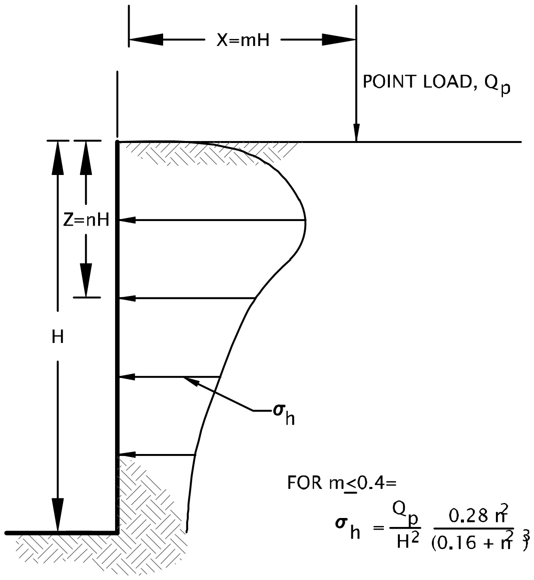
3MRCAP-1-01
APRIL 2019

CANTILEVERED AND BRACED WALLS DESIGN CRITERIA

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE 3

Printed By: aday | Print Date: 4/26/2019 8:22:17 AM
File Name: J:\0-9\3MRCap\3MRCap-1\3MRCap-1-01\Figures\CAD\3MRCap-1-01-det-silep.dwg | Layout: FIGURE 4

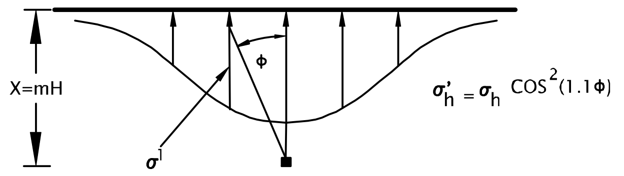


FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_p}{H^2}\,\frac{0.28\,n^2}{(0.16+n^2)^3}$$

FOR m>0.4=

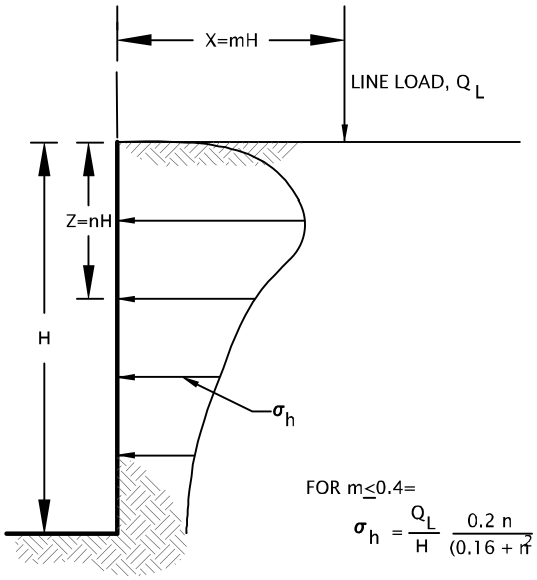
$$\sigma_h = \frac{Q_p}{H^2}\,\frac{1.77m^2\,n^2}{(m^2+n^2)^3}$$



$$\sigma'_h = \sigma_h\ \mathrm{COS}^2(1.1\phi)$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD



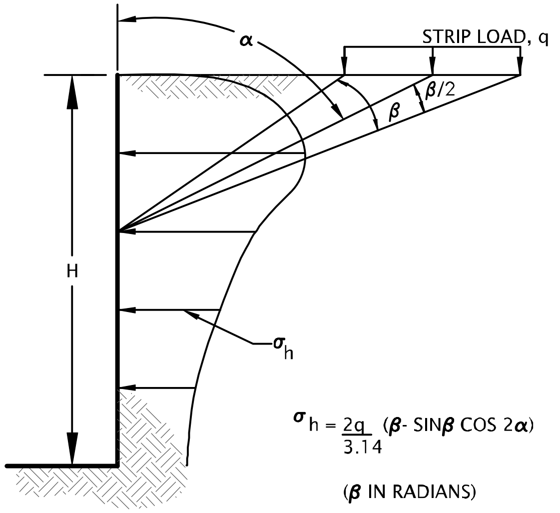
FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_L}{H}\,\frac{0.2\,n}{(0.16+n^2)^2}$$

FOR m>0.4=

$$\sigma_h = \frac{Q_L}{H}\,\frac{1.28m^2\,n}{(m^2+n^2)^2}$$

LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{3.14}\,(\beta - \mathrm{SIN}\beta\ \mathrm{COS}\ 2\alpha)$$

(β IN RADIANS)

STRIP LOAD PARALLEL TO WALL

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.
3. VALUES IN THIS FIGURE ARE UNFACTORED.

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

APRIL 2019

SURCHARGE-INDUCED LATERAL EARTH PRESSURES

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE 4

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling four borings (B-1 through B-4) using hollow-stem auger drilling techniques to depths between 51.3 and 71.5 feet BGS and advancing three CPT probes (CPT-1 through CPT-3) to depths between 29.9 and 53.6 feet BGS. Drilling services were provided on March 25 and 26, 2019 by Martini Drilling of Huntington Beach, California. The CPT probes were completed on March 25, 2019 by Kehoe Testing & Engineering of Huntington Beach, California. The boring logs are presented in this appendix. The results of the CPT probes are presented in Appendix B.

The locations of the explorations were determined in the field by pacing and taping from surveyed existing site features. This information should be considered accurate only to the degree implied by the methods used.

A member of our geotechnical staff observed the explorations. We collected representative samples of the various soils encountered in the explorations.

SOIL SAMPLING

Soil samples were collected from the borings using SPTs. The SPTs were performed in general conformance with ASTM D1586. Relatively undisturbed samples were also collected from the borings using a modified California split-spoon sampler in general accordance with ASTM D3550. The samplers were driven with a 140-pound automatic trip hammer free-falling 30 inches. The samplers were driven a total of 18 inches or to refusal. The number of blows required to drive the samplers the final 12 inches is recorded on the exploration logs, unless otherwise noted. Sampling methods and intervals are shown on the exploration logs.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

MOISTURE CONTENT

We tested the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

ATTERBERG LIMITS TESTING

The plastic limit and liquid limit (Atterberg limits) of select soil samples were determined in accordance with ASTM D4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil and evaluation of liquefaction susceptibility. The plastic limit is defined as the moisture content (in percent) where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

DRY DENSITY

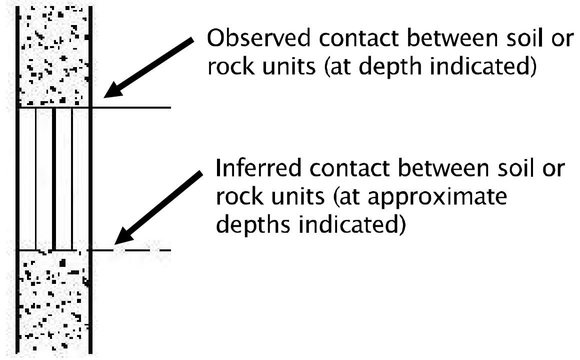
We tested select soil samples to determine the in situ dry densities in general accordance with ASTM D2937. The dry density is defined as the ratio of the dry weight of the soil sample to the volume of that sample. The dry density typically is expressed in units of pcf. The test results are presented in this appendix.

