PROJECT NO.: CPRKVALL0001

HYDROLOGY REPORT

-FOR-

VALLEY CENTER

APN(S): 8206-027-058 & 8206-027-080
LOT(S)/PARCEL(S):
PLANNING APPLICTION #: TBD
GRADING PERMIT #: TBD
BUILDING PERMIT #: TBD

PREPARED FOR: CRP IV Valley Center, LLC
1300 Dove St., Suite 200
Newport Beach, CA 92660
Attn: Kevin Koh
kkoh@caprock-partners.com

PREPARED BY: **Jose Cruz, PE**
DAVID EVANS & ASSOCIATES, INC
25152 Springfield Court, Suite 350
Santa Clarita, CA 91355
661.284.7462

DATE PREPARED: JULY 2025

DAVID EVANS AND ASSOCIATES INC.

This report has been prepared by or under the direction of the following registered civil engineer who attests to the technical information contained herein. The registered civil engineer has also judged the qualifications of any technical specialists providing engineering data upon which recommendations, conclusions, and decisions are based.

Jose Cruz, PE	1/21/26 Date

TABLE OF CONTENTS

SECTION I. INTRODUCTION AND PURPOSE

The purpose of this report is to provide engineering calculations in support of the Valley Center project for 50-year storm event peak flows resulting from the developed site and to demonstrate that 'post-developed' hydrologic conditions do not exceed 'pre-developed' levels.

SECTION II. PROJECT LOCATION AND DESCRIPTION

II.1 EXISTING CONDITIONS

The project address is 14428 Valley Boulevard, which is located at the south corner of South 6th Avenue and Valley Boulevard in the city of Industry. The APNs are 8206-027-058 & 8206-027-080. Surrounding uses predominantly include industrial warehouses.

The existing site consists of 7 buildings, providing warehouse and office space facilities. The exterior space is comprised of asphalt hardscape used for parking and loading/unloading. The existing site drainage is collected into concrete v-gutters that sheet flow out to 6th Avenue. There are no existing on-site below ground storm facilities. Street flows are then collected at the existing catch basins on 6th Avenue. There is one existing curb inlet catch basin directly fronting the site and another inlet along 6th closer to Proctor Avenue. Refer to Appendix A, Pre-Development Hydrology Exhibit There is no discharge into Valley Boulevard from the existing site.

II.2 EXISTING GEOTECHNICAL CONDITIONS

The site is located within the San Gabriel Valley, an alluvial basin drained by the Rio Hondo and San Gabriel Rivers, in the Peninsular Ranges geomorphic province.

Based on the geotechnical findings, the site is underlain by a mantle of artificial fill over native sediments. Because reporting of the engineering and placement of the artificial fill was not available at the time the report was prepared, the artificial fill was documented as undocumented.

The historical high groundwater levels in the region of the site were found to be approximartely 33 feet bgs. However, groundwater was not encountered in the exploratory borings performed on April 12, 2022, which reached as deep as 51.5 feet below ground surface (bgs).

Infiltration testing was performed at two locations at depths of 10.2 and 25.4 feet bgs; raw infiltration rates of 0.8 in/hr and 1.7 in/hr were measured respectively. Based on these results, the Geotechnical report suggests that infiltration is only feasible at depths 20-25 feet bgs with a raw, design infiltration rate of 1.0 in/hr. The adjusted, recommended design infiltration rate is 0.3 in/hr at depths of 20-25 feet bgs, which includes the LAC required factor of safety of 3.

Refer to Geotechnical report included in Appendix 'E' of this study for additional information.

II.3 PROPOSED CONDITIONS

The project proposes a 135,720 square foot industrial building with office space. No subterranean levels are anticipated. Additional improvements include new drive aisle, at-grade parking and site landscaping.

Runoff from the site will be conveyed via sheet flow into concrete valley gutters which drain to several catch basin inlets before being conveyed into two separate underground corrugated metal pipe (CMP) chamber systems (Basin 'A' and Basin 'B'), as shown in Appendix B Post-Construction Hydrology Exhibit. Runoff will discharge from each CMP chamber system into a modular wetland system (MWS).

The MWS's will provide the required water quality treatment for the 85th percentile storm in conformance with the Low Impact Development (LID) requirements. The CMP chambers are sized to detain runoff before being treated by the MWS's for the 85th percentile storm. Outflow for storm events greater than the 85th percentile, including the 50-year storm, will overflow through an external upstream bypass pipe for both basins and into the public storm drain systems in S. 6th Ave and Valley Blvd. Drainage from the building roof will be piped routed through and on-site storm drain system, with 50% of the roof discharging to Basin 'A' CMP chambers and 50% of the roof discharging to Basin 'B' CMP Chambers. Refer to the Post-Developed Hydrology Exhibit in Appendix A.

SECTION III. METHODOLOGY

The calculations and stormwater management methods contained in this report are based on the current City of Industry and Los Angeles County Low Impact Development (LID) standards, and the LA County Hydrology Manual (LACHM).

III.1 HYDROLOGY

Hydrologic calculations for the project were performed using the methods outlined in the LA County Hydrology Manual to calculate the 50-year Storm event. For the small sub watersheds of this project, a time of concentration of 5 minutes is assumed as a minimum. Due to the onsite retention of the majority of the new paved surfaces, flows leaving the site in the "post-developed" condition will be less than the "pre-developed" condition.

Hydrocalc software version 1.0.2, provided by LA County, was used to calculate the storm volumes and flowrates. Input parameters for soil class and isohyetals were gathered from the LA County Hydrology Map at https://dpw.lacounty.gov/wrd/hydrologygis/.

See Appendix B & C' for Hydrocalc inputs and calculations.

III.1.1. 50-YEAR STORM EVENT

The 50-year storm event was calculated using hydrocalc software, with a rainfall depth of 6.25 inches. Storm events greater than the 85th percentile storm, including the 50-year storm event, will overflow to the proposed treatment facilities and drain towards their respective point of connection at the existing catch basins on Valley Boulevard and 6th Avenue. Review of the as built for the offsite storm drain suggest that the storm drain has sufficient capacity to accept the onsite projects 50-year storm events flows, since storm water in the existing condition is already tributary to this storm drain system and reshuffled with a lower peak flow than existing (18.15 cfs vs 16.96 cfs) in the proposed condition. Due to the lower post-developed peak flows, it is assumed that this project is exempt from hydromodification. As-built for the existing public storm drain is referenced in Appendix E of this report.

Hydrology Summary Tables: Existing and Proposed Condition

Existing Subarea ID	Area (Acres)	Impervious Ratio	Q_{ex50} (cfs)
A	4.1	0.91	11.24
B	1.4	0.91	4.11
C	1.0	0.91	2.80
TOTAL	6.5		18.15

Proposed Subarea ID	Area (Acres)	Impervious Ratio	BMP Designation	Q_{Dev50} (cfs)
A	2.6	0.95	Basin 'A'	7.36
B	0.9	0.91	Basin 'B'	2.91
C	2.4	0.94	Basin 'B'	5.58
D	0.6	.01	Self-Treating	1.11
TOTAL	6.5			16.96

III.2 HYDRAULIC ANALYSIS

Storm drain pipes are preliminarily sized to convey 50-yr storm event flows. Pipe sizing will be refined during final engineering. See Appendix 'D' for hydraulic calculations of the Basin A and Basin B systems.

SECTION IV. WATER QUALITY BMP DESIGN

IV.1 LID BMP SELECTION

The project development is required to infiltrate unless site conditions are deemed unfeasible per the latest version of the Los Angeles LID Design Manual. Infiltration was concluded to be infeasible per the Geotechnical Exploration study that was prepared by Leighton Consulting, Inc. Per the study, the infiltration rate was measured at a depth of approximately 5-10 feet below ground surface (BGS) and resulted in an observed rate of 0.8-inch/hr, or 0.27 inches/hr for the design rate with a factor of safety of 3 applied. A second percolation test was done at a depth of 20-25 feet BGS and resulted in a measured rate of 1.7 inches/hr, or 0.56 inches/hr for the design rate. Per the LAC LID handbook, the minimum design rate for infiltration is 0.3 inches/hr and therefore infiltration is infeasible onsite at depths less than 20 feet BGS.

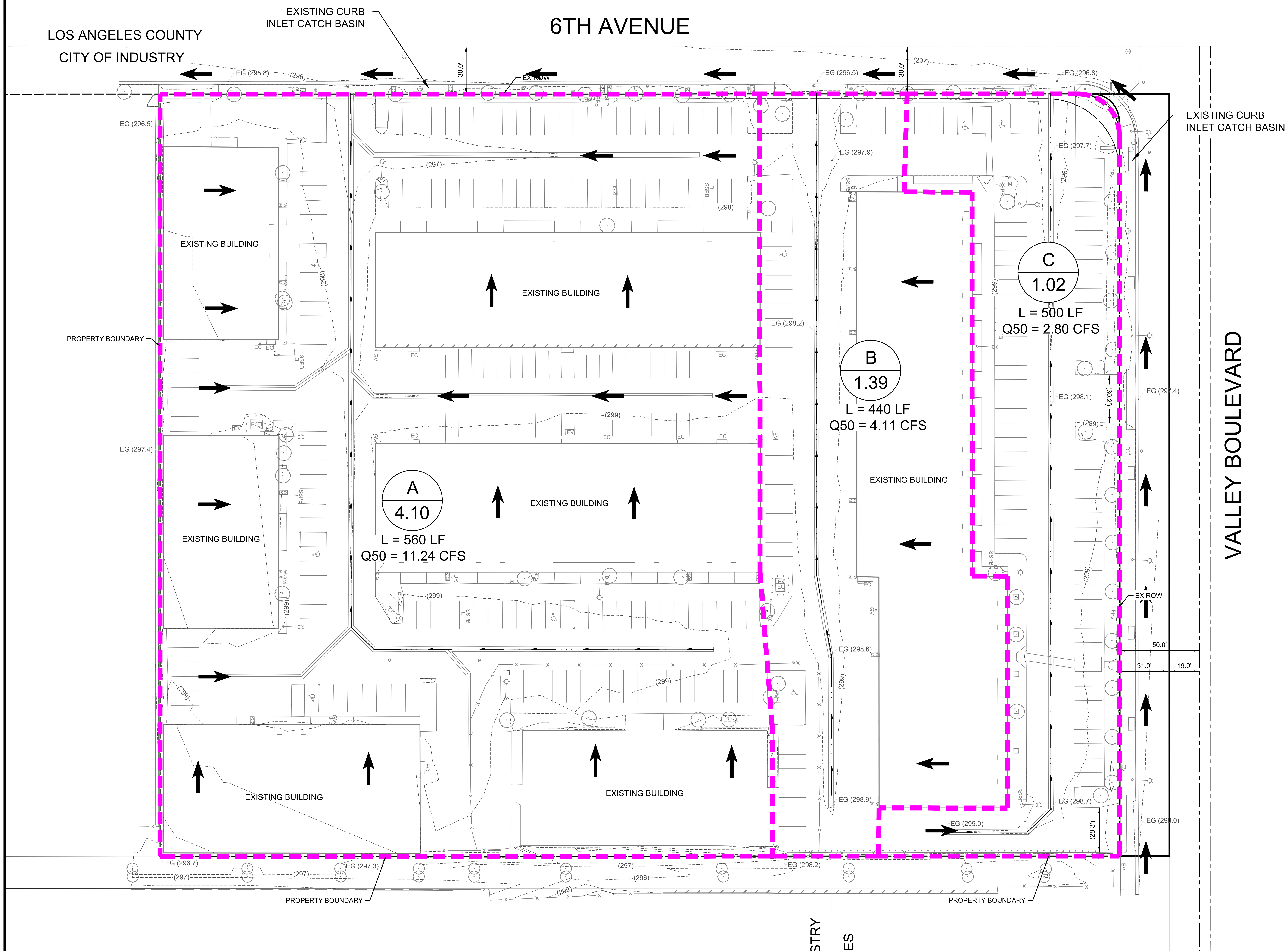
Bioretention was then evaluated and was determined to be infeasible since the project footprint does not have the available space for a traditional above-ground bioretention basin. For that reason, a proprietary bioretention treatment system, Modular Wetland System, is proposed to treat the onsite stormwater. The MWS is designed to treat the water quality volume, in combination of upstream storage, and treats storm water within the allowed time of 96 hours. Refer to the LID Report for additional information.

SECTION V. CONCLUSION

Based on the proposed storm drain improvements of the site, the storm drain facilities have been sized to adequately convey water to the public storm drain system for the 50-year storm event, while also providing conveyance capacity for the treatment of the 85th percentile storm. . The project does not anticipate downstream flooding as a result of the project since land use remains the same.

APPENDIX A: DRAINAGE EXHIBITS

- Pre-development Hydrology Exhibit
- Post-development Hydrology Exhibit



LEGEND

DRAINAGE AREA BOUNDARY

FLOW DIRECTION

FLOW PATH

SUBAREA ID (X)

AREA (Z) IN ACRES

SITE HYDROLOGY INFORMATION

TOTAL AREA (PRE DEDICATION): 6.51 AC
% IMPERVIOUS (LA COUNTY): 91%
85TH % RAINFALL DEPTH: 1.05 INCH
50-YR ISOHYET: 6.25 INCH
Q50: 18.15 CFS
SOIL TYPE: 003

EXISTING HYDROLOGY INFORMATION

SUBAREA ID	AREA (AC)	% IMPERVIOUS	Q_{50} (CFS)
A	4.10	91	11.24
B	1.39	91	4.11
C	1.02	91	2.80
TOTAL	6.51		18.15

SCALE
30 0 15 30 60
(FEET)
1 INCH = 30 FT.

DAVID EVANS AND ASSOCIATES INC.
4141 E. Inland Empire Blvd
Suite 250
Ontario, CA 91764
909.481.5750

VALLEY CENTER

PRE-DEVELOPMENT HYDROLOGY EXHIBIT

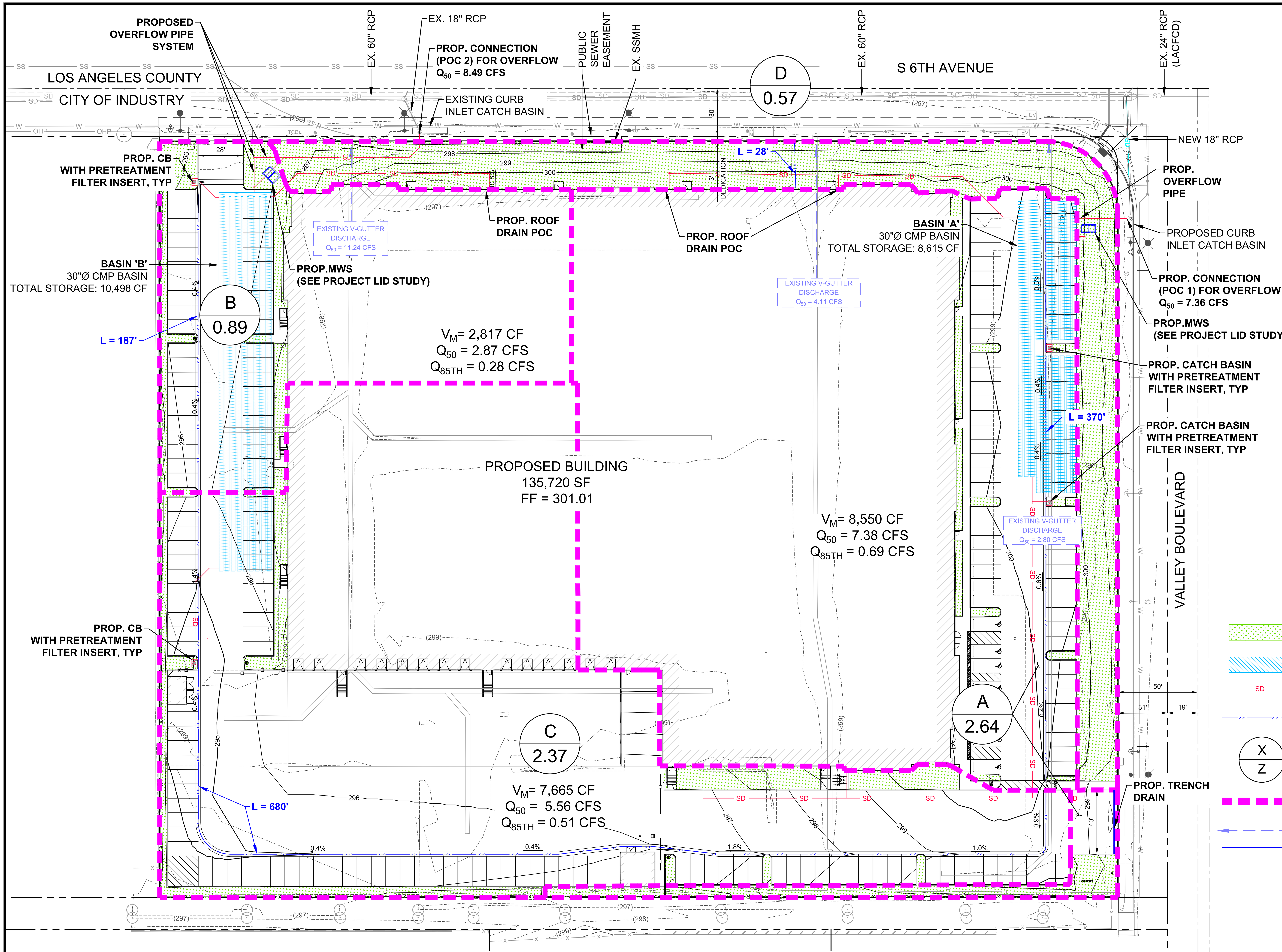
CITY OF INDUSTRY CA

PROJECT NO. CPRKVALL-0001

DATE 4-30-2024

SHEET NO. 1 of 1

Plot Date: 1/20/2026 8:36 AM By: Aaron Routh File: P:\C\CPRKVALL00010400\CAD\EXHIBITS\01_Hydrology Exhibits\CPRKVALL0001-Pre-development Hydrology.dwg
Save Date: 1/19/2026 5:49 PM By: AXRO