PARTICLE-SIZE TESTING

Particle-size testing was completed on selected soil samples. Testing consisted of percent fines determinations conducted in general accordance with ASTM D1140. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
◧	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test with recovery
◧	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
◧	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
◧	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
◧	Location of sample collected using 3-inch-O.D. California split-spoon sampler and 140-pound hammer with recovery
☒	Location of grab sample
▮	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium Dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium Stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very Stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

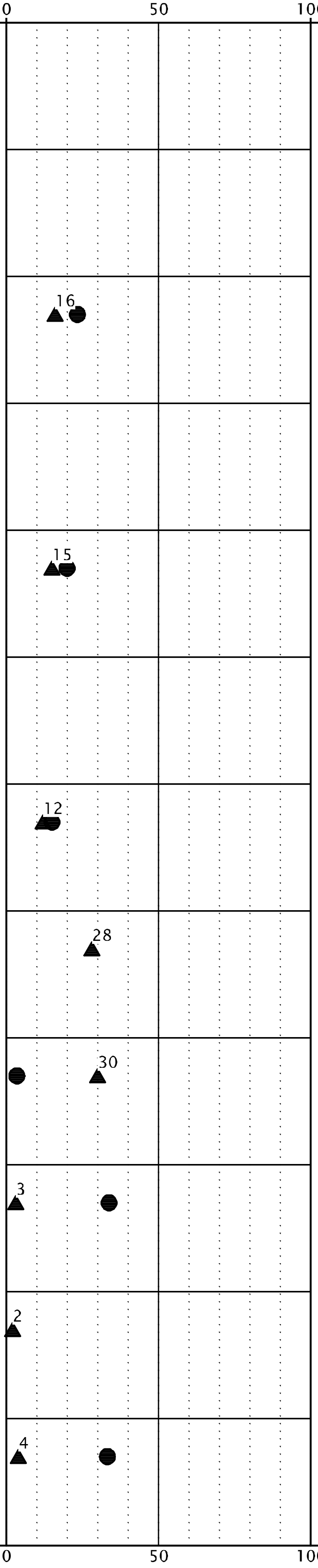
Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

SOIL CLASSIFICATION SYSTEM

TABLE A-2

▲ BLOW COUNT
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (2.5 inches).	35.0 34.8 0.2			
	AGGREGATE BASE (3.0 inches).	34.5 0.5			
	Stiff, dark brown CLAY (CL), trace sand; moist, sand is fine.				
5.0	brown at 5.0 feet		DD	16	DD = 102 pcf
7.5	Stiff, brown, sandy CLAY (CL); moist, sand is fine, micaceous.	28.0 7.0			
10.0			DD	15	DD = 106 pcf
15.0	Medium dense, clayey SAND with silt (SC); moist, sand is fine, iron oxide staining, micaceous.	21.0 14.0	P200	12	P200 = 37%
17.5	Medium dense, light brown SAND (SP); moist, sand is medium to coarse.	18.0 17.0		28	
20.0	dense, orange-brown, trace gravel; gravel is fine to coarse and subrounded to subangular at 20.0 feet			30	
22.5	Soft, gray-brown CLAY (CH); moist, micaceous.	13.0 22.0	ATT	3	LL = 58% PL = 20%
25.0	iron oxide veins at 25.0 feet			2	
27.5	medium stiff, blue-gray, sandy at 27.5 feet			4	
30.0	Medium dense, blue-gray, silty SAND (SM); moist, sand is fine, micaceous.	6.0 29.0			

DRILLED BY: Martini Drilling
LOGGED BY: B. Watkins
COMPLETED: 03/25/19

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 8 inches

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

APRIL 2019

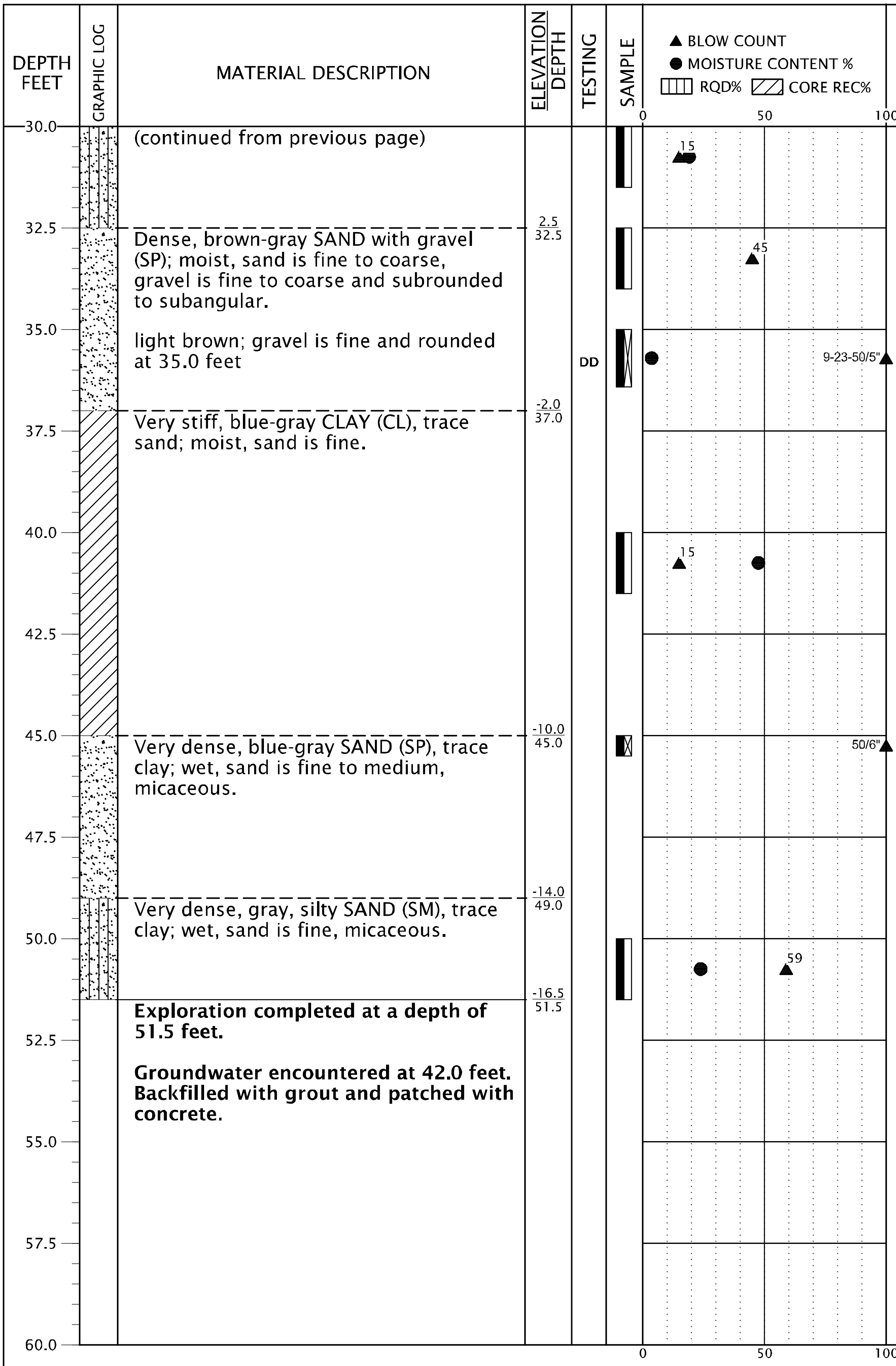
BORING B-1

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-1

BORING LOG 3MRCAP-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 4/26/19:KM:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
30.0	(continued from previous page)			
32.5	Dense, brown-gray SAND with gravel (SP); moist, sand is fine to coarse, gravel is fine to coarse and subrounded to subangular.	2.5 32.5		Rig chatter at 33.0 feet.
35.0	light brown; gravel is fine and rounded at 35.0 feet		DD	DD = 112 pcf
37.0	Very stiff, blue-gray CLAY (CL), trace sand; moist, sand is fine.	-2.0 37.0		⊻ 42.0 feet during drilling
45.0	Very dense, blue-gray SAND (SP), trace clay; wet, sand is fine to medium, micaceous.	-10.0 45.0		
49.0	Very dense, gray, silty SAND (SM), trace clay; wet, sand is fine, micaceous.	-14.0 49.0		
51.5	**Exploration completed at a depth of 51.5 feet.** **Groundwater encountered at 42.0 feet. Backfilled with grout and patched with concrete.**	-16.5 51.5		



DRILLED BY: Martini Drilling

LOGGED BY: B. Watkins

COMPLETED: 03/25/19

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

GEODESIGN INC

9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

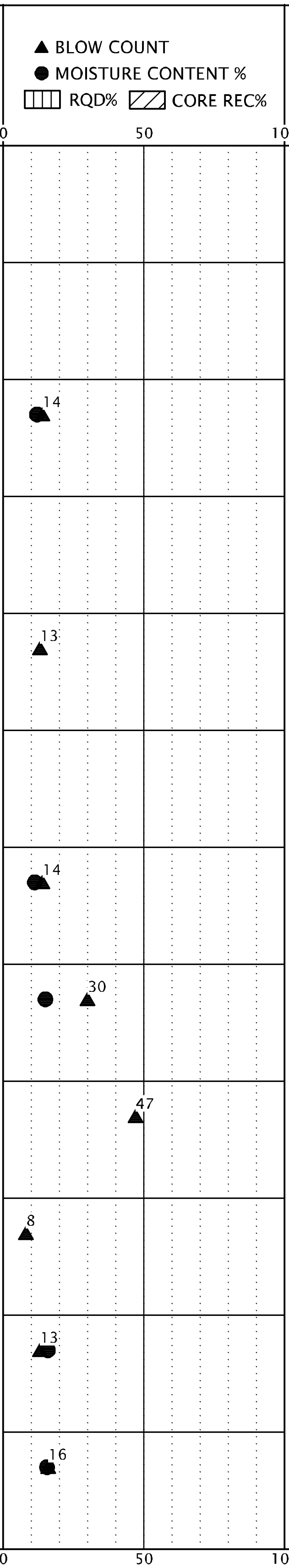
APRIL 2019

BORING B-1
(continued)

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (5.5 inches).	36.0		
0.5	Stiff, dark brown, sandy CLAY (CL); moist, sand is fine. layer of SAND; sand is coarse (1 inch thick) at 0.6 foot with sand at 6.0 feet	35.5 0.5		
14.0	Medium dense, light brown, clayey SAND (SC), trace silt; moist, sand is fine. dense at 19.0 feet	22.0 14.0	DD	DD = 105 pcf
23.0	Stiff, gray-brown with brown mottled, sandy CLAY (CL), minor silt; moist, sand is fine, micaceous. very stiff at 27.5 feet	13.0 23.0	P200	P200 = 66%