SITE HYDROLOGY INFORMATION

TOTAL AREA (POST DEDICATION):	6.47 AC
85TH % RAINFALL DEPTH:	1.05 INCH
50-YR ISOHYET:	6.25 INCH
SOIL TYPE:	003

LEGEND:

- PROPOSED LANDSCAPING
- PROPOSED UNDERGROUND CONTECH CMP BASIN
- SD — PROPOSED ONSITE STORM DRAIN
- FLOW DIRECTION
- X / Z — SUBAREA ID (X) AREA (Z) IN ACRES
- DRAINAGE BOUNDARY
- EXISTING DISCHARGE LOCATION FROM SITE
- TRENCH DRAIN

SCALE 1 INCH = 30 FT. (FEET)

PROPOSED HYDROLOGY INFORMATION

SUBAREA ID	AREA (AC)	% IMPERVIOUS	Q_{50} (CFS)	Q_{85TH} (CFS)
A	2.64	95	7.38	0.69
B	0.89	91	2.87	0.28
C	2.37	94	5.56	0.51
D	0.57	1	1.11	0.03
TOTAL	6.47		16.92	1.51

EXISTING HYDROLOGY INFORMATION

SUBAREA ID	AREA (AC)	% IMPERVIOUS	Q_{50} (CFS)
A	4.10	91	11.24
B	1.39	91	4.11
C	1.02	91	2.80
TOTAL	6.51		18.15

LID INFORMATION

SUBAREA ID	AREA (AC)	% IMPERVIOUS	V_m (CF)	BMP DESIGNATION	BMP SIZE (CF)
A	2.64	95	8,550	BASIN 'A'	8,615
B	0.89	91	2,817	BASIN 'B'	10,498
C	2.37	94	7,665	BASIN 'B'	
D	0.57	1	233	SELF-TREATING	
TOTAL	6.47		19,265		19,113

DAVID EVANS AND ASSOCIATES INC.
4141 E. Inland Empire Blvd
Suite 250
Ontario, CA 91764
909.481.5750

INDUSTRY VALLEY CENTER SITE DEVELOPMENT PLANS

CRP IV INDUSTRY VALLEY CENTER, LLC

POST-DEVELOPED HYDROLOGY EXHIBIT

CITY OF INDUSTRY CA

NO. DATE REVISION REVIEWED BY: DATE: BY CK

90% CD'S

NOT FOR CONSTRUCTION

REGISTERED PROFESSIONAL ENGINEER JOSE JUAN ... CIVIL STATE OF CALIFORNIA

EXPIRES: 03-31-2027

CHECKED BY: AOM
DESIGNED BY: NJRU
DRAWN BY: NJRU

FIRST SUBMITTAL DATE: 03/24/25

PROJECT NO. CPRKVALL0001

SHEET NO. 1 OF 1

Plot Date: 1/20/2026 9:23 AM By: Aaron Routh
Save Date: 1/20/2026 9:23 AM By: AXRO File: P:\C\CPRKVALL00010400CAD\EXHIBITS\01_Hydrology Exhibits\CPRKVALL0001-Post-development Hydrology.dwg

APPENDIX B: HYDROLOGY DATA

- LA County 50-year, 24-hr Isohyetal Except
- LA County Soil Map Excerpt

About | Legend | Layers

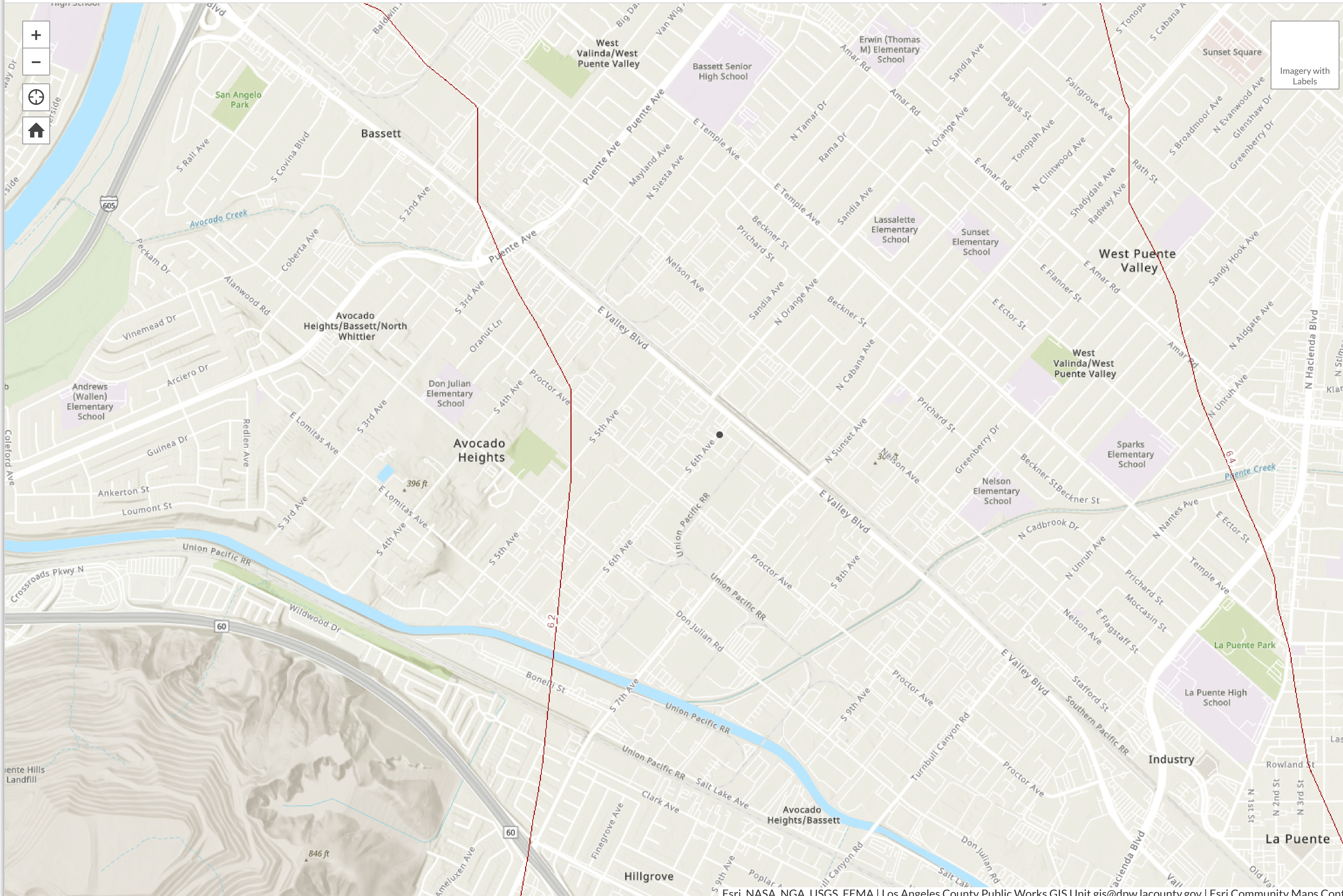
LA County Hydrology Map

110 s 6th street, industry

Legend

Hydrology GIS

50yr Two Tenths (Rainfall)



About | Legend | Layers

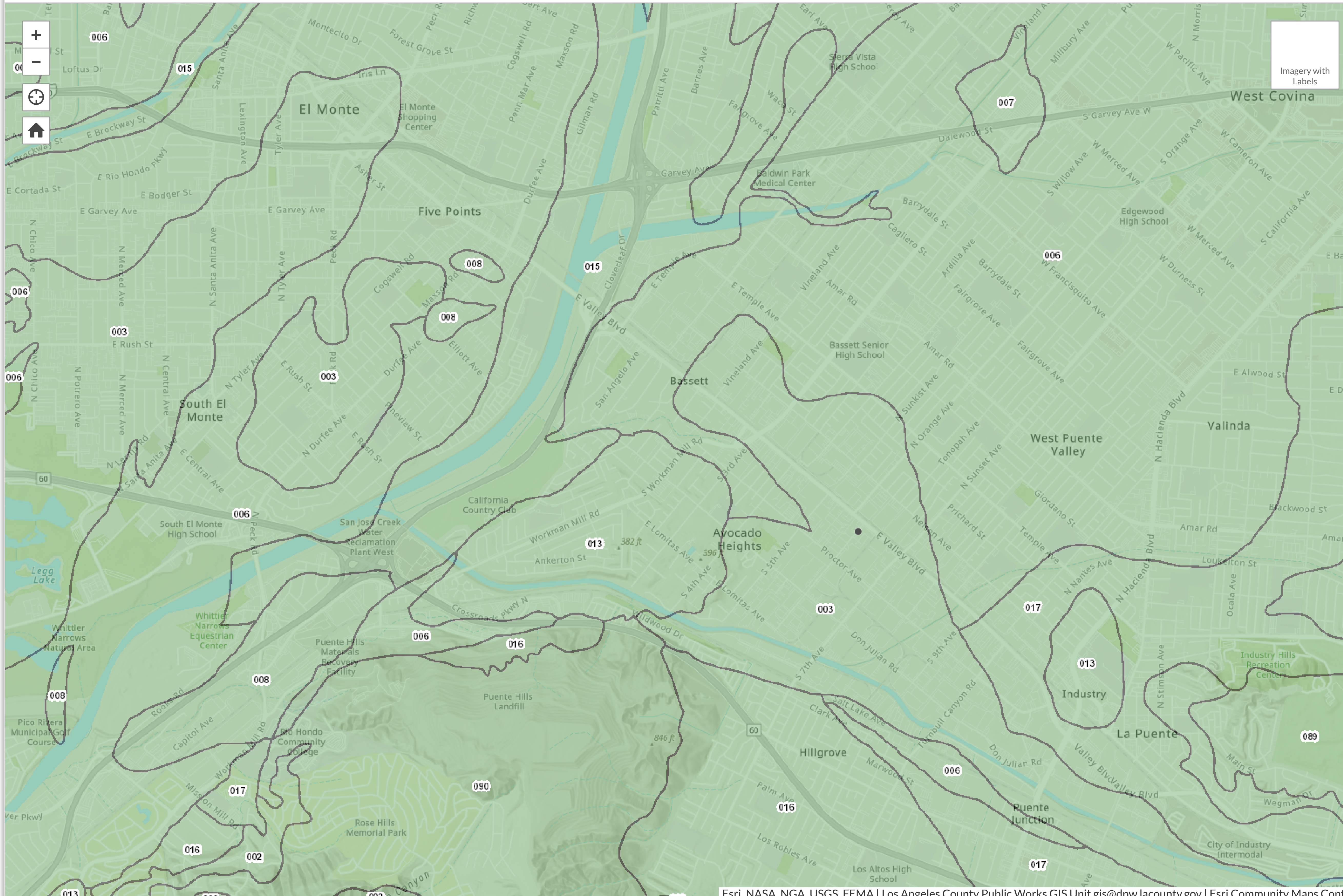
Legend

Hydrology GIS

Soils 2004

LA County Hydrology Map

110 s 6th street, industry



APPENDIX C: HYDROLOGY CALCULATIONS

- 50-year Pre-development Hydrocalc
- 50-year Post-development Hydrocalc
- Proportion of Impervious Area

PRE-DEVELOPED --- SUBAREA 'A' --- 50-YR DESIGN STORM

Peak Flow Hydrologic Analysis

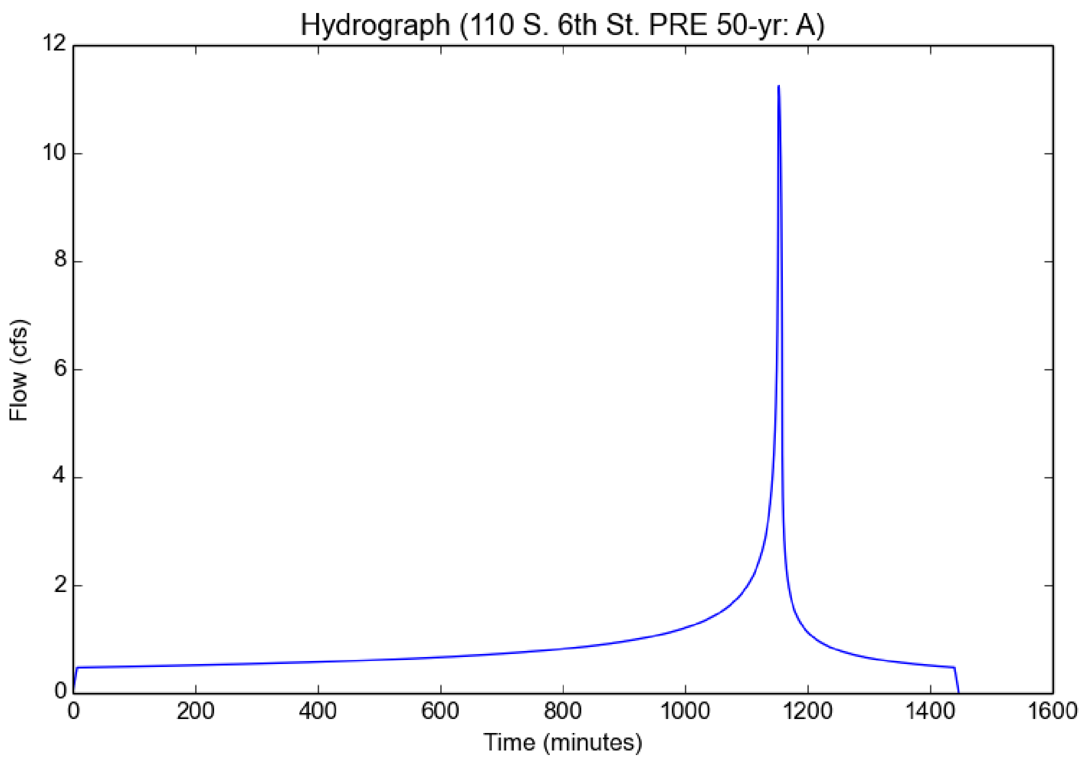
File location: P:/C/CPRKVALL0001/0600INFO/0670Reports/LID & Hydrology Report/Hydrocalc/Pre-development/110 S. 6th St. PRE Report.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	110 S. 6th St. PRE 50-yr
Subarea ID	A
Area (ac)	4.1
Flow Path Length (ft)	560.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	6.25
Percent Impervious	0.91
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	6.25
Peak Intensity (in/hr)	3.1835
Undeveloped Runoff Coefficient (Cu)	0.4694
Developed Runoff Coefficient (Cd)	0.8612
Time of Concentration (min)	7.0
Clear Peak Flow Rate (cfs)	11.2412
Burned Peak Flow Rate (cfs)	11.2412
24-Hr Clear Runoff Volume (ac-ft)	1.7583
24-Hr Clear Runoff Volume (cu-ft)	76592.0187