DRILLED BY: Martini Drilling

LOGGED BY: B. Watkins

COMPLETED: 03/26/19

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

APRIL 2019

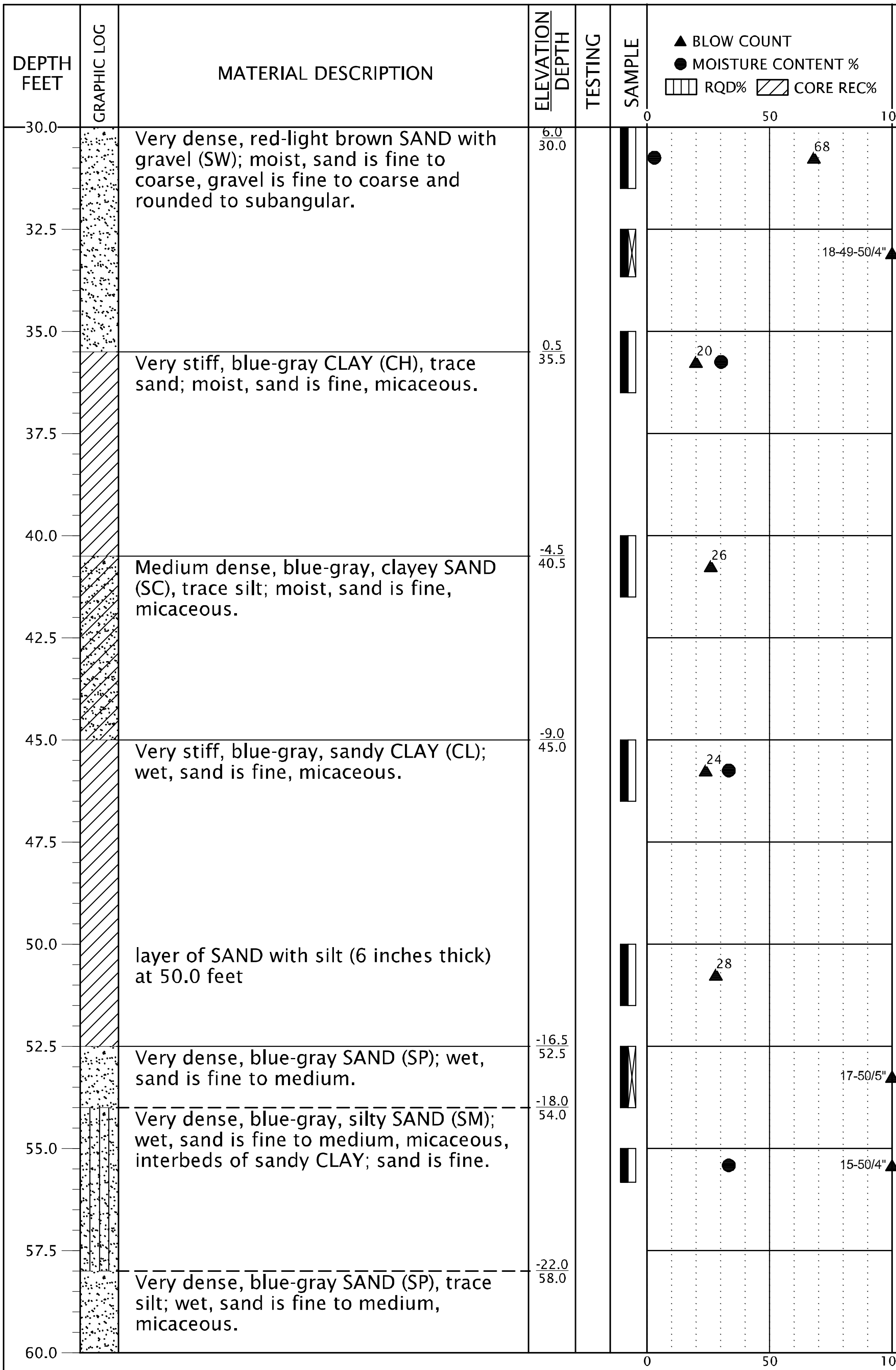
BORING B-2

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-2

BORING LOG 3MRCAP-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 4/26/19:KM:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
30.0	Very dense, red-light brown SAND with gravel (SW); moist, sand is fine to coarse, gravel is fine to coarse and rounded to subangular.	6.0 30.0	Rig chatter at 30.0 feet.
35.5	Very stiff, blue-gray CLAY (CH), trace sand; moist, sand is fine, micaceous.	0.5 35.5	
40.5	Medium dense, blue-gray, clayey SAND (SC), trace silt; moist, sand is fine, micaceous.	-4.5 40.5	▽ 43.0 feet during drilling
45.0	Very stiff, blue-gray, sandy CLAY (CL); wet, sand is fine, micaceous.	-9.0 45.0	
50.0	layer of SAND with silt (6 inches thick) at 50.0 feet		
52.5	Very dense, blue-gray SAND (SP); wet, sand is fine to medium.	-16.5 52.5	
54.0	Very dense, blue-gray, silty SAND (SM); wet, sand is fine to medium, micaceous, interbeds of sandy CLAY; sand is fine.	-18.0 54.0	
58.0	Very dense, blue-gray SAND (SP), trace silt; wet, sand is fine to medium, micaceous.	-22.0 58.0	



DRILLED BY: Martini Drilling

LOGGED BY: B. Watkins

COMPLETED: 03/26/19

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

GEODESIGN INC

9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

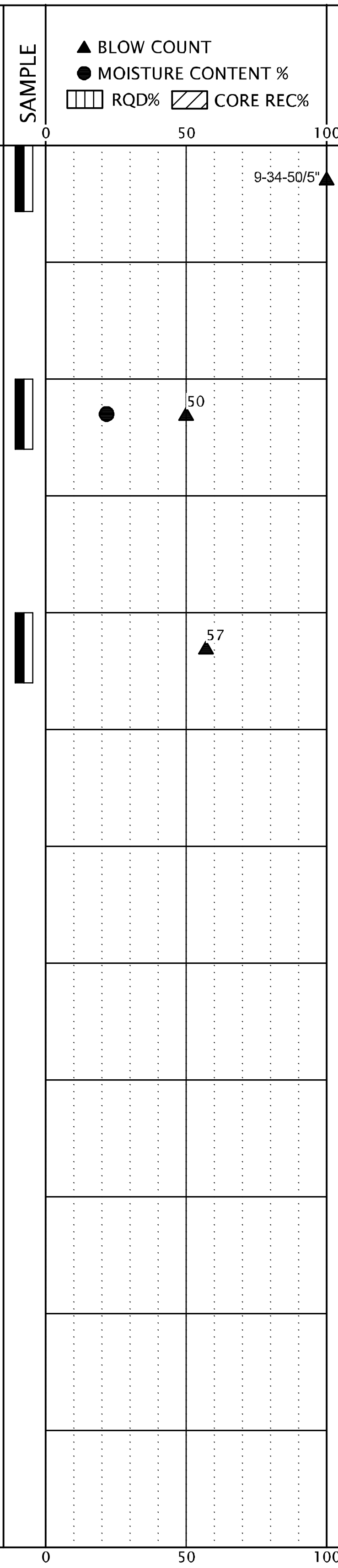
APRIL 2019

BORING B-2
(continued)

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING
60.0	(continued from previous page)		
64.0	Dense, blue-gray SAND with silt (SP-SM); wet, sand is fine to medium, micaceous.	-28.0 64.0	
71.5	**Exploration completed at a depth of 71.5 feet.** **Groundwater encountered at 43.0 feet. Backfilled with grout and patched with concrete.**	-35.5 71.5	



INSTALLATION AND COMMENTS

DRILLED BY: Martini Drilling

LOGGED BY: B. Watkins

COMPLETED: 03/26/19

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

GEODESIGN INC

9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

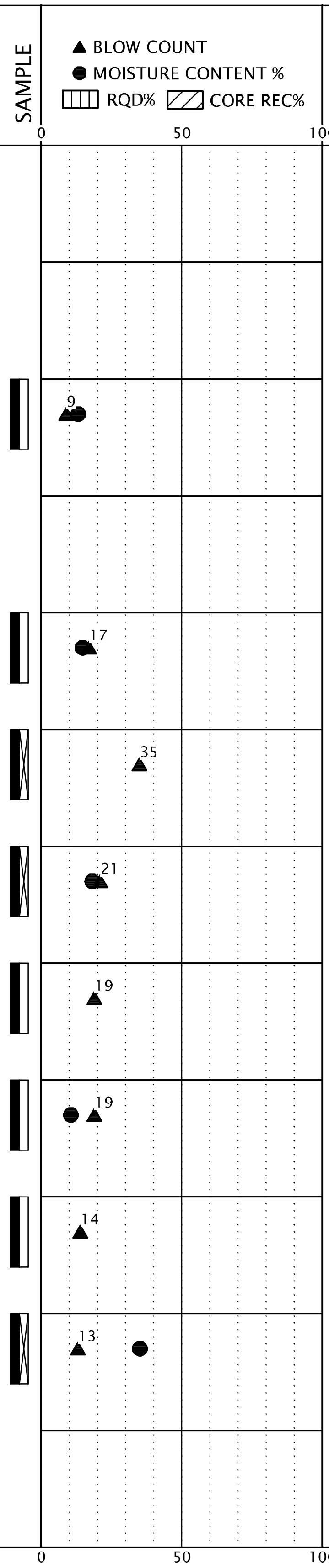
APRIL 2019

BORING B-2
(continued)

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (2.5 inches).	35.0		
	Stiff, dark brown CLAY (CL), trace sand; moist, sand is fine. layer of SAND with clay; sand is coarse (3.5 inches thick) at 0.5 foot	34.8 0.2		Bulk sample collected from 0.0 to 5.0 feet.
2.5				
5.0	brown at 5.5 feet			
7.5				
10.0	Very stiff, brown, sandy CLAY (CL); moist, sand is fine, micaceous.	25.0 10.0		
	Medium dense, brown, silty SAND (SM), trace clay; moist, sand is fine, micaceous.	23.0 12.0		
12.5				
15.0	gray-brown at 15.0 feet		P200	P200 = 47%
17.5	interbeds of SAND, trace silt and sandy CLAY at 17.5 feet			
20.0			P200	P200 = 18%
22.5				
	Stiff, brown CLAY (CH), trace sand; moist, sand is fine. dark blue-gray at 25.0 feet	11.5 23.5	ATT	LL = 62% PL = 21%
25.0				
27.5				
	Medium dense, gray-brown, silty SAND (SM), trace clay; moist, sand is fine,	6.0 29.0		
30.0				



DRILLED BY: Martini Drilling

LOGGED BY: B. Watkins

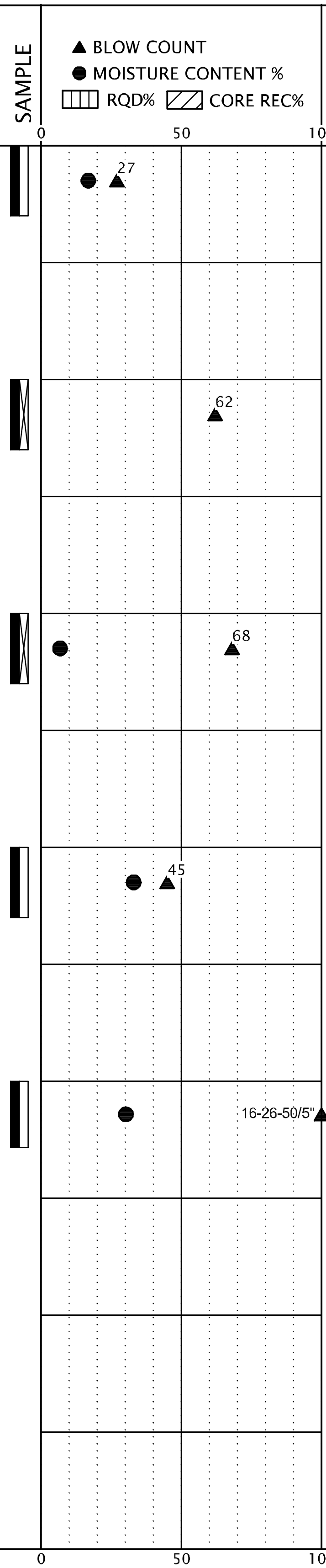
COMPLETED: 03/25/19

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	3MRCAP-1-01	**BORING B-3**
	APRIL 2019	PROPOSED MIXED-USE DEVELOPMENT CULVER CITY, CA
		FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
30.0	micaceous.			
35.0	Dense, light brown SAND with gravel (SP); wet, sand is medium to coarse, gravel is fine to coarse and rounded to subangular.	0.0 35.0		Rig chatter at 35.0 feet.
37.5				∇ 38.0 feet during drilling
40.0	gravel is rounded at 40.0 feet		DD	DD = 117 pcf
43.0	Dense, blue-gray SAND with clay (SP-SC); wet, sand is medium to coarse, micaceous, interbeds of SAND, trace clay and sandy CLAY.	-8.0 43.0		
48.0	Very dense, blue-gray, silty SAND (SM), trace clay; wet, sand is fine, micaceous.	-13.0 48.0		
51.4	**Exploration completed at a depth of 51.4 feet.** **Groundwater encountered at 38.0 feet. Backfilled with grout and patched with concrete.**	-16.4 51.4		



DRILLED BY: Martini Drilling

LOGGED BY: B. Watkins

COMPLETED: 03/25/19

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

GEODESIGN INC

9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

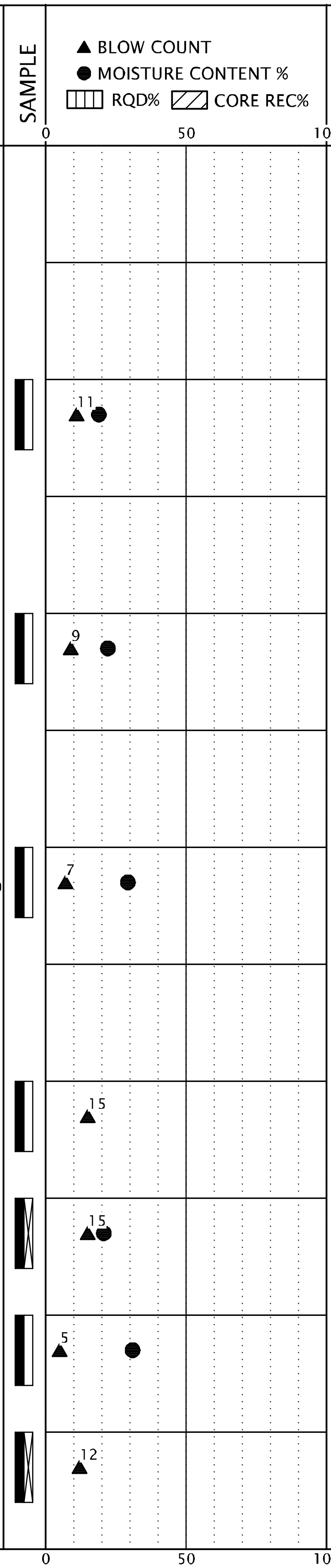
3MRCAP-1-01

APRIL 2019

BORING B-3
(continued)

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-3



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.5 inches).	35.0 34.7 0.3			Bulk sample collected from 0.0 to 5.0 feet.
	AGGREGATE BASE (4.0 inches).	34.4 0.6			
	Stiff, dark brown CLAY (CL), trace sand; moist, sand is fine.				
2.5					
5.0				11	
7.5					
	Stiff, light brown with gray-brown mottled CLAY with sand (CL); moist, sand is fine, micaceous.	26.0 9.0			
10.0				9	
12.5					
15.0	medium stiff, minor sand at 15.0 feet		P200	7	P200 = 91%
17.5					
	Medium dense, light brown SAND with clay (SP-SC), trace gravel; moist, sand is medium to coarse, gravel is fine to coarse and rounded to subrounded, interbeds of SAND, trace clay and sandy CLAY.	16.0 19.0			
20.0				15	
	Stiff, dark brown CLAY (CH), trace sand; moist, sand is fine.	13.5 21.5			
22.5	layer of SAND, trace clay (6 inches thick) at 23.5 feet		DD	15	DD = 110 pcf
25.0	medium stiff at 25.0 feet			5	
	Medium stiff, blue-gray, sandy CLAY (CL), some silt; moist, sand is fine, micaceous.	8.0 27.0			
27.5				12	
	Loose, blue-gray, silty SAND (SM); moist, sand is fine to medium, micaceous.	6.0 29.0			
30.0					