PRE-DEVELOPED --- SUBAREA 'B' --- 50-YR DESIGN STORM

Peak Flow Hydrologic Analysis

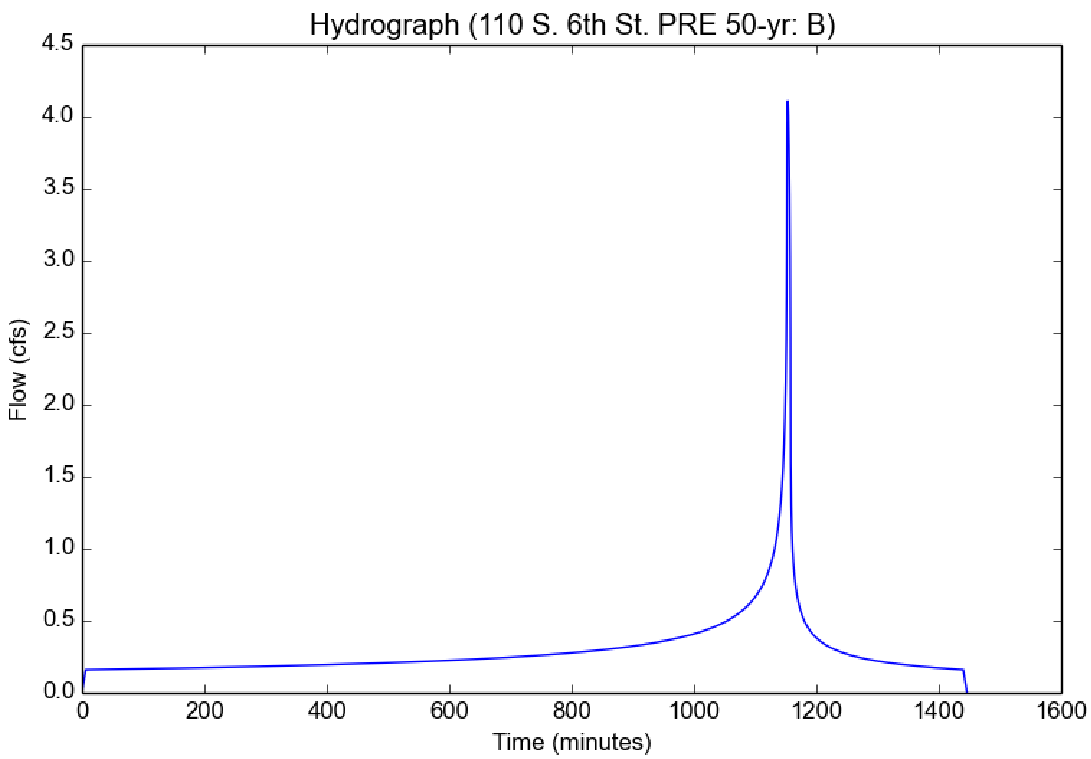
File location: P:/C/CPRKVALL0001/0600INFO/0670Reports/LID & Hydrology Report/Hydrocalc/Pre-development/110 S. 6th St. PRE Report.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	110 S. 6th St. PRE 50-yr
Subarea ID	B
Area (ac)	1.39
Flow Path Length (ft)	440.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	6.25
Percent Impervious	0.91
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	6.25
Peak Intensity (in/hr)	3.4227
Undeveloped Runoff Coefficient (Cu)	0.4945
Developed Runoff Coefficient (Cd)	0.8635
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	4.1082
Burned Peak Flow Rate (cfs)	4.1082
24-Hr Clear Runoff Volume (ac-ft)	0.5961
24-Hr Clear Runoff Volume (cu-ft)	25968.173



PRE-DEVELOPED --- SUBAREA 'C' --- 50-YR DESIGN STORM

Peak Flow Hydrologic Analysis

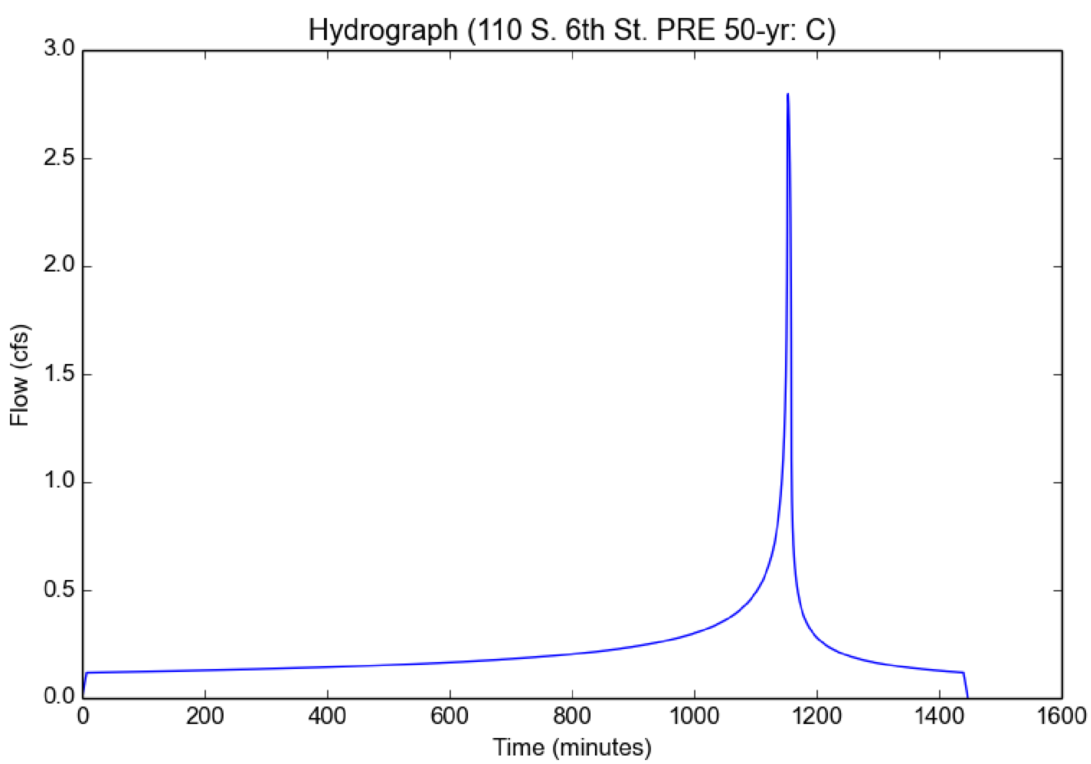
File location: P:/C/CPRKVALL0001/0600INFO/0670Reports/LID & Hydrology Report/Hydrocalc/Pre-development/110 S. 6th St. PRE Report.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	110 S. 6th St. PRE 50-yr
Subarea ID	C
Area (ac)	1.02
Flow Path Length (ft)	500.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	6.25
Percent Impervious	0.91
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	6.25
Peak Intensity (in/hr)	3.1835
Undeveloped Runoff Coefficient (Cu)	0.4694
Developed Runoff Coefficient (Cd)	0.8612
Time of Concentration (min)	7.0
Clear Peak Flow Rate (cfs)	2.7966
Burned Peak Flow Rate (cfs)	2.7966
24-Hr Clear Runoff Volume (ac-ft)	0.4374
24-Hr Clear Runoff Volume (cu-ft)	19054.5998



Peak Flow Hydrologic Analysis

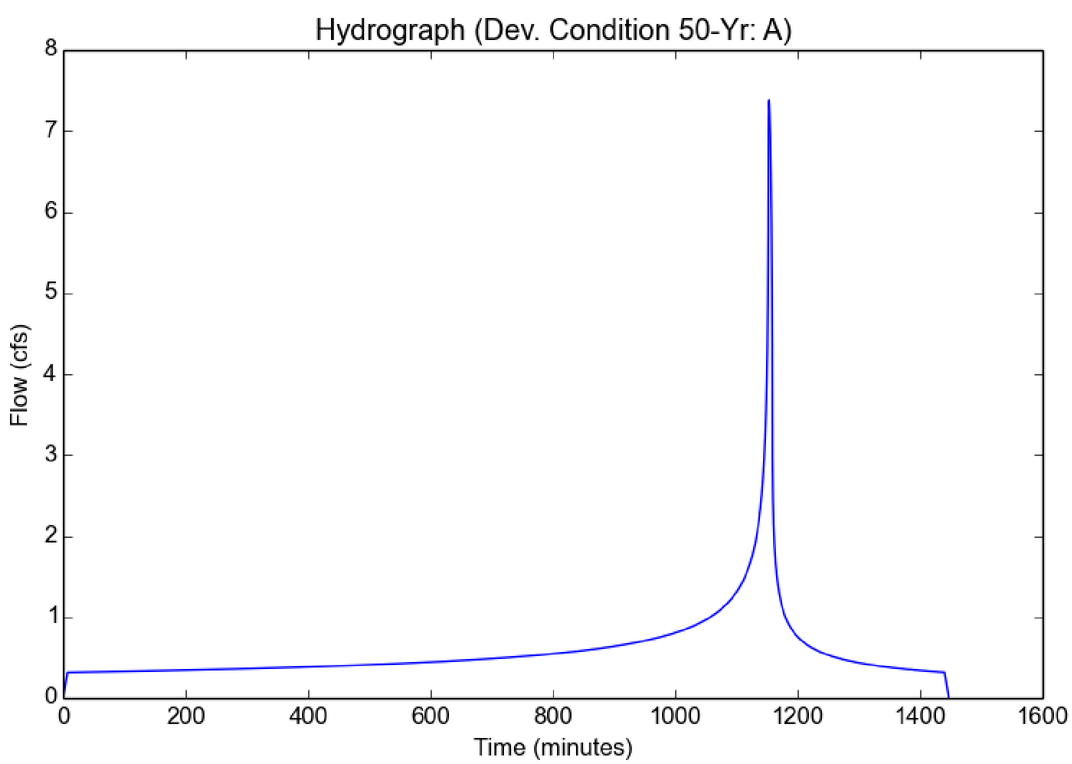
File location: C:/Users/Axro/Downloads/Subarea A Dev. Condition 50-Yr - A.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Dev. Condition 50-Yr
Subarea ID	A
Area (ac)	2.64
Flow Path Length (ft)	370.0
Flow Path Slope (vft/hft)	0.004
50-yr Rainfall Depth (in)	6.25
Percent Impervious	0.95
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	6.25
Peak Intensity (in/hr)	3.1835
Undeveloped Runoff Coefficient (Cu)	0.4694
Developed Runoff Coefficient (Cd)	0.8785
Time of Concentration (min)	7.0
Clear Peak Flow Rate (cfs)	7.383
Burned Peak Flow Rate (cfs)	7.383
24-Hr Clear Runoff Volume (ac-ft)	1.1744
24-Hr Clear Runoff Volume (cu-ft)	51158.7855



Peak Flow Hydrologic Analysis

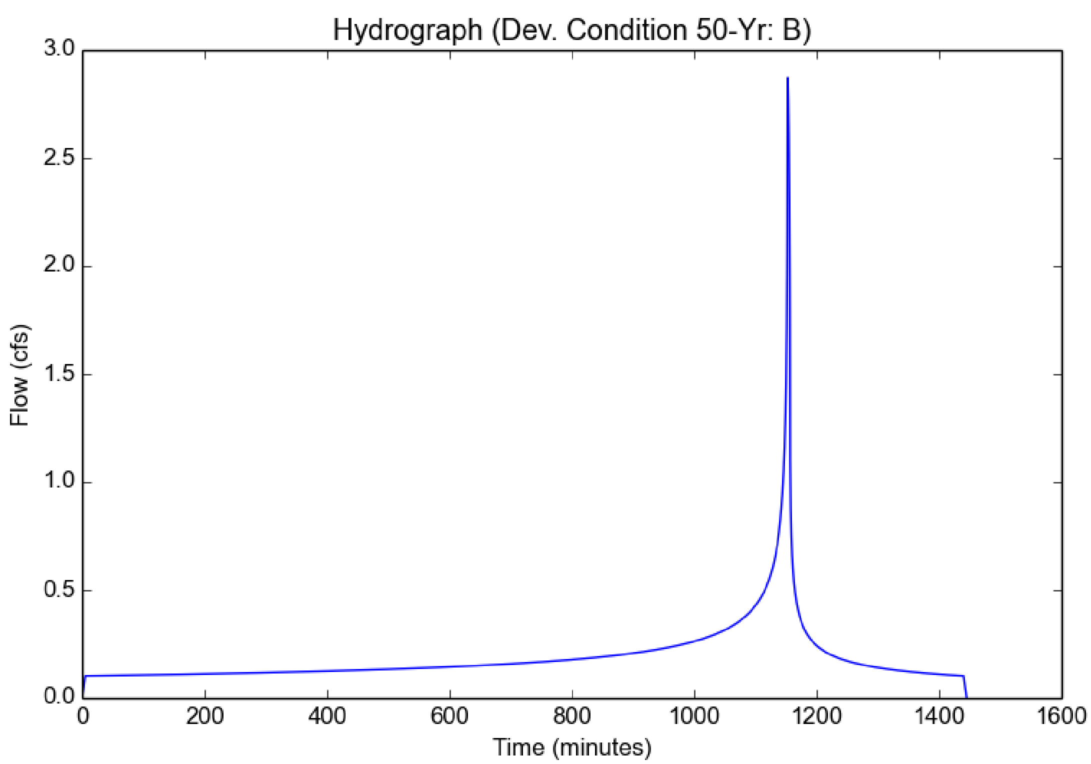
File location: C:/Users/Axro/Downloads/Subarea B Dev. Condition 50-Yr - A.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Dev. Condition 50-Yr
Subarea ID	B
Area (ac)	0.89
Flow Path Length (ft)	187.0
Flow Path Slope (vft/hft)	0.004
50-yr Rainfall Depth (in)	6.25
Percent Impervious	0.91
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	6.25
Peak Intensity (in/hr)	3.7289
Undeveloped Runoff Coefficient (Cu)	0.5193
Developed Runoff Coefficient (Cd)	0.8657
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.8732
Burned Peak Flow Rate (cfs)	2.8732
24-Hr Clear Runoff Volume (ac-ft)	0.3817
24-Hr Clear Runoff Volume (cu-ft)	16628.1051



Peak Flow Hydrologic Analysis

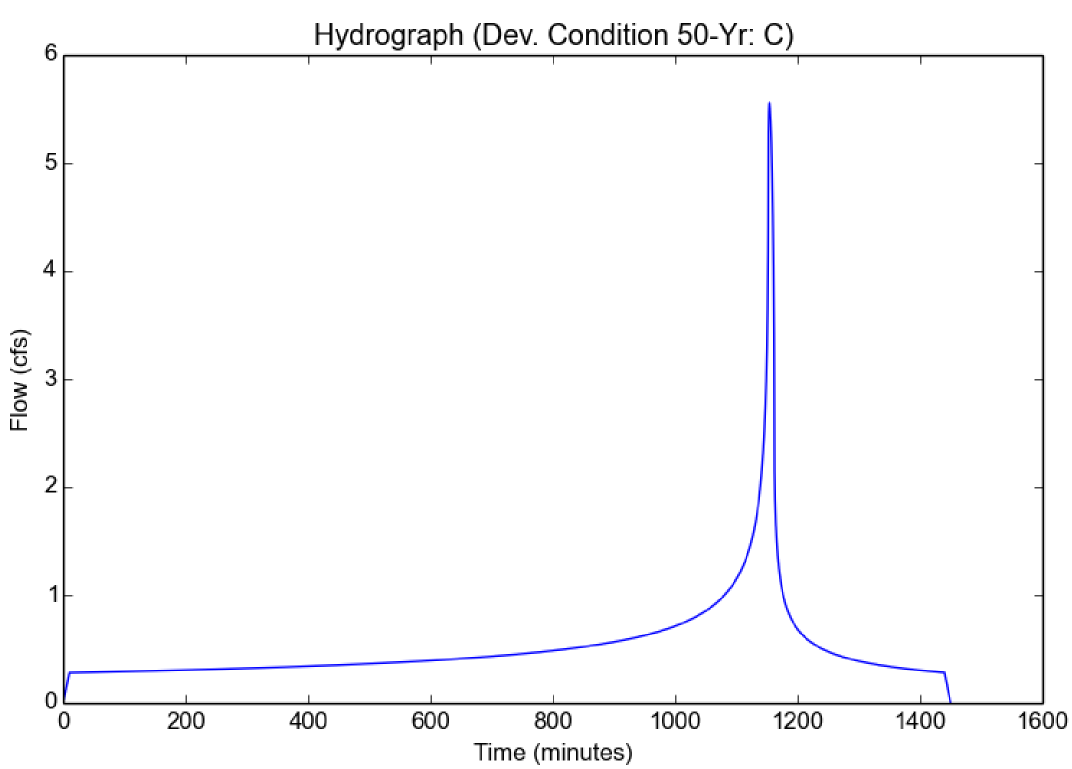
File location: C:/Users/Axro/Downloads/Subarea C Dev. Condition 50-Yr - A.pdf
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Dev. Condition 50-Yr
Subarea ID	C
Area (ac)	2.37
Flow Path Length (ft)	680.0
Flow Path Slope (vft/hft)	0.004
50-yr Rainfall Depth (in)	6.25
Percent Impervious	0.94
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	6.25
Peak Intensity (in/hr)	2.6921
Undeveloped Runoff Coefficient (Cu)	0.4178
Developed Runoff Coefficient (Cd)	0.8711
Time of Concentration (min)	10.0
Clear Peak Flow Rate (cfs)	5.5577
Burned Peak Flow Rate (cfs)	5.5577
24-Hr Clear Runoff Volume (ac-ft)	1.0447
24-Hr Clear Runoff Volume (cu-ft)	45509.1129