DRILLED BY: Martini Drilling
LOGGED BY: B. Watkins
COMPLETED: 03/25/19

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 8 inches

BORING LOG 3MRCAP-1-01-B1_4.GPJ GEODESIGN.GDT PRINT DATE: 4/26/19:KM:KT

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

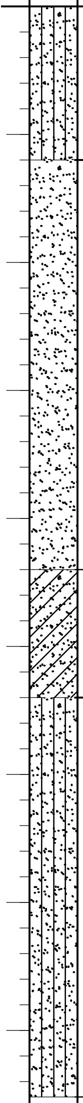
APRIL 2019

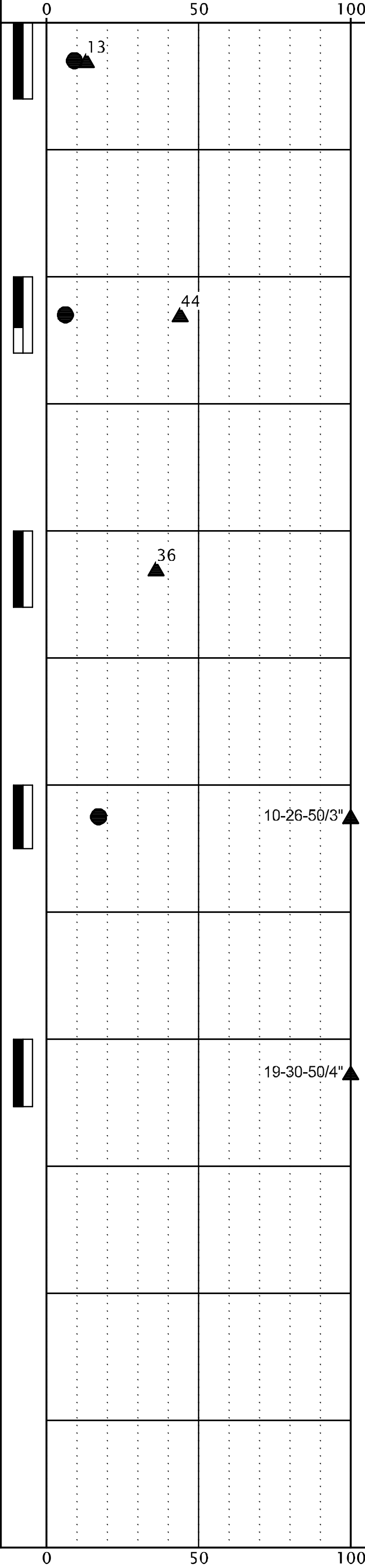
BORING B-4

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
30.0	(continued from previous page)		P200	13	P200 = 13%
33.0	Dense, light brown SAND (SP), trace silt; moist, sand is fine to medium.	2.0 33.0			Driller Comment: increased hardness during drilling at 33.0 feet.
35.0				44	
40.0	trace gravel; sand is coarse, gravel is fine to coarse and rounded to subangular at 40.0 feet			36	
41.0	Dense, blue-gray, clayey SAND (SC), trace silt; moist, sand is fine, micaceous.	-6.0 41.0			⊻ 42.0 feet during drilling
43.5	Very dense, blue-gray, silty SAND with clay (SM); wet, sand is fine to medium, micaceous, interbeds of silty SAND and sandy CLAY.	-8.5 43.5		10-26-50/3"	
50.0				19-30-50/4"	
51.3	**Exploration completed at a depth of 51.3 feet.** **Groundwater encountered at 42.0 feet. Backfilled with grout and patched with concrete.**	-16.3 51.3			





DRILLED BY: Martini Drilling

LOGGED BY: B. Watkins

COMPLETED: 03/25/19

BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

GEODESIGN INC

9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

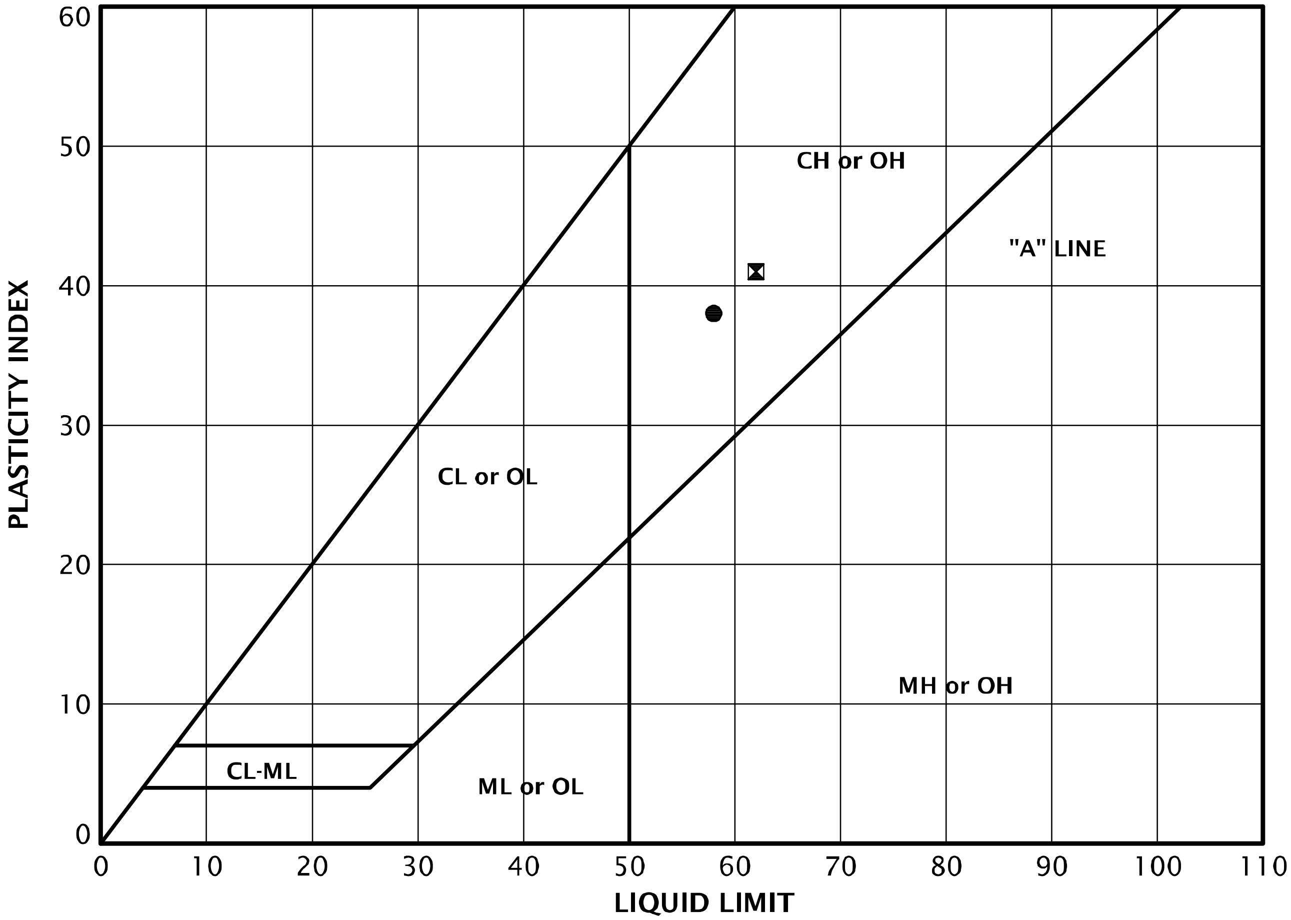
3MRCAP-1-01

APRIL 2019

BORING B-4
(continued)

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-4



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	22.5	34	58	20	38
☒	B-3	25.0	35	62	21	41

GEODESIGN INC

9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

APRIL 2019

ATTERBERG LIMITS TEST RESULTS

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-5

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	5.0	30.0	23	102						
B-1	10.0	25.0	20	106						
B-1	15.0	20.0	15				37			
B-1	20.0	15.0	3							
B-1	22.5	12.5	34					58	20	38
B-1	27.5	7.5	33							
B-1	30.0	5.0	19							
B-1	35.0	0.0	4	112						
B-1	40.0	-5.0	47							
B-1	50.0	-15.0	24							
B-2	5.0	31.0	12							
B-2	15.0	21.0	11							
B-2	17.5	18.5	15	105						
B-2	25.0	11.0	16							
B-2	27.5	8.5	16				66			
B-2	30.0	6.0	3							
B-2	35.0	1.0	30							
B-2	45.0	-9.0	33							
B-2	55.0	-19.0	33							
B-2	65.0	-29.0	22							
B-3	5.0	30.0	13							
B-3	10.0	25.0	15							
B-3	15.0	20.0	18				47			
B-3	20.0	15.0	11				18			
B-3	25.0	10.0	35					62	21	41
B-3	30.0	5.0	17							
B-3	40.0	-5.0	7	117						

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

3MRCAP-1-01

APRIL 2019

SUMMARY OF LABORATORY DATA

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-6

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-3	45.0	-10.0	33							
B-3	50.0	-15.0	30							
B-4	5.0	30.0	19							
B-4	10.0	25.0	22							
B-4	15.0	20.0	29				91			
B-4	22.5	12.5	21	110						
B-4	25.0	10.0	31							
B-4	30.0	5.0	9				13			
B-4	35.0	0.0	6							
B-4	45.0	-10.0	17							

3MRCAP-1-01

APRIL 2019

SUMMARY OF LABORATORY DATA
(continued)

PROPOSED MIXED-USE DEVELOPMENT
CULVER CITY, CA

FIGURE A-6

APPENDIX B

APPENDIX B

CONE PENETRATION TESTING

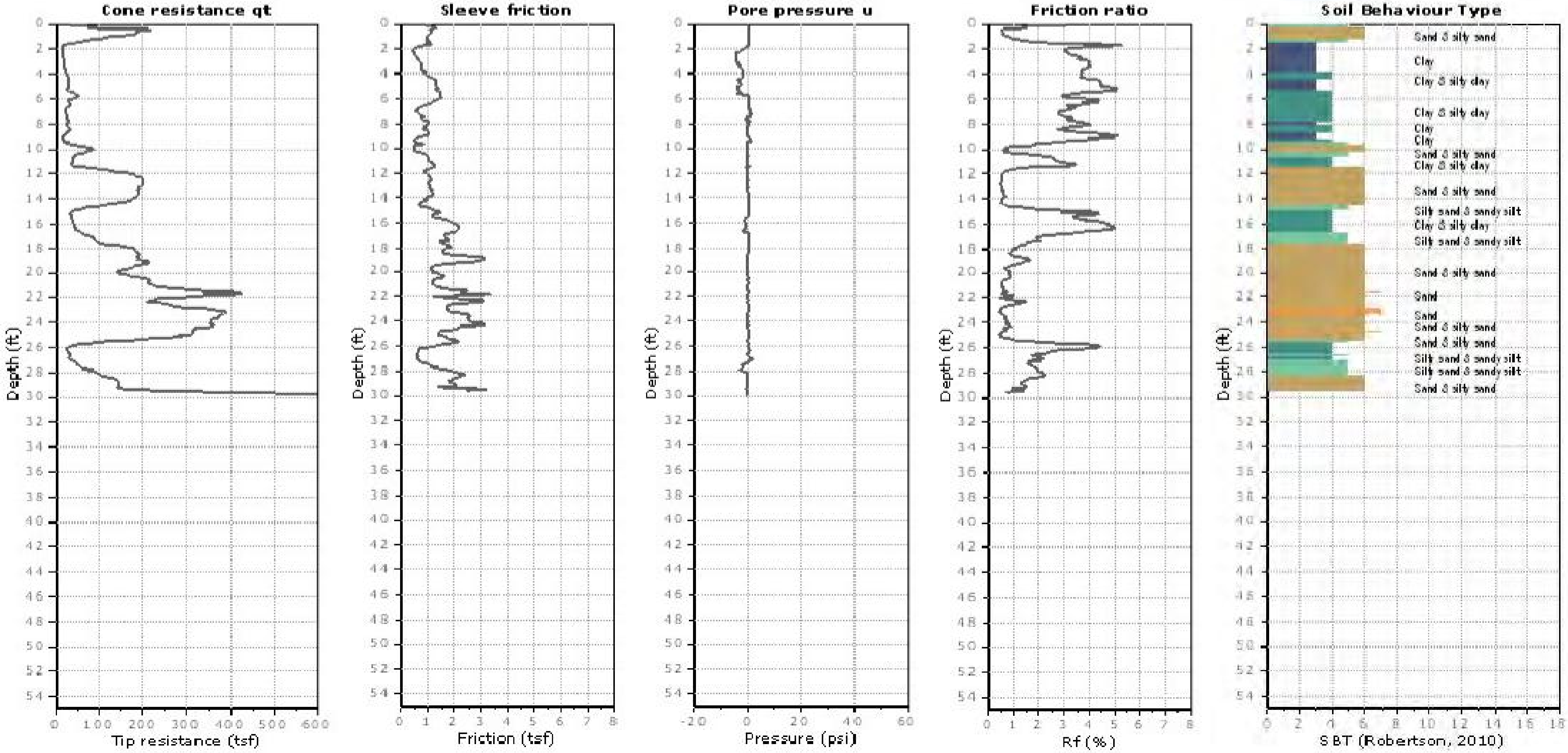
Kehoe Testing & Engineering of Huntington Beach, California, advanced three CPT probes (CPT-1 through CPT-3) to depths between 29.9 and 53.6 feet BGS on March 25, 2019. The CPT probes were completed in accordance with ASTM D5778 using an integrated electronic cone system manufactured by Vertek. One pore-pressure dissipation test was conducted at a depth of 50.1 feet BGS in CPT-3.

The approximate locations of the probes are shown on Figure 2. The CPT logs are presented in this appendix.