POST-DEVELOPED --- SUBAREA 'D' --- 50-YR DESIGN STORM

Peak Flow Hydrologic Analysis

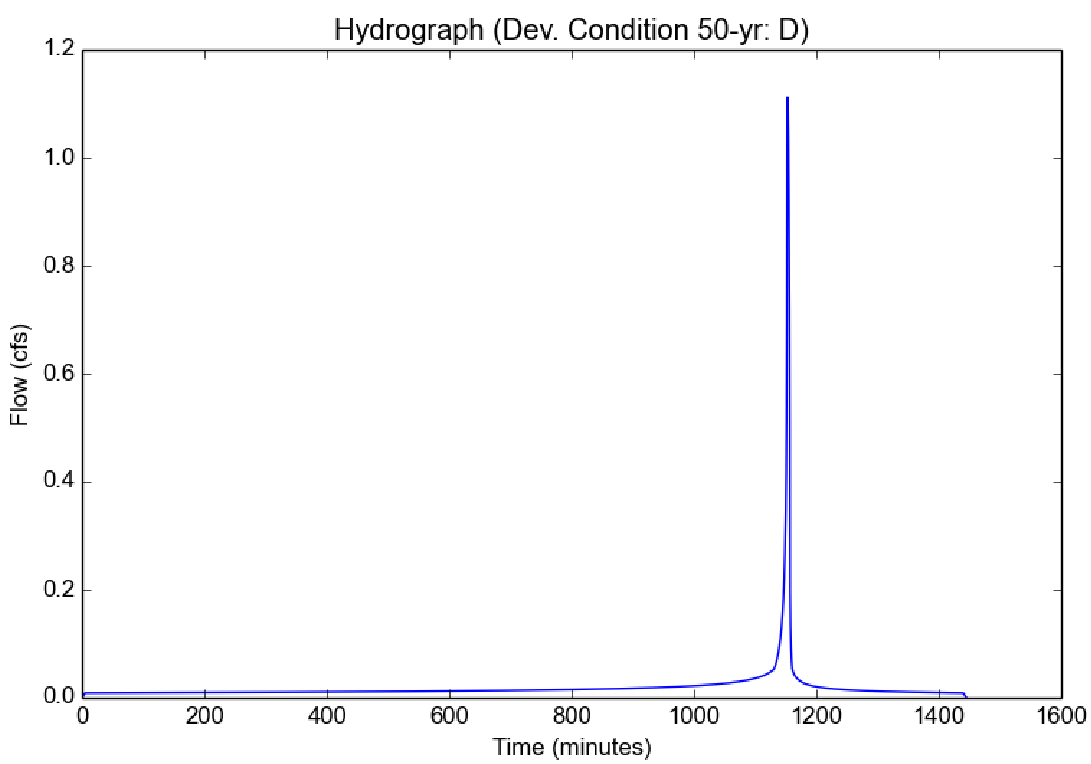
File location: //deainc.com/files/PROJECT/C/CPRKVALL0001/0600INFO/0670Reports/LID & Hydrology Report/Replacement PDFs/Dev. Condition 50-yr
Version: HydroCalc 1.0.3

Input Parameters

Project Name	Dev. Condition 50-yr
Subarea ID	D
Area (ac)	0.57
Flow Path Length (ft)	28.0
Flow Path Slope (vft/hft)	0.15
50-yr Rainfall Depth (in)	6.25
Percent Impervious	0.01
Soil Type	3
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	6.25
Peak Intensity (in/hr)	3.7289
Undeveloped Runoff Coefficient (Cu)	0.5193
Developed Runoff Coefficient (Cd)	0.5231
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	1.1119
Burned Peak Flow Rate (cfs)	1.1119
24-Hr Clear Runoff Volume (ac-ft)	0.0395
24-Hr Clear Runoff Volume (cu-ft)	1719.0301



Proportion Impervious Data

Code	Land Use Description	% Impervious
1111	High-Density Single Family Residential	42
1112	Low-Density Single Family Residential	21
1121	Mixed Multi-Family Residential	74
1122	Duplexes, Triplexes and 2-or 3-Unit Condominiums and Townhouses	55
1123	Low-Rise Apartments, Condominiums, and Townhouses	86
1124	Medium-Rise Apartments and Condominiums	86
1125	High-Rise Apartments and Condominiums	90
1131	Trailer Parks and Mobile Home Courts, High-Density	91
1132	Mobile Home Courts and Subdivisions, Low-Density	42
1140	Mixed Residential	59
1151	Rural Residential, High-Density	15
1152	Rural Residential, Low-Density	10
1211	Low- and Medium-Rise Major Office Use	91
1212	High-Rise Major Office Use	91
1213	Skyscrapers	91
1221	Regional Shopping Center	95
1222	Retail Centers (Non-Strip With Contiguous Interconnected Off-Street	96
1223	Modern Strip Development	96
1224	Older Strip Development	97
1231	Commercial Storage	90
1232	Commercial Recreation	90
1233	Hotels and Motels	96
1234	Attended Pay Public Parking Facilities	91
1241	Government Offices	91
1242	Police and Sheriff Stations	91
1243	Fire Stations	91
1244	Major Medical Health Care Facilities	74
1245	Religious Facilities	82
1246	Other Public Facilities	91
1247	Non-Attended Public Parking Facilities	91
1251	Correctional Facilities	91
1252	Special Care Facilities	74
1253	Other Special Use Facilities	86
1261	Pre-Schools/Day Care Centers	68
1262	Elementary Schools	82
1263	Junior or Intermediate High Schools	82
1264	Senior High Schools	82
1265	Colleges and Universities	47
1266	Trade Schools and Professional Training Facilities	91
1271	Base (Built-up Area)	65
1271.01	Base High-Density Single Family Residential	42
1271.02	Base Duplexes, Triplexes and 2-or 3-Unit Condominiums and T	55

Code	Land Use Description	% Impervious
1271.03	Base Government Offices	91
1271.04	Base Fire Stations	91
1271.05	Base Non-Attended Public Parking Facilities	91
1271.06	Base Air Field	45
1271.07	Base Petroleum Refining and Processing	91
1271.08	Base Mineral Extraction - Oil and Gas	10
1271.09	Base Harbor Facilities	91
1271.10	Base Navigation Aids	47
1271.11	Base Developed Local Parks and Recreation	10
1271.12	Base Vacant Undifferentiated	1
1272	Vacant Area	2
1273	Air Field	45
1274	Former Base (Built-up Area)	65
1275	Former Base Vacant Area	2
1276	Former Base Air Field	91
1311	Manufacturing, Assembly, and Industrial Services	91
1312	Motion Picture and Television Studio Lots	82
1313	Packing Houses and Grain Elevators	96
1314	Research and Development	91
1321	Manufacturing	91
1322	Petroleum Refining and Processing	91
1323	Open Storage	66
1324	Major Metal Processing	91
1325	Chemical Processing	91
1331	Mineral Extraction - Other Than Oil and Gas	10
1332	Mineral Extraction - Oil and Gas	10
1340	Wholesaling and Warehousing	91
1411	Airports	91
1411.01	Airstrip	10
1412	Railroads	15
1412.01	Railroads-Attended Pay Public Parking Facilities	91
1412.02	Railroads-Non-Attended Public Parking Facilities	91
1412.03	Railroads-Manufacturing, Assembly, and Industrial Services	91
1412.04	Railroads-Petroleum Refining and Processing	91
1412.05	Railroads-Open Storage	66
1412.06	Railroads-Truck Terminals	91
1413	Freeways and Major Roads	91
1414	Park-and-Ride Lots	91
1415	Bus Terminals and Yards	91
1416	Truck Terminals	91
1417	Harbor Facilities	91
1418	Navigation Aids	47
1420	Communication Facilities	82
1420.01	Communication Facilities-Antenna	2

Code	Land Use Description	% Impervious
1431	Electrical Power Facilities	47
1431.01	Electrical Power Facilities-Powerlines (Urban)	2
1431.02	Electrical Power Facilities-Powerlines (Rural)	1
1432	Solid Waste Disposal Facilities	15
1433	Liquid Waste Disposal Facilities	96
1434	Water Storage Facilities	91
1435	Natural Gas and Petroleum Facilities	91
1435.01	Natural Gas and Petroleum Facilities-Manufacturing, Assembly, and In	91
1435.02	Natural Gas and Petroleum Facilities-Petroleum Refining and Processing	91
1435.03	Natural Gas and Petroleum Facilities-Mineral Extraction – Oil and Gas	10
1435.04	Natural Gas and Petroleum Facilities-Vacant Undifferentiated	1
1436	Water Transfer Facilities	96
1437	Improved Flood Waterways and Structures	100
1440	Maintenance Yards	91
1450	Mixed Transportation	90
1460	Mixed Transportation and Utility	91
1460.01	Mixed Utility and Transportation-Improved Flood Waterways and Structures	100
1460.02	Mixed Utility and Transportation-Railroads	15
1460.03	Mixed Utility and Transportation-Freeways and Major Roads	91
1500	Mixed Commercial and Industrial	91
1600	Mixed Urban	89
1700	Under Construction (Use appropriate value)	91
1810	Golf Courses	3
1821	Developed Local Parks and Recreation	10
1822	Undeveloped Local Parks and Recreation	2
1831	Developed Regional Parks and Recreation	2
1832	Undeveloped Regional Parks and Recreation	1
1840	Cemeteries	10
1850	Wildlife Preserves and Sanctuaries	2
1850.01	Wildlife-Commercial Recreation	90
1850.02	Wildlife-Other Special Use Facilities	86
1850.03	Wildlife-Developed Local Parks and Recreation	10
1860	Specimen Gardens and Arboreta	15
1870	Beach Parks	10
1880	Other Open Space and Recreation	10
2110	Irrigated Cropland and Improved Pasture Land	2
2120	Non-Irrigated Cropland and Improved Pasture Land	2
2200	Orchards and Vineyards	2
2300	Nurseries	15
2400	Dairy, Intensive Livestock, and Associated Facilities	42
2500	Poultry Operations	62
2600	Other Agriculture	42
2700	Horse Ranches	42

Code	Land Use Description	% Impervious
3100	Vacant Undifferentiated	1
3200	Abandoned Orchards and Vineyards	2
3300	Vacant With Limited Improvements (Use appropriate value)	42
3400	Beaches (Vacant)	1
4100	Water, Undifferentiated	100
4200	Harbor Water Facilities	100
4300	Marina Water Facilities	100
4400	Water Within a Military Installation	100

APPENDIX D: HYDRAULIC ANALYSIS

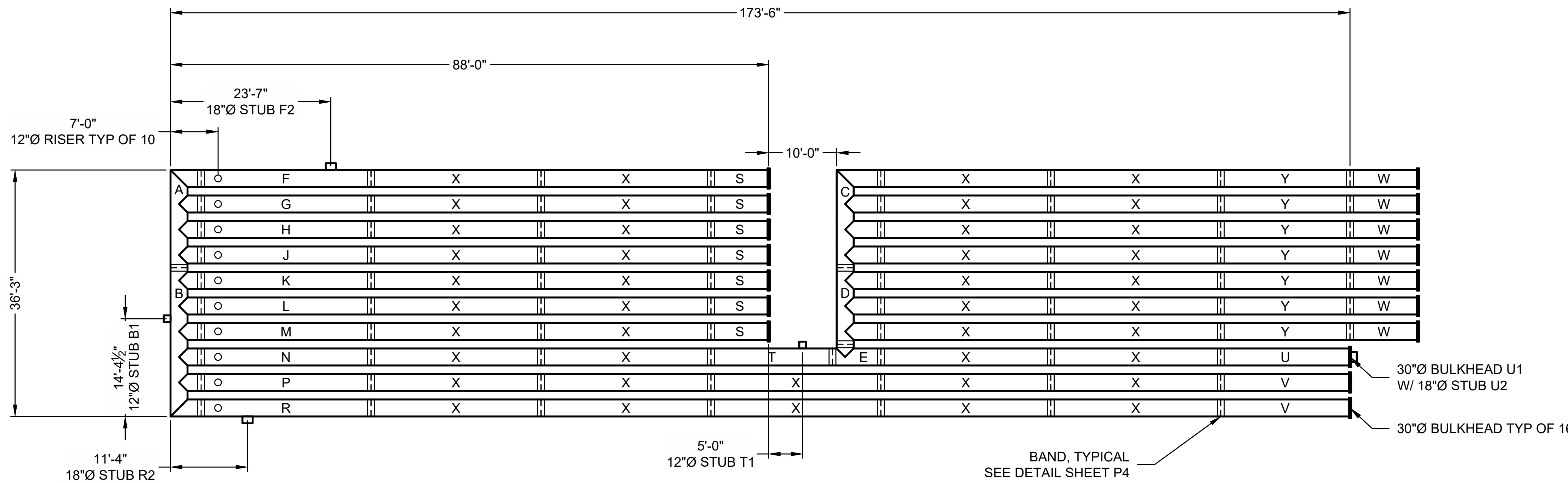
- Basin A & B Details
- Storm Drain Sizing
- Catch Basin for Area A
- Catch Basin for Area B
- Catch Basin for Area C

STUB INFORMATION

PIECE	STUB INVERT	SYSTEM INVERT
12"Ø STUB B1	292.19	292.19
18"Ø STUB F2	292.19	292.19
18"Ø STUB R2	292.19	292.19
12"Ø STUB T1	293.69	292.19
18"Ø STUB U2	292.18	292.19

RISER INFORMATION

PIECE	RIM ELEV.	SYSTEM INVERT
12"Ø RISER F1	TBD	292.19
12"Ø RISER G1	TBD	292.19
12"Ø RISER H1	TBD	292.19
12"Ø RISER J1	TBD	292.19
12"Ø RISER K1	TBD	292.19
12"Ø RISER L1	TBD	292.19
12"Ø RISER M1	TBD	292.19
12"Ø RISER N1	TBD	292.19
12"Ø RISER P1	TBD	292.19
12"Ø RISER R1	TBD	292.19



ASSEMBLY

SCALE: 1" = 20'
LOADING: H20
PIPE INV. = 219.19'±

NOTES

- BULKHEADS SHALL BE CONSTRUCTED USING 12 GAGE OR HEAVIER MATERIAL, WITH BOTH THE WATER AND SOIL SIDE FINAL COATINGS MATCHING THE SPECIFIED CMP COATING. BULKHEAD PLATES MUST BE FULLY WELDED TO THE CONNECTING PIPE. THE DESIGN OF BULKHEADS SHALL ADHERE TO CHAPTER 8 OF THE NCSPA CSP DESIGN MANUAL, MEETING THE HEIGHT OF COVER DESIGN REQUIREMENTS WITH APPROPRIATE REINFORCEMENTS OR A MINIMUM REQUIRED PLATE THICKNESS. ADDITIONALLY, REINFORCING MEMBERS SHALL BE POST-COATED WITH ZINC RICH PAINT IN ACCORDANCE WITH AASHTO M36 FOR GALVANIZED AND ALUMINUM CMP SYSTEMS, OR AASHTO M245 FOR POLYMER CMP SYSTEMS.
- ALL FITTINGS SHALL BE STRUCTURALLY CHECKED FOR REINFORCEMENTS PER ASTM A998 AND PROVIDED TO THE EOR FOR APPROVAL UPON REQUEST.
- CONNECTING BANDS FOR DETENTION SYSTEMS SHALL BE HUGGER TYPE OR FULLY CORRUGATED WITH APPROPRIATE BOLTED CONNECTIONS THAT CAN BE TORQUED TO 35 FOOT POUNDS. BANDS SHALL MATCH THE SPECIFIED CMP COATING AND MEET THE REQUIREMENTS OF AASHTO M 36.
- ALL METALLIC COATINGS AFFECTED BY MANUFACTURING FABRICATION SHALL BE REPAIRED PER AASHTO M 36 SECTION 11 REQUIREMENTS (E.G. ZINC-RICH PAINT ON ALL WELDS). IF POLYMER COATINGS ARE USED THE REPAIR OF DAMAGED COATINGS WILL BE IN CONFORMANCE WITH AASHTO M 245 SECTION 11 REQUIREMENTS.
- ACCESS LADDERS SHALL BE ATTACHED BY THE MANUFACTURER PRIOR TO DELIVERY, NOT INSTALLED ON THE JOBSITE.

THE UNDERSIGNED HEREBY APPROVES THE ATTACHED (5) PAGES INCLUDING THE FOLLOWING:

- **PIPE STORAGE = 8,615 CF**
- **MAINLINE PIPE GAGE = 16**
- **WALL TYPE = SOLID**
- **DIAMETER = 30"**
- **FINISH = ALT2**
- **CORRUGATION = 2-2/3x1/2**

CUSTOMER ______________________ DATE ____________

MARK	DATE	REVISION DESCRIPTION	BY

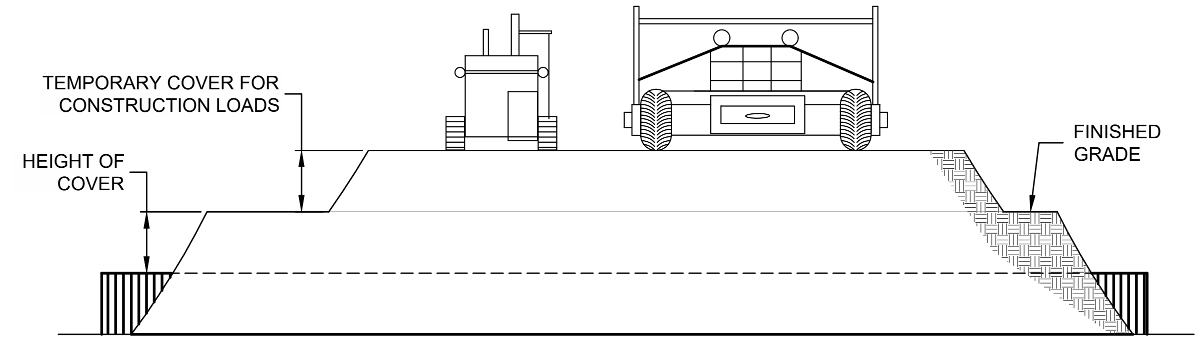
CONTECH® ENGINEERED SOLUTIONS LLC
A QUIKRETE® COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH® CMP DETENTION SYSTEMS

CONTECH **PROPOSAL** DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-010
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN A

PROJECT No.: 780630	SEQ. No.: 010	DATE: 6/19/25
DESIGNED: RLH		DRAWN: RLH
CHECKED:		APPROVED:
SHEET NO.: P1 OF 5		



CONSTRUCTION LOADS

FOR TEMPORARY CONSTRUCTION VEHICLE LOADS, AN EXTRA AMOUNT OF COMPACTED COVER MAY BE REQUIRED OVER THE TOP OF THE PIPE. THE HEIGHT-OF-COVER SHALL MEET THE MINIMUM REQUIREMENTS SHOWN IN THE TABLE BELOW. THE USE OF HEAVY CONSTRUCTION EQUIPMENT NECESSITATES GREATER PROTECTION FOR THE PIPE THAN FINISHED GRADE COVER MINIMUMS FOR NORMAL HIGHWAY TRAFFIC.

PIPE SPAN, INCHES	AXLE LOADS (kips)			
	18-50	50-75	75-110	110-150
	MINIMUM COVER (FT)			
12-42	2.0	2.5	3.0	3.0
48-72	3.0	3.0	3.5	4.0
78-120	3.0	3.5	4.0	4.0
126-144	3.5	4.0	4.5	4.5

*MINIMUM COVER MAY VARY, DEPENDING ON LOCAL CONDITIONS. THE CONTRACTOR MUST PROVIDE THE ADDITIONAL COVER REQUIRED TO AVOID DAMAGE TO THE PIPE. MINIMUM COVER IS MEASURED FROM THE TOP OF THE PIPE TO THE TOP OF THE MAINTAINED CONSTRUCTION ROADWAY SURFACE.

CONSTRUCTION LOADING DIAGRAM

NOT TO SCALE

SPECIFICATION FOR CORRUGATED STEEL PIPE-ALUMINIZED TYPE 2 STEEL

SCOPE

THIS SPECIFICATION COVERS THE MANUFACTURE AND INSTALLATION OF THE CORRUGATED STEEL PIPE (CSP) DETAILED IN THE PROJECT PLANS.

MATERIAL

THE ALUMINIZED TYPE 2 STEEL COILS SHALL CONFORM TO THE APPLICABLE REQUIREMENTS OF AASHTO M274 OR ASTM A929.

PIPE

THE CSP SHALL BE MANUFACTURED IN ACCORDANCE WITH THE APPLICABLE REQUIREMENTS OF AASHTO M36 OR ASTM A760. THE PIPE SIZES, GAGES AND CORRUGATIONS SHALL BE AS SHOWN ON THE PROJECT PLANS.

ALL FABRICATION OF THE PRODUCT SHALL OCCUR WITHIN THE UNITED STATES.

HANDLING AND ASSEMBLY

SHALL BE IN ACCORDANCE WITH RECOMMENDATIONS OF THE NATIONAL CORRUGATED STEEL PIPE ASSOCIATION (NCSPA)

INSTALLATION

SHALL BE IN ACCORDANCE WITH AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, SECTION 26, DIVISION II OR ASTM A798 AND IN CONFORMANCE WITH THE PROJECT PLANS AND SPECIFICATIONS. IF THERE ARE ANY INCONSISTENCIES OR CONFLICTS THE CONTRACTOR SHOULD DISCUSS AND RESOLVE WITH THE SITE ENGINEER.

IT IS ALWAYS THE RESPONSIBILITY OF THE CONTRACTOR TO FOLLOW OSHA GUIDELINES FOR SAFE PRACTICES.

ANTI-FLOTATION PROVISIONS DUE TO HIGH GROUNDWATER OR OTHER FLOTATION CONCERNS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.

MATERIAL SPECIFICATION

NOT TO SCALE

\\QUIKRETE.NET\CONTECH\MERLIN\PROJECT\ACTIVE\780600\780630\780630-10-CMP DETENTION\DRAWINGS\780630-010-CMP CONFAB.DWG 5/22/2025 2:22 PM

MARK	DATE	REVISION DESCRIPTION	BY

CONTECH ENGINEERED SOLUTIONS LLC
A QUIKRETE COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH CMP DETENTION SYSTEMS

CONTECH PROPOSAL DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-010
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN A

PROJECT No.: 780630	SEQ. No.: 010	DATE: 5/21/2025
DESIGNED: RLH		DRAWN: RLH
CHECKED:		APPROVED:
SHEET NO.: P2	OF	5

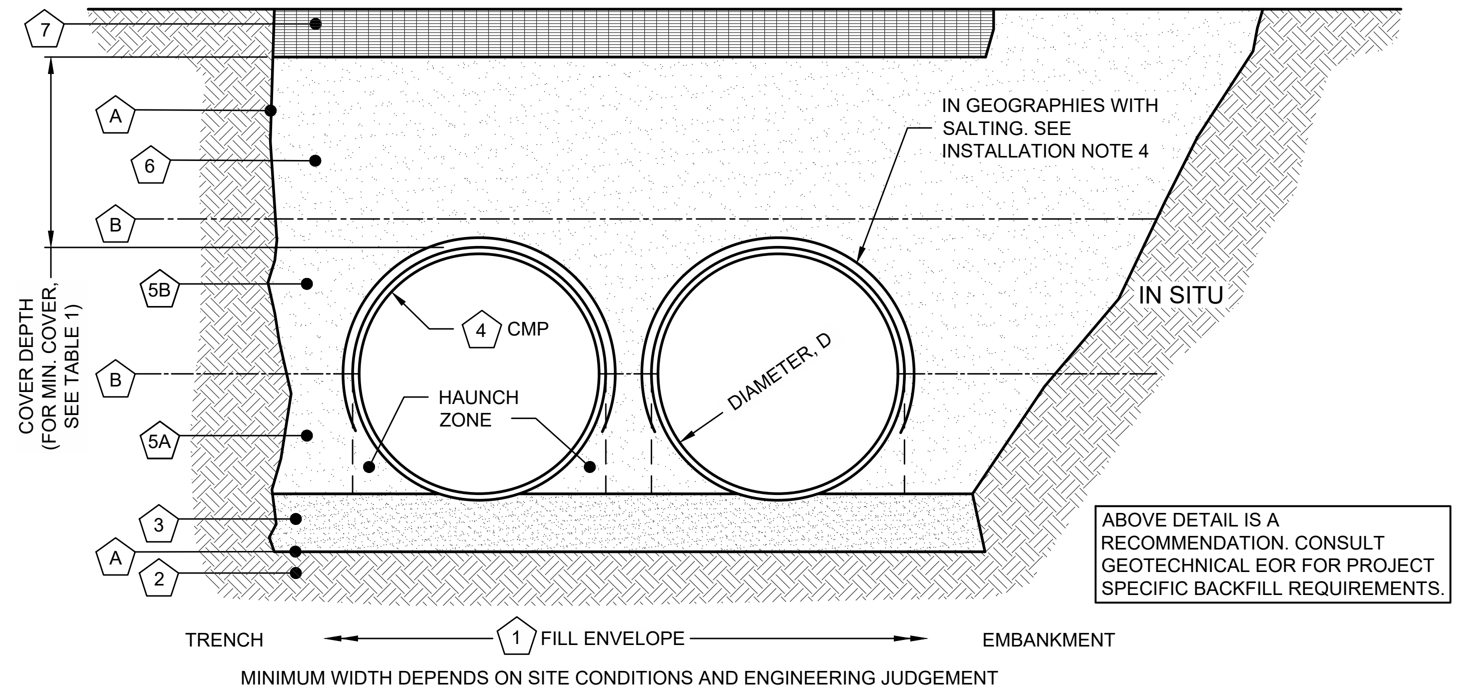


TABLE 1:

DIAMETER, D	MIN. COVER	CORR. PROFILE
6"-10"	12"	1 1/2" x 1/4"
12"-48"	12"	2 2/3" x 1/2"
>48"-96"	12"	3" x 1", 5" x 1"
>96"	D/8	3" x 1", 5" x 1"

- STRUCTURAL BACKFILL MUST EXTEND TO LIMITS OF THE TABLE
- TOTAL HEIGHT OF COMPACTED COVER FOR CONVENTIONAL HIGHWAY LOADS IS MEASURED FROM TOP OF PIPE TO BOTTOM OF FLEXIBLE PAVEMENT OR TOP OF RIGID PAVEMENT
- ULTRAFLO ALSO AVAILABLE FOR SIZES 18" - 120" WITH 3/4"x 3/4"x 7 1/2" CORRUGATION

INSTALLATION NOTES

1. WHEN PLACING THE FIRST LIFTS OF BACKFILL IT IS IMPORTANT TO MAKE SURE THAT THE BACKFILL IS PROPERLY COMPACTED UNDER AND AROUND THE PIPE HAUNCHES.
2. OTHER ALTERNATE BACKFILL MATERIAL MAY BE ALLOWED DEPENDING ON SITE SPECIFIC CONDITIONS, AS APPROVED BY SITE ENGINEER.
3. BACKFILL USING CONTROLLED LOW-STRENGTH MATERIAL (CLSM, "FLASH FILL" OR "FLOWABLE FILL") MAY BE USED WHEN THE SPACING BETWEEN THE PIPES WILL NOT ALLOW FOR PLACEMENT AND ADEQUATE COMPACTION OF THE BACKFILL. CONTACT CONTECH FOR FURTHER EVALUATION.
4. IF SALTING AGENTS FOR SNOW AND ICE REMOVAL ARE USED ON OR NEAR THE PROJECT, A GEOMEMBRANE BARRIER IS RECOMMENDED OVER THE UPPER HALF OF THE PIPE. THE GEOMEMBRANE LINER IS INTENDED TO HELP PROTECT THE SYSTEM FROM THE POTENTIAL ADVERSE EFFECTS THAT MAY RESULT FROM A CHANGE IN THE SURROUNDING ENVIRONMENT OVER A PERIOD OF TIME. PLEASE REFER TO THE CORRUGATED METAL PIPE DETENTION DESIGN GUIDE FOR ADDITIONAL INFORMATION.

TABLE 2:

CMP DETENTION AND CMP DRAINAGE STANDARD BACKFILL SPECIFICATIONS

	MATERIAL LOCATION	MATERIAL SPECIFICATION	DESCRIPTION	
1	FILL ENVELOPE WIDTH	PER ENGINEER OF RECORD	MINIMUM TRENCH WIDTH MUST ALLOW ROOM FOR PROPER COMPACTION OF HAUNCH MATERIALS UNDER THE PIPE. THE SUGGESTED MINIMUM TRENCH WIDTH, OR EOR RECOMMENDATION: PIPE ≤ 12": D + 16" PIPE > 12": 1.5D + 12"	MINIMUM EMBANKMENT WIDTH (IN FEET) FOR INITIAL FILL ENVELOPE: PIPE < 24": 3.0D PIPE 24" - 144": D + 4'0" PIPE > 144": D + 10'0"
2	FOUNDATION	**AASHTO 26.5.2** OR PER ENGINEER OF RECORD	PRIOR TO PLACING THE BEDDING, THE FOUNDATION MUST BE CONSTRUCTED TO A UNIFORM AND STABLE GRADE. IN THE EVENT THAT UNSUITABLE FOUNDATION MATERIALS ARE ENCOUNTERED DURING EXCAVATION, THEY SHALL BE REMOVED AND FOUNDATION BROUGHT BACK TO GRADE WITH A FILL MATERIAL APPROVED BY THE ENGINEER OF RECORD.	
3	BEDDING	**AASHTO M 43: 3, 357, 4, 467, 5, 56, 57** (APPROVED REGIONAL EQUIVALENTS INCLUDE CA-7)	ENGINEER OF RECORD TO DETERMINE IF BEDDING IS REQUIRED. PIPE MAY BE PLACED ON THE TRENCH BOTTOM OF A RELATIVELY LOOSE, NATIVE SUITABLE WELL GRADED GRANULAR MATERIAL THAT IS ROUGHLY SHAPED TO FIT THE BOTTOM OF THE PIPE, 2" MIN DEPTH. THE BEDDING MATERIAL MAY BE SUITABLE FOUNDATION SOILS CONFORMING TO AASHTO SOIL CLASSIFICATIONS A1, A2, OR A3 WITH MAXIMUM PARTICLE SIZE OF 3" PER AASHTO 26.3.8.1	
4	CORRUGATED METAL PIPE			
5A	CRITICAL BACKFILL	**AASHTO M 145: A-1, A-2, A-3** *	HAUNCH ZONE MATERIAL SHALL BE HAND SHOVELED OR SHOVEL SLICED INTO PLACE TO ALLOW FOR PROPER COMPACTION WITHOUT SOFT SPOTS. BACKFILL SHALL BE PLACED IN 8" +/- LOOSE LIFTS AND COMPACTED TO 90% STANDARD PROCTOR PER AASHTO T 99. BACKFILL SHALL BE PLACED SUCH THAT THERE IS NO MORE THAN A THREE LIFT (24") DIFFERENTIAL BETWEEN ANY OF THE PIPES AT ANY TIME DURING THE BACKFILL PROCESS. THE BACKFILL SHOULD BE ADVANCED ALONG THE LENGTH OF THE SYSTEM TO AVOID DIFFERENTIAL LOADING. WELL GRADED GRANULAR MATERIAL WHICH MAY CONTAIN SMALL AMOUNTS OF SILT OR CLAY AND MAXIMUM PARTICLE SIZE OF 3" (PER AASHTO 26.3.8.1 AND 12.4-1.3).	
5B	BACKFILL	**AASHTO M 145: A-1, A-2, A-3**		
6	COVER MATERIAL	UP TO MIN. COVER - SEE 5A AND 5B ABOVE ABOVE MIN. COVER - PER ENGINEER OF RECORD	COVER MATERIAL MAY INCLUDE NON-BITUMINOUS, GRANULAR ROAD BASE MATERIAL WITHIN MIN COVER LIMITS	
7	RIGID OR FLEXIBLE PAVEMENT (IF APPLICABLE)	PER ENGINEER OF RECORD	FLEXIBLE PAVEMENT SHOULD NOT BE COUNTED AS PART OF THE FILL HEIGHT OVER THE CMP. FINAL BACKFILL MATERIAL SELECTION AND COMPACTION REQUIREMENTS SHALL FOLLOW THE PROJECT PLANS AND SPECIFICATIONS PER THE ENGINEER OF RECORD.	
A	OPTIONAL SIDE GEOTEXTILE	NONE	GEOTEXTILE LAYER IS RECOMMENDED ON SIDES OF EXCAVATION TO PREVENT SOIL MIGRATION.	
B	OPTIONAL GEOTEXTILE BETWEEN LAYERS	NONE	IF SOIL TYPES DIFFER AT ANY POINT ABOVE PIPE INVERT, A GEOTEXTILE LAYER IS RECOMMENDED TO BE PLACED BETWEEN THE LAYERS TO PREVENT SOIL MIGRATION.	

NOTES:
- FOR MULTIPLE BARREL INSTALLATIONS, THE RECOMMENDED STANDARD SPACING BETWEEN PARALLEL PIPE RUNS SHALL BE THE PIPE DIAMETER /2 BUT NO LESS THAN 12" FOR DIAMETERS <72". FOR 72" AND LARGER DIAMETERS, THE MINIMUM SPACING IS 36". CONTACT YOUR CONTECH REPRESENTATIVE FOR NONSTANDARD SPACING.

* APPROVED REGIONAL EQUIVALENTS FOR SECTION 5A INCLUDE CA-7, MIDOT 2G, 34G, OR 21AA STONE OR GRAVEL; #8; #57; MIDOT 6A, 2G, 3G, 34G.

MANUFACTURER RECOMMENDED BACKFILL

NOT TO SCALE

MARK	DATE	REVISION DESCRIPTION	BY

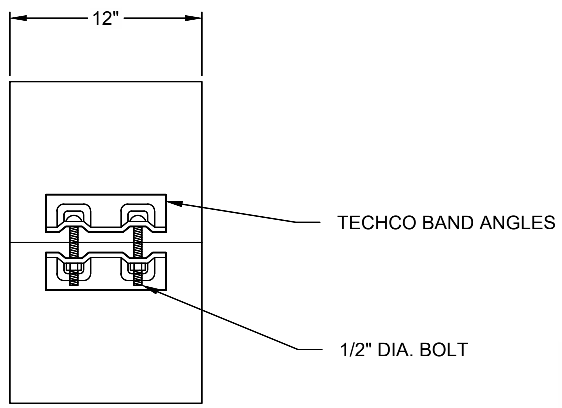
CONTECH ENGINEERED SOLUTIONS LLC
A QUIKRETE COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH CMP DETENTION SYSTEMS

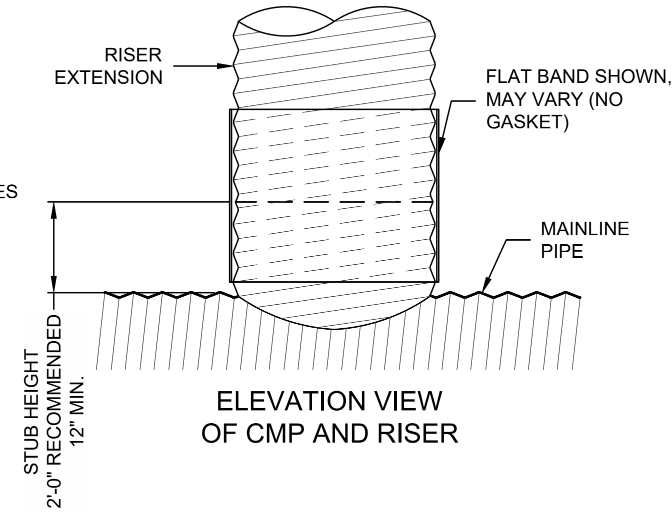
CONTECH PROPOSAL DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-010
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN A

PROJECT No.: 780630	SEQ. No.: 010	DATE: 5/21/2025
DESIGNED: RLH		DRAWN: RLH
CHECKED:		APPROVED:
SHEET NO.: P3	OF	5



CONNECTION DETAIL
7 1/2" TECHCO SHOWN - MAY VARY



ELEVATION VIEW
OF CMP AND RISER

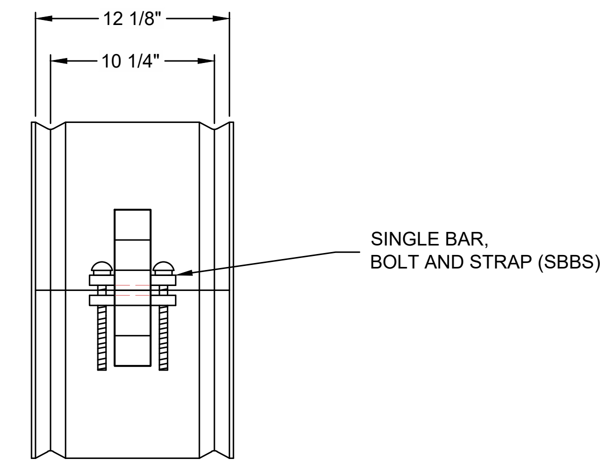
PLAIN END CMP RISER PIPE

GENERAL NOTES:

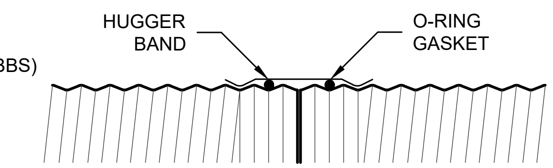
1. **DELIVERED BAND STYLE AND FASTENER TYPE MAY VARY BY FABRICATION PLANT.**
2. JOINT IS TO BE ASSEMBLED PER AASHTO BRIDGE CONSTRUCTION SPECIFICATION SEC 26.4.2.4.
3. BAND MATERIAL AND GAGE TO BE SAME AS RISER MATERIAL.
4. IF RISER HAS A HEIGHT OF COVER OF 10' OR MORE, USE A SLIP JOINT.
5. BANDS ARE NORMALLY FURNISHED AS FOLLOWS:
   - 12" THRU 48" 1-PIECE
   - 54" 2-PIECES
6. ALL RISER JOINT COMPONENTS WILL BE FIELD ASSEMBLED.
7. MANHOLE RISERS IN APPLICATIONS WHERE TRAFFIC LOADS ARE IMPOSED REQUIRE SPECIAL DESIGN CONSIDERATIONS.
8. DIMENSIONS SUBJECT TO MANUFACTURING TOLERANCES.

12" RISER BAND DETAIL

NOT TO SCALE



CONNECTION DETAIL
(SBBS)



O-RING GASKET

2 2/3"x1/2" RE-ROLLED END HEL-COR PIPE

GENERAL NOTES:

1. JOINT IS TO BE ASSEMBLED PER AASHTO BRIDGE CONSTRUCTION SPECIFICATION SEC 26.4.2.4.
2. BAND MATERIALS AND/OR COATING CAN VARY BY LOCATION. CONTACT YOUR CONTECH REPRESENTATIVE FOR AVAILABILITY.
3. BANDS ARE SHAPED TO MATCH THE PIPE-ARCH WHEN APPLICABLE.
4. BANDS ARE NORMALLY FURNISHED AS FOLLOWS:
   - 12" THRU 48" 1-PIECE
   - 54" THRU 96" 2-PIECES
   - 102" THRU 144" 3-PIECES
5. BAND FASTENERS ARE ATTACHED WITH SPOT WELDS, RIVETS OR HAND WELDS.
6. ALL CMP IS REROLLED TO HAVE ANNULAR END CORRUGATIONS OF 2 2/3"x1/2"
7. DIMENSIONS ARE SUBJECT TO MANUFACTURING TOLERANCES.
8. ORDER SHALL DESIGNATE GASKET OPTION, IF REQUIRED (SEE DETAILS ABOVE).

H-12 HUGGER BAND DETAIL

NOT TO SCALE

MARK	DATE	REVISION DESCRIPTION	BY

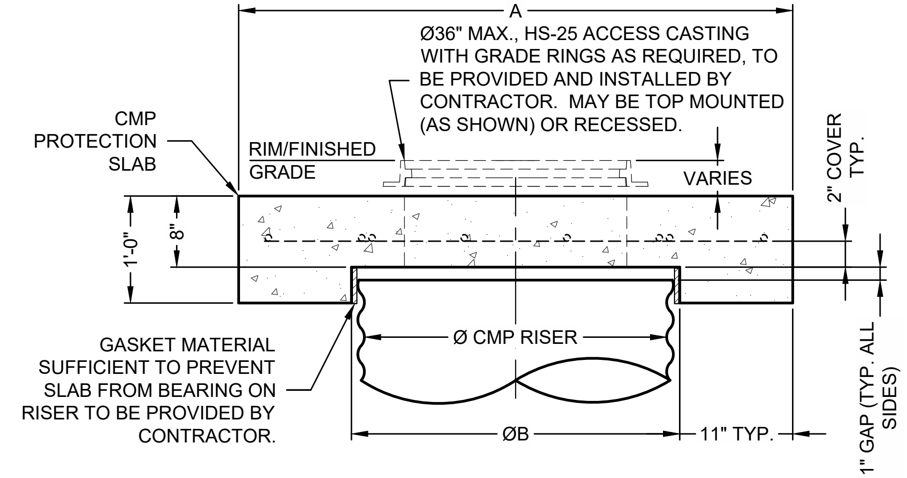
CONTECH® ENGINEERED SOLUTIONS LLC
A QUIKRETE® COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH® CMP DETENTION SYSTEMS

CONTECH
PROPOSAL
DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-010
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN A

PROJECT No.: 780630	SEQ. No.: 010	DATE: 5/21/2025
DESIGNED: RLH	DRAWN: RLH	
CHECKED:	APPROVED:	
SHEET NO.: P4 OF 5		



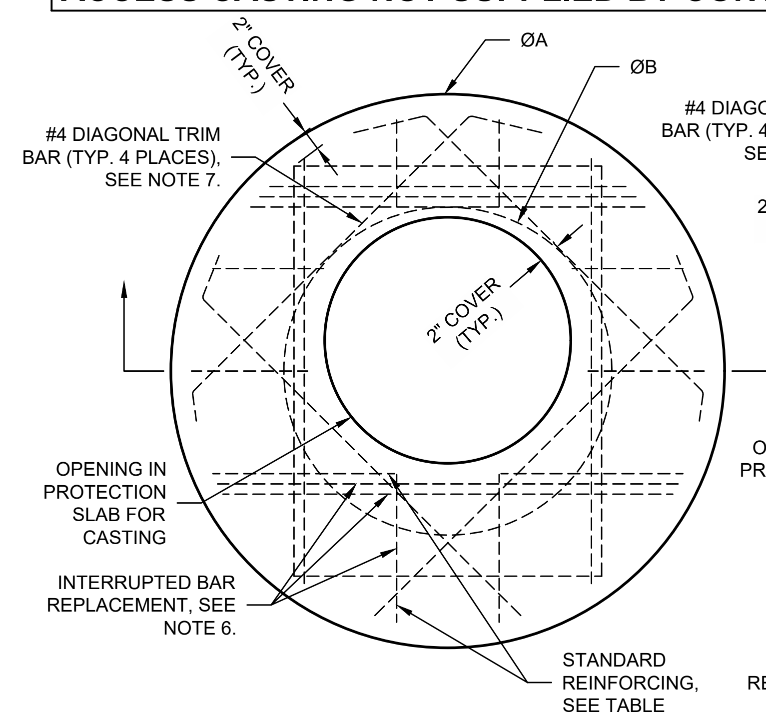
SECTION VIEW

ACCESS CASTING NOT SUPPLIED BY CONTECH

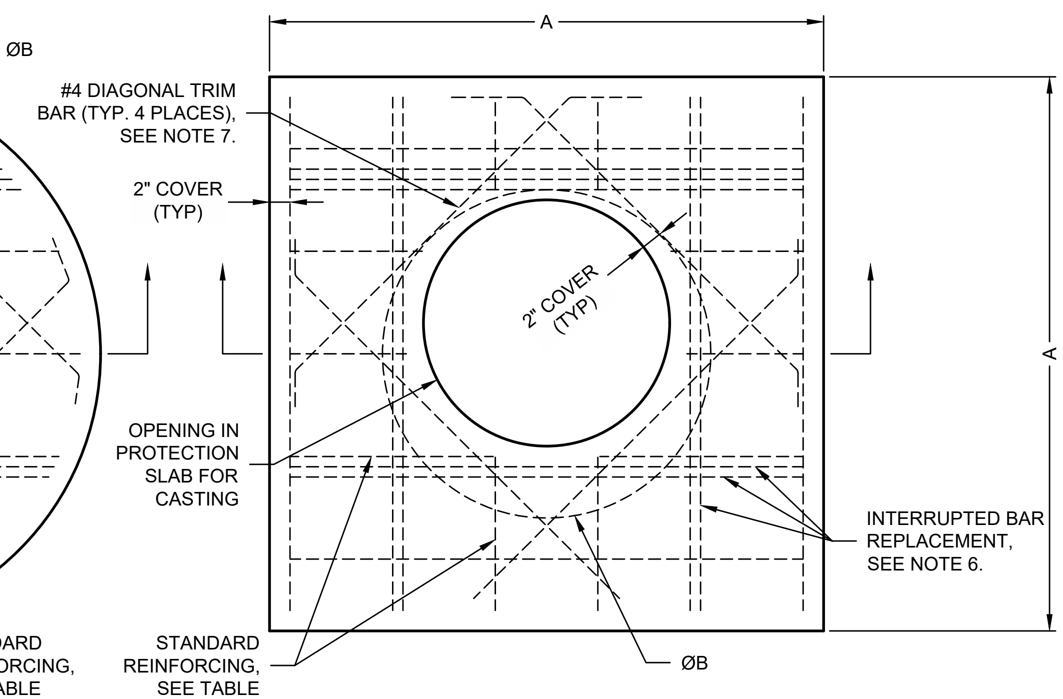
REINFORCING TABLE

Ø CMP RISER	A	Ø B	REINFORCING	**BEARING PRESSURE (PSF)
24"	Ø 4' 4'x4'	26"	#5 @ 10" OCEW #5 @ 10" OCEW	2,540 1,900
30"	Ø 4'-6" 4'-6" x 4'-6"	32"	#5 @ 10" OCEW #5 @ 9" OCEW	2,260 1,670
36"	Ø 5' 5' x 5'	38"	#5 @ 9" OCEW #5 @ 8" OCEW	2,060 1,500
42"	Ø 5'-6" 5'-6" x 5'-6"	44"	#5 @ 8" OCEW #5 @ 8" OCEW	1,490 1,370
48"	Ø 6' 6' x 6'	50"	#5 @ 7" OCEW #5 @ 7" OCEW	1,210 1,270

** ASSUMED SOIL BEARING CAPACITY



ROUND OPTION PLAN VIEW



SQUARE OPTION PLAN VIEW

NOTES:

1. DESIGN IN ACCORDANCE WITH AASHTO, 17th EDITION AND ACI 350.
2. DESIGN LOAD HS25.
3. EARTH COVER = 1' MAX.
4. CONCRETE STRENGTH = 4,000 psi
5. REINFORCING STEEL = ASTM A615, GRADE 60.
6. PROVIDE ADDITIONAL REINFORCING AROUND OPENINGS EQUAL TO THE BARS INTERRUPTED, HALF EACH SIDE. ADDITIONAL BARS TO BE IN THE SAME PLANE.
7. TRIM OPENING WITH DIAGONAL #4 BARS, EXTEND BARS A MINIMUM OF 12" BEYOND OPENING, BEND BARS AS REQUIRED TO MAINTAIN BAR COVER.
8. PROTECTION SLAB AND ALL MATERIALS TO BE PROVIDED AND INSTALLED BY CONTRACTOR.
9. DETAIL DESIGN BY DELTA ENGINEERS, ARCHITECTS AND LAND SURVEYORS, ENDWELL, NY.
10. THIS DETAIL REFLECTS CAST-IN-PLACE MANHOLE CAP PROTECTION SLAB. PRECAST MANHOLE CAP AVAILABLE IN SELECT MARKETS UNDER SEPARATE SUBMITTAL & DESIGN.

CAST-IN-PLACE MANHOLE CAP DETAIL

NOT TO SCALE

MARK	DATE	REVISION DESCRIPTION	BY

CONTECH ENGINEERED SOLUTIONS LLC
A QUIKRETE COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH CMP DETENTION SYSTEMS

CONTECH PROPOSAL DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-010
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN A

PROJECT No.: 780630	SEQ. No.: 010	DATE: 5/21/2025
DESIGNED: RLH		DRAWN: RLH
CHECKED:		APPROVED:
SHEET NO.: P5	OF 5	

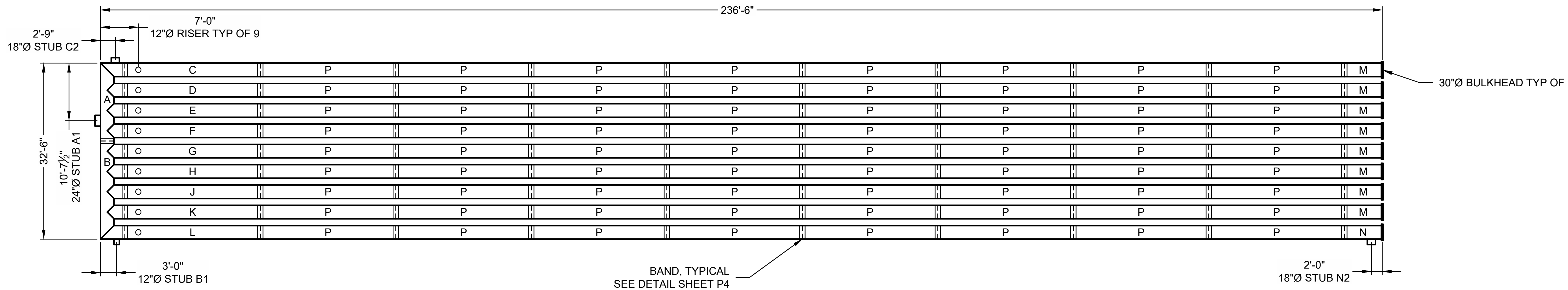
\\QUIKRETE.NET\CONTECH\MERLIN\PROJECT\ACTIVE\780600\780630\780630-10-CMP DETENTION\DRAWINGS\780630-010-CMP CONFAB.DWG 5/22/2025 2:22 PM

RISER INFORMATION

PIECE	RIM ELEV.	SYSTEM INVERT
12"Ø RISER C1	TBD	290.77
12"Ø RISER D1	TBD	290.77
12"Ø RISER E1	TBD	290.77
12"Ø RISER F1	TBD	290.77
12"Ø RISER G1	TBD	290.77
12"Ø RISER H1	TBD	290.77
12"Ø RISER J1	TBD	290.77
12"Ø RISER K1	TBD	290.77
12"Ø RISER L1	TBD	290.77

STUB INFORMATION

PIECE	STUB INVERT	SYSTEM INVERT
24"Ø STUB A1	290.77	290.77
12"Ø STUB B1	290.77	290.77
18"Ø STUB C2	290.77	290.77
18"Ø STUB N2	290.77	290.77



ASSEMBLY
SCALE: 1" = 20'
LOADING: H20
PIPE INV. = 290.77'±

NOTES

- BULKHEADS SHALL BE CONSTRUCTED USING 12 GAGE OR HEAVIER MATERIAL, WITH BOTH THE WATER AND SOIL SIDE FINAL COATINGS MATCHING THE SPECIFIED CMP COATING. BULKHEAD PLATES MUST BE FULLY WELDED TO THE CONNECTING PIPE. THE DESIGN OF BULKHEADS SHALL ADHERE TO CHAPTER 8 OF THE NCSPA CSP DESIGN MANUAL, MEETING THE HEIGHT OF COVER DESIGN REQUIREMENTS WITH APPROPRIATE REINFORCEMENTS OR A MINIMUM REQUIRED PLATE THICKNESS. ADDITIONALLY, REINFORCING MEMBERS SHALL BE POST-COATED WITH ZINC RICH PAINT IN ACCORDANCE WITH AASHTO M36 FOR GALVANIZED AND ALUMINUM CMP SYSTEMS, OR AASHTO M245 FOR POLYMER CMP SYSTEMS.
- ALL FITTINGS SHALL BE STRUCTURALLY CHECKED FOR REINFORCEMENTS PER ASTM A998 AND PROVIDED TO THE EOR FOR APPROVAL UPON REQUEST.
- CONNECTING BANDS FOR DETENTION SYSTEMS SHALL BE HUGGER TYPE OR FULLY CORRUGATED WITH APPROPRIATE BOLTED CONNECTIONS THAT CAN BE TORQUED TO 35 FOOT POUNDS. BANDS SHALL MATCH THE SPECIFIED CMP COATING AND MEET THE REQUIREMENTS OF AASHTO M 36.
- ALL METALLIC COATINGS AFFECTED BY MANUFACTURING FABRICATION SHALL BE REPAIRED PER AASHTO M 36 SECTION 11 REQUIREMENTS (E.G. ZINC-RICH PAINT ON ALL WELDS). IF POLYMER COATINGS ARE USED THE REPAIR OF DAMAGED COATINGS WILL BE IN CONFORMANCE WITH AASHTO M 245 SECTION 11 REQUIREMENTS.
- ACCESS LADDERS SHALL BE ATTACHED BY THE MANUFACTURER PRIOR TO DELIVERY, NOT INSTALLED ON THE JOBSITE.

THE UNDERSIGNED HEREBY APPROVES THE ATTACHED (5) PAGES INCLUDING THE FOLLOWING:

- **PIPE STORAGE = 10,498 CF**
- **MAINLINE PIPE GAGE = 16**
- **WALL TYPE = SOLID**
- **DIAMETER = 30"**
- **FINISH = ALT2**
- **CORRUGATION = 2 2/3x1/2**

CUSTOMER ____________________ DATE ____________

MARK	DATE	REVISION DESCRIPTION	BY

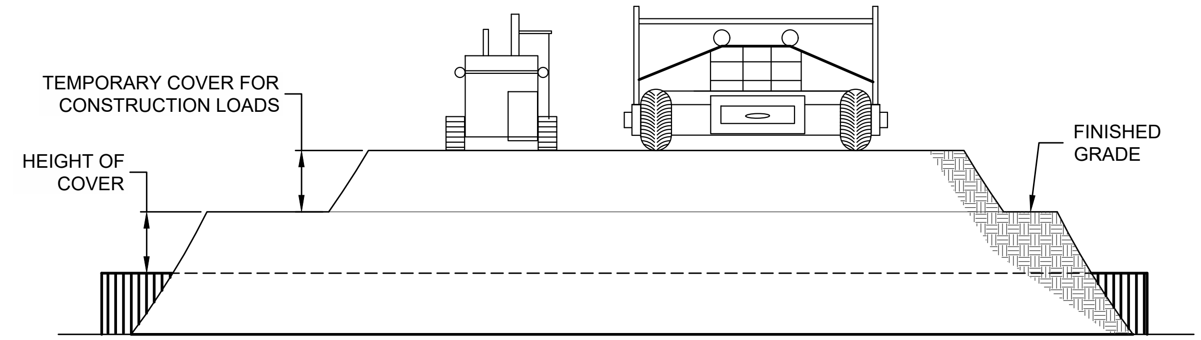
CONTECH® ENGINEERED SOLUTIONS LLC
A QUIKRETE® COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH® CMP DETENTION SYSTEMS
CONTECH **PROPOSAL** DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-015
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN B

PROJECT No.: 780630	SEQ. No.: 015	DATE: 5/21/25
DESIGNED: RLH		DRAWN: RLH
CHECKED:		APPROVED:
SHEET NO.: P1 OF 5		

\\QUIKRETE.NET\CONTECH\MERLIN\PROJECT\ACTIVE\780600\780630\780630-15-CMP DETENTION\DRAWINGS\780630-015-CMP CONFAB.DWG 5/21/2025 11:13 AM



CONSTRUCTION LOADS

FOR TEMPORARY CONSTRUCTION VEHICLE LOADS, AN EXTRA AMOUNT OF COMPACTED COVER MAY BE REQUIRED OVER THE TOP OF THE PIPE. THE HEIGHT-OF-COVER SHALL MEET THE MINIMUM REQUIREMENTS SHOWN IN THE TABLE BELOW. THE USE OF HEAVY CONSTRUCTION EQUIPMENT NECESSITATES GREATER PROTECTION FOR THE PIPE THAN FINISHED GRADE COVER MINIMUMS FOR NORMAL HIGHWAY TRAFFIC.

PIPE SPAN, INCHES	AXLE LOADS (kips)			
	18-50	50-75	75-110	110-150
	MINIMUM COVER (FT)			
12-42	2.0	2.5	3.0	3.0
48-72	3.0	3.0	3.5	4.0
78-120	3.0	3.5	4.0	4.0
126-144	3.5	4.0	4.5	4.5

*MINIMUM COVER MAY VARY, DEPENDING ON LOCAL CONDITIONS. THE CONTRACTOR MUST PROVIDE THE ADDITIONAL COVER REQUIRED TO AVOID DAMAGE TO THE PIPE. MINIMUM COVER IS MEASURED FROM THE TOP OF THE PIPE TO THE TOP OF THE MAINTAINED CONSTRUCTION ROADWAY SURFACE.

CONSTRUCTION LOADING DIAGRAM

NOT TO SCALE

SPECIFICATION FOR CORRUGATED STEEL PIPE-ALUMINIZED TYPE 2 STEEL

SCOPE

THIS SPECIFICATION COVERS THE MANUFACTURE AND INSTALLATION OF THE CORRUGATED STEEL PIPE (CSP) DETAILED IN THE PROJECT PLANS.

MATERIAL

THE ALUMINIZED TYPE 2 STEEL COILS SHALL CONFORM TO THE APPLICABLE REQUIREMENTS OF AASHTO M274 OR ASTM A929.

PIPE

THE CSP SHALL BE MANUFACTURED IN ACCORDANCE WITH THE APPLICABLE REQUIREMENTS OF AASHTO M36 OR ASTM A760. THE PIPE SIZES, GAGES AND CORRUGATIONS SHALL BE AS SHOWN ON THE PROJECT PLANS.

ALL FABRICATION OF THE PRODUCT SHALL OCCUR WITHIN THE UNITED STATES.

HANDLING AND ASSEMBLY

SHALL BE IN ACCORDANCE WITH RECOMMENDATIONS OF THE NATIONAL CORRUGATED STEEL PIPE ASSOCIATION (NCSPA)

INSTALLATION

SHALL BE IN ACCORDANCE WITH AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, SECTION 26, DIVISION II OR ASTM A798 AND IN CONFORMANCE WITH THE PROJECT PLANS AND SPECIFICATIONS. IF THERE ARE ANY INCONSISTENCIES OR CONFLICTS THE CONTRACTOR SHOULD DISCUSS AND RESOLVE WITH THE SITE ENGINEER.

IT IS ALWAYS THE RESPONSIBILITY OF THE CONTRACTOR TO FOLLOW OSHA GUIDELINES FOR SAFE PRACTICES.

ANTI-FLOTATION PROVISIONS DUE TO HIGH GROUNDWATER OR OTHER FLOTATION CONCERNS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.

MATERIAL SPECIFICATION

NOT TO SCALE

MARK	DATE	REVISION DESCRIPTION	BY

ENGINEERED SOLUTIONS LLC
A QUIKRETE® COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CMP DETENTION SYSTEMS

CONTECH PROPOSAL DRAWING

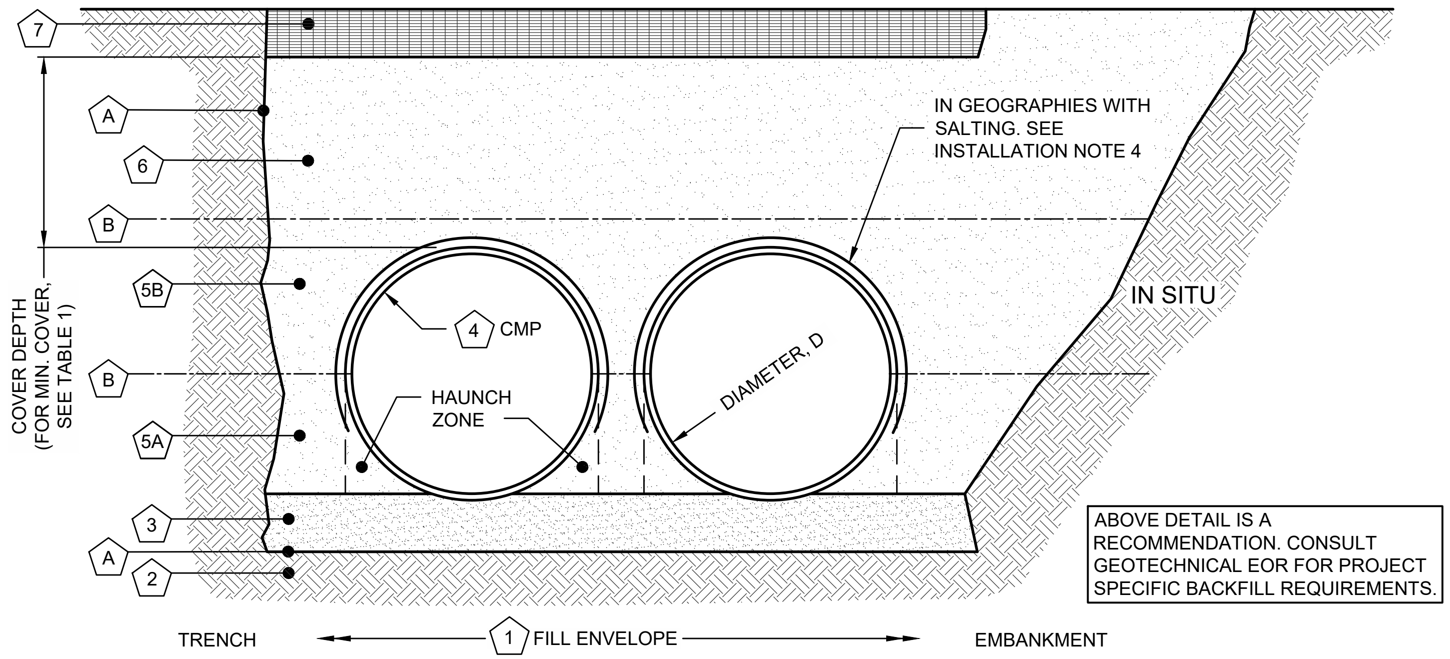
30"Ø UNDERGROUND DETENTION SYSTEM - 780630-015
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN B

PROJECT No.: 780630	SEQ. No.: 015	DATE: 5/21/25
DESIGNED: RLH	DRAWN: RLH	
CHECKED:	APPROVED:	
SHEET NO.: P2 OF 5		

TABLE 1:

DIAMETER, D	MIN. COVER	CORR. PROFILE
6"-10"	12"	1 1/2" x 1/4"
12"-48"	12"	2 2/3" x 1/2"
>48"-96"	12"	3" x 1", 5" x 1"
>96"	D/8	3" x 1", 5" x 1"

- STRUCTURAL BACKFILL MUST EXTEND TO LIMITS OF THE TABLE
- TOTAL HEIGHT OF COMPACTED COVER FOR CONVENTIONAL HIGHWAY LOADS IS MEASURED FROM TOP OF PIPE TO BOTTOM OF FLEXIBLE PAVEMENT OR TOP OF RIGID PAVEMENT
- ULTRAFLO ALSO AVAILABLE FOR SIZES 18" - 120" WITH 3/4"x 3/4"x 7 1/2" CORRUGATION



MINIMUM WIDTH DEPENDS ON SITE CONDITIONS AND ENGINEERING JUDGEMENT

INSTALLATION NOTES

1. WHEN PLACING THE FIRST LIFTS OF BACKFILL IT IS IMPORTANT TO MAKE SURE THAT THE BACKFILL IS PROPERLY COMPACTED UNDER AND AROUND THE PIPE HAUNCHES.
2. OTHER ALTERNATE BACKFILL MATERIAL MAY BE ALLOWED DEPENDING ON SITE SPECIFIC CONDITIONS, AS APPROVED BY SITE ENGINEER.
3. BACKFILL USING CONTROLLED LOW-STRENGTH MATERIAL (CLSM, "FLASH FILL" OR "FLOWABLE FILL") MAY BE USED WHEN THE SPACING BETWEEN THE PIPES WILL NOT ALLOW FOR PLACEMENT AND ADEQUATE COMPACTION OF THE BACKFILL. CONTACT CONTECH FOR FURTHER EVALUATION.
4. IF SALTING AGENTS FOR SNOW AND ICE REMOVAL ARE USED ON OR NEAR THE PROJECT, A GEOMEMBRANE BARRIER IS RECOMMENDED OVER THE UPPER HALF OF THE PIPE. THE GEOMEMBRANE LINER IS INTENDED TO HELP PROTECT THE SYSTEM FROM THE POTENTIAL ADVERSE EFFECTS THAT MAY RESULT FROM A CHANGE IN THE SURROUNDING ENVIRONMENT OVER A PERIOD OF TIME. PLEASE REFER TO THE CORRUGATED METAL PIPE DETENTION DESIGN GUIDE FOR ADDITIONAL INFORMATION.

TABLE 2:

CMP DETENTION AND CMP DRAINAGE STANDARD BACKFILL SPECIFICATIONS

	MATERIAL LOCATION	MATERIAL SPECIFICATION	DESCRIPTION	
1	FILL ENVELOPE WIDTH	PER ENGINEER OF RECORD	MINIMUM TRENCH WIDTH MUST ALLOW ROOM FOR PROPER COMPACTION OF HAUNCH MATERIALS UNDER THE PIPE. THE SUGGESTED MINIMUM TRENCH WIDTH, OR EOR RECOMMENDATION: PIPE ≤ 12": D + 16" PIPE > 12": 1.5D + 12"	MINIMUM EMBANKMENT WIDTH (IN FEET) FOR INITIAL FILL ENVELOPE: PIPE < 24": 3.0D PIPE 24" - 144": D + 4'0" PIPE > 144": D + 10'0"
2	FOUNDATION	**AASHTO 26.5.2** OR PER ENGINEER OF RECORD	PRIOR TO PLACING THE BEDDING, THE FOUNDATION MUST BE CONSTRUCTED TO A UNIFORM AND STABLE GRADE. IN THE EVENT THAT UNSUITABLE FOUNDATION MATERIALS ARE ENCOUNTERED DURING EXCAVATION, THEY SHALL BE REMOVED AND FOUNDATION BROUGHT BACK TO GRADE WITH A FILL MATERIAL APPROVED BY THE ENGINEER OF RECORD.	
3	BEDDING	**AASHTO M 43: 3, 357, 4, 467, 5, 56, 57** (APPROVED REGIONAL EQUIVALENTS INCLUDE CA-7)	ENGINEER OF RECORD TO DETERMINE IF BEDDING IS REQUIRED. PIPE MAY BE PLACED ON THE TRENCH BOTTOM OF A RELATIVELY LOOSE, NATIVE SUITABLE WELL GRADED GRANULAR MATERIAL THAT IS ROUGHLY SHAPED TO FIT THE BOTTOM OF THE PIPE, 2" MIN DEPTH. THE BEDDING MATERIAL MAY BE SUITABLE FOUNDATION SOILS CONFORMING TO AASHTO SOIL CLASSIFICATIONS A1, A2, OR A3 WITH MAXIMUM PARTICLE SIZE OF 3" PER AASHTO 26.3.8.1	
4	CORRUGATED METAL PIPE			
5A	CRITICAL BACKFILL	**AASHTO M 145: A-1, A-2, A-3** *	HAUNCH ZONE MATERIAL SHALL BE HAND SHOVELED OR SHOVEL SLICED INTO PLACE TO ALLOW FOR PROPER COMPACTION WITHOUT SOFT SPOTS. BACKFILL SHALL BE PLACED IN 8" +/- LOOSE LIFTS AND COMPACTED TO 90% STANDARD PROCTOR PER AASHTO T 99. BACKFILL SHALL BE PLACED SUCH THAT THERE IS NO MORE THAN A THREE LIFT (24") DIFFERENTIAL BETWEEN ANY OF THE PIPES AT ANY TIME DURING THE BACKFILL PROCESS. THE BACKFILL SHOULD BE ADVANCED ALONG THE LENGTH OF THE SYSTEM TO AVOID DIFFERENTIAL LOADING. WELL GRADED GRANULAR MATERIAL WHICH MAY CONTAIN SMALL AMOUNTS OF SILT OR CLAY AND MAXIMUM PARTICLE SIZE OF 3" (PER AASHTO 26.3.8.1 AND 12.4-1.3).	
5B	BACKFILL	**AASHTO M 145: A-1, A-2, A-3**		
6	COVER MATERIAL	UP TO MIN. COVER - SEE 5A AND 5B ABOVE ABOVE MIN. COVER - PER ENGINEER OF RECORD	COVER MATERIAL MAY INCLUDE NON-BITUMINOUS, GRANULAR ROAD BASE MATERIAL WITHIN MIN COVER LIMITS	
7	RIGID OR FLEXIBLE PAVEMENT (IF APPLICABLE)	PER ENGINEER OF RECORD	FLEXIBLE PAVEMENT SHOULD NOT BE COUNTED AS PART OF THE FILL HEIGHT OVER THE CMP. FINAL BACKFILL MATERIAL SELECTION AND COMPACTION REQUIREMENTS SHALL FOLLOW THE PROJECT PLANS AND SPECIFICATIONS PER THE ENGINEER OF RECORD.	
A	OPTIONAL SIDE GEOTEXTILE	NONE	GEOTEXTILE LAYER IS RECOMMENDED ON SIDES OF EXCAVATION TO PREVENT SOIL MIGRATION.	
B	OPTIONAL GEOTEXTILE BETWEEN LAYERS	NONE	IF SOIL TYPES DIFFER AT ANY POINT ABOVE PIPE INVERT, A GEOTEXTILE LAYER IS RECOMMENDED TO BE PLACED BETWEEN THE LAYERS TO PREVENT SOIL MIGRATION.	

NOTES:
- FOR MULTIPLE BARREL INSTALLATIONS, THE RECOMMENDED STANDARD SPACING BETWEEN PARALLEL PIPE RUNS SHALL BE THE PIPE DIAMETER /2 BUT NO LESS THAN 12" FOR DIAMETERS <72". FOR 72" AND LARGER DIAMETERS, THE MINIMUM SPACING IS 36". CONTACT YOUR CONTECH REPRESENTATIVE FOR NONSTANDARD SPACING.

* APPROVED REGIONAL EQUIVALENTS FOR SECTION 5A INCLUDE CA-7, MIDOT 2G, 34G, OR 21AA STONE OR GRAVEL; #8; #57; MIDOT 6A, 2G, 3G, 34G.

MANUFACTURER RECOMMENDED BACKFILL

NOT TO SCALE

\\QUIKRETE.NET\CONTECH\MERLIN\PROJECT\ACTIVE\780000\780630\780630-15-CMP DETENTION\DRAWINGS\780630-015-CMP CONFAB.DWG 5/21/2025 11:13 AM

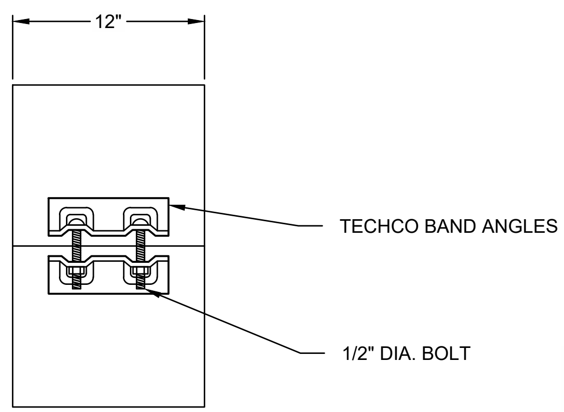
MARK	DATE	REVISION DESCRIPTION	BY

CONTECH ENGINEERED SOLUTIONS LLC
A QUIKRETE COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

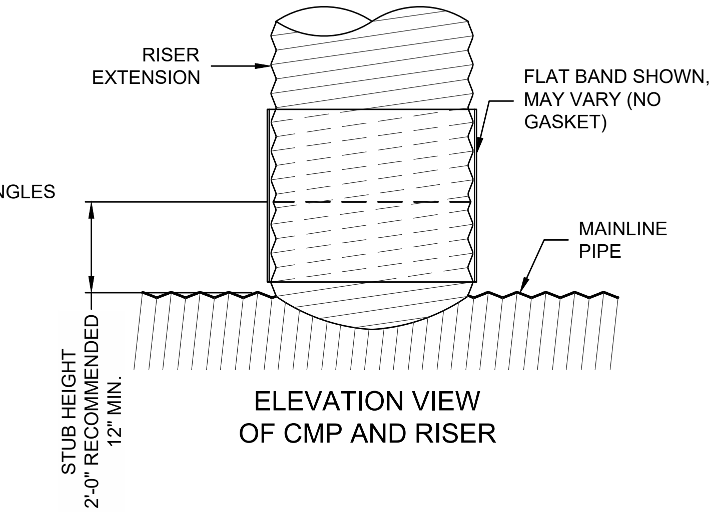
CONTECH CMP DETENTION SYSTEMS
CONTECH PROPOSAL DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-015
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN B

PROJECT No.: 780630	SEQ. No.: 015	DATE: 5/21/25
DESIGNED: RLH		DRAWN: RLH
CHECKED:		APPROVED:
SHEET NO.: P3	OF	5



CONNECTION DETAIL
7 1/2" TECHCO SHOWN - MAY VARY



ELEVATION VIEW
OF CMP AND RISER

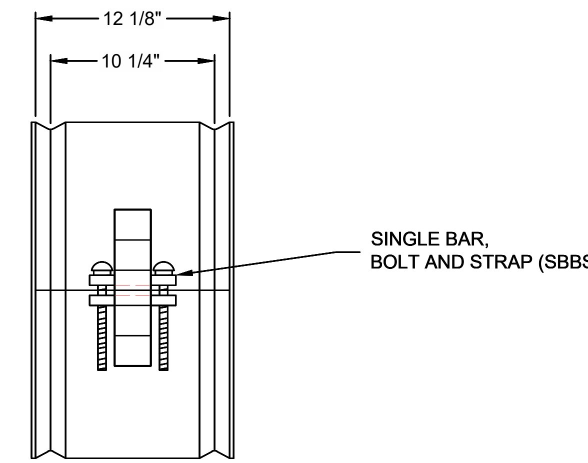
PLAIN END CMP RISER PIPE

GENERAL NOTES:

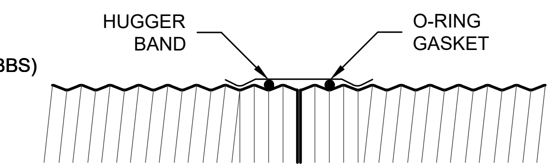
1. **DELIVERED BAND STYLE AND FASTENER TYPE MAY VARY BY FABRICATION PLANT.**
2. JOINT IS TO BE ASSEMBLED PER AASHTO BRIDGE CONSTRUCTION SPECIFICATION SEC 26.4.2.4.
3. BAND MATERIAL AND GAGE TO BE SAME AS RISER MATERIAL.
4. IF RISER HAS A HEIGHT OF COVER OF 10' OR MORE, USE A SLIP JOINT.
5. BANDS ARE NORMALLY FURNISHED AS FOLLOWS:
   - 12" THRU 48" 1-PIECE
   - 54" 2-PIECES
6. ALL RISER JOINT COMPONENTS WILL BE FIELD ASSEMBLED.
7. MANHOLE RISERS IN APPLICATIONS WHERE TRAFFIC LOADS ARE IMPOSED REQUIRE SPECIAL DESIGN CONSIDERATIONS.
8. DIMENSIONS SUBJECT TO MANUFACTURING TOLERANCES.

12" RISER BAND DETAIL

NOT TO SCALE



CONNECTION DETAIL
(SBBS)



O-RING GASKET

2 2/3"x1/2" RE-ROLLED END HEL-COR PIPE

GENERAL NOTES:

1. JOINT IS TO BE ASSEMBLED PER AASHTO BRIDGE CONSTRUCTION SPECIFICATION SEC 26.4.2.4.
2. BAND MATERIALS AND/OR COATING CAN VARY BY LOCATION. CONTACT YOUR CONTECH REPRESENTATIVE FOR AVAILABILITY.
3. BANDS ARE SHAPED TO MATCH THE PIPE-ARCH WHEN APPLICABLE.
4. BANDS ARE NORMALLY FURNISHED AS FOLLOWS:
   - 12" THRU 48" 1-PIECE
   - 54" THRU 96" 2-PIECES
   - 102" THRU 144" 3-PIECES
5. BAND FASTENERS ARE ATTACHED WITH SPOT WELDS, RIVETS OR HAND WELDS.
6. ALL CMP IS REROLLED TO HAVE ANNULAR END CORRUGATIONS OF 2 2/3"x1/2"
7. DIMENSIONS ARE SUBJECT TO MANUFACTURING TOLERANCES.
8. ORDER SHALL DESIGNATE GASKET OPTION, IF REQUIRED (SEE DETAILS ABOVE).

H-12 HUGGER BAND DETAIL

NOT TO SCALE

MARK	DATE	REVISION DESCRIPTION	BY

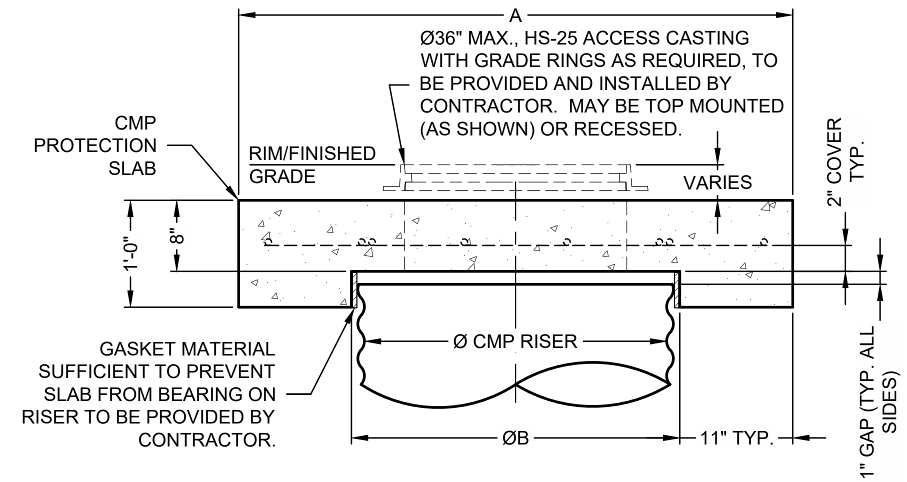
CONTECH ENGINEERED SOLUTIONS LLC
A QUIKRETE® COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH CMP DETENTION SYSTEMS

CONTECH PROPOSAL DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-015
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN B

PROJECT No.:	SEQ. No.:	DATE:
780630	015	5/21/25
DESIGNED: RLH		DRAWN: RLH
CHECKED:		APPROVED:
SHEET NO.: P4 OF 5		

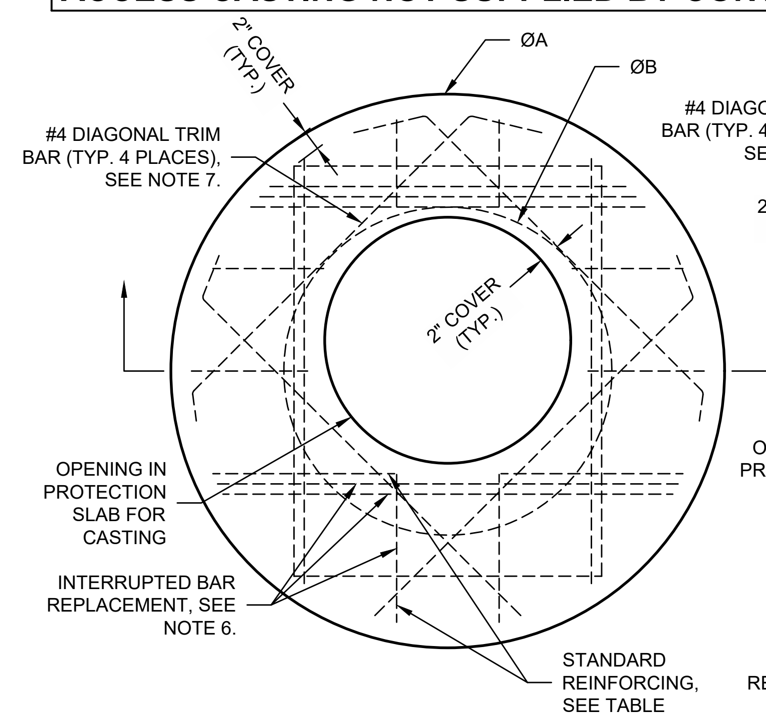


SECTION VIEW

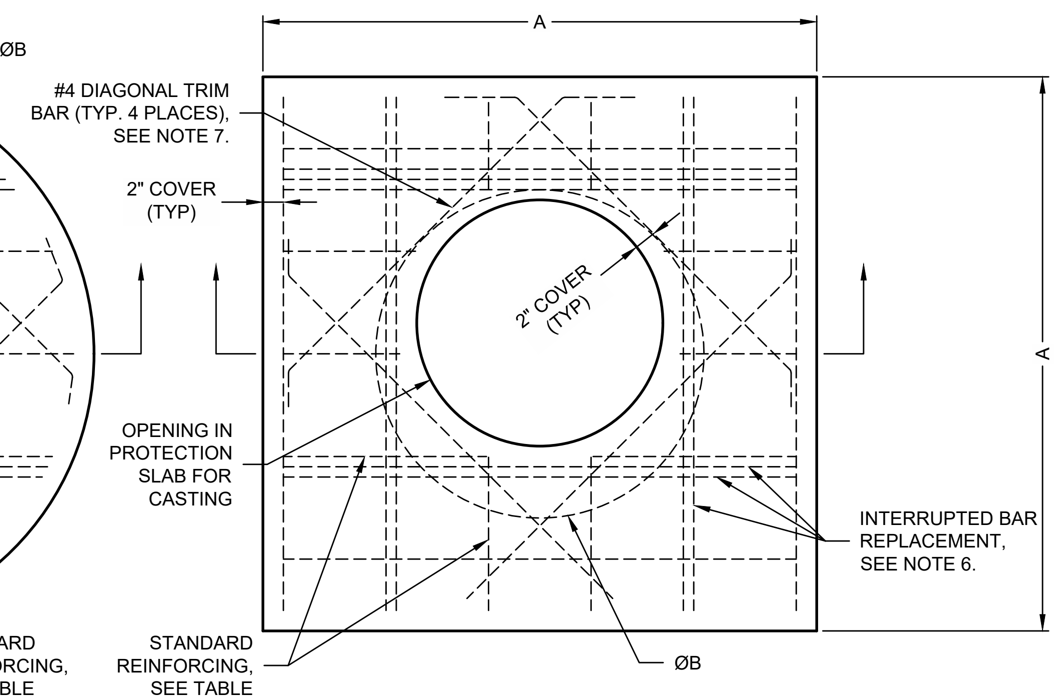
ACCESS CASTING NOT SUPPLIED BY CONTECH

REINFORCING TABLE				
Ø CMP RISER	A	Ø B	REINFORCING	**BEARING PRESSURE (PSF)
24"	Ø 4' 4'x4'	26"	#5 @ 10" OCEW #5 @ 10" OCEW	2,540 1,900
30"	Ø 4'-6" 4'-6" x 4'-6"	32"	#5 @ 10" OCEW #5 @ 9" OCEW	2,260 1,670
36"	Ø 5' 5' x 5'	38"	#5 @ 9" OCEW #5 @ 8" OCEW	2,060 1,500
42"	Ø 5'-6" 5'-6" x 5'-6"	44"	#5 @ 8" OCEW #5 @ 8" OCEW	1,490 1,370
48"	Ø