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: GeoDesign
Location: 11111 Jefferson Blvd, Culver City, CA

CPT-1
Total depth: 29.93 ft, Date: 3/26/2019



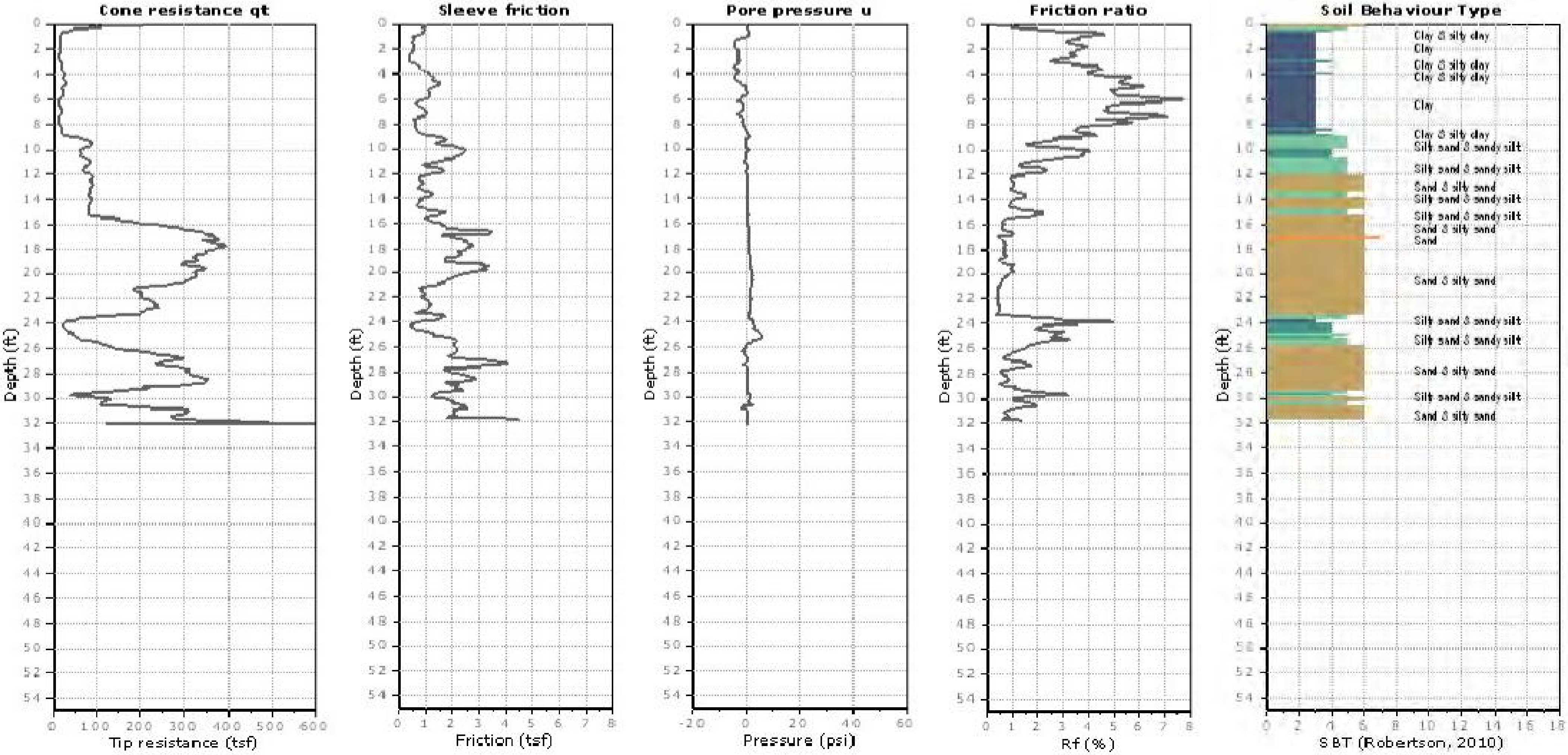
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **GeoDesign**

Location: **11111 Jefferson Blvd, Culver City, CA**

CPT-2

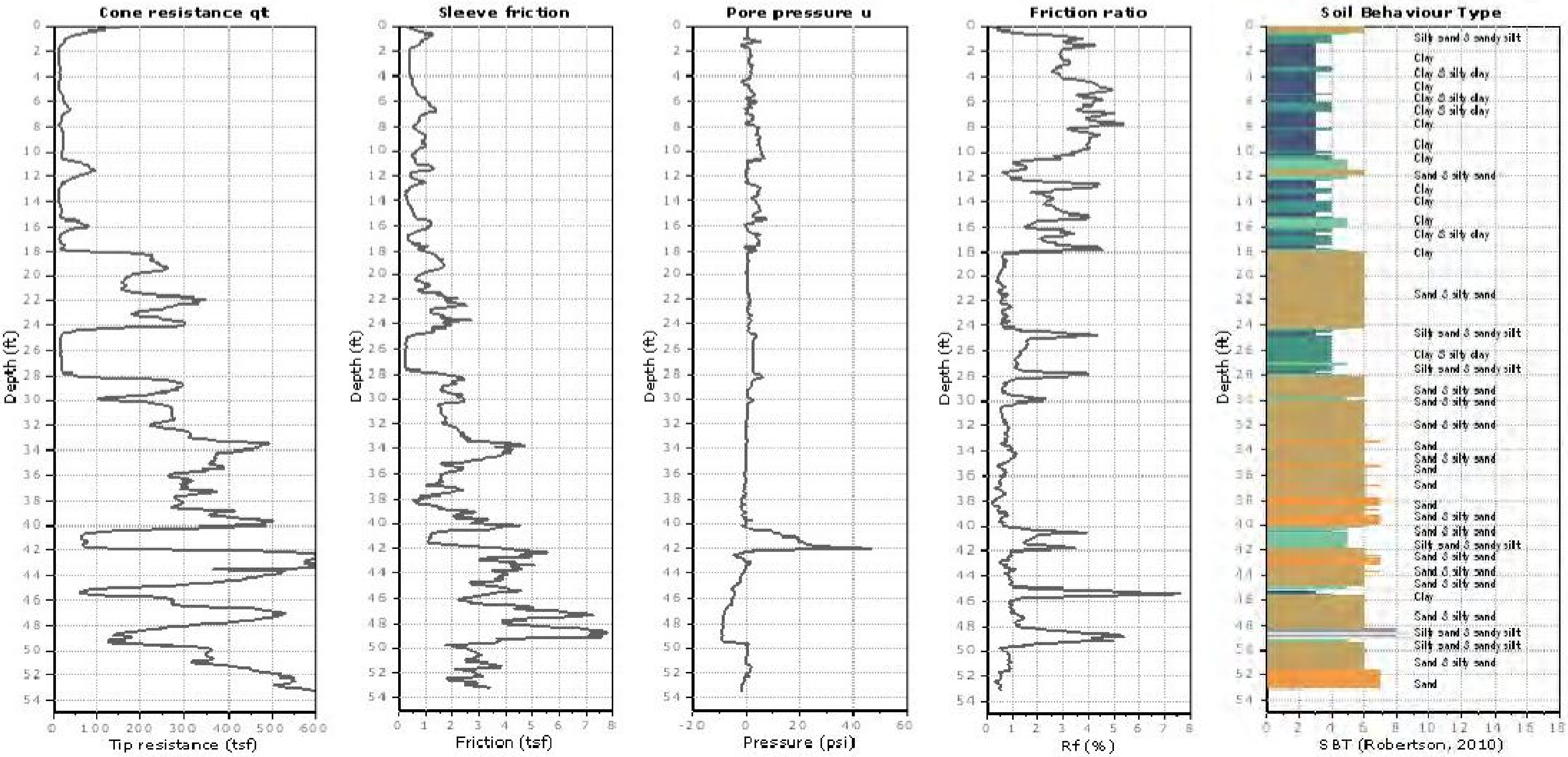
Total depth: 32.16 ft, Date: 3/26/2019



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **GeoDesign**
Location: **11111 Jefferson Blvd, Culver City, CA**

CPT-3
Total depth: 53.55 ft, Date: 3/26/2019



www.geodesigninc.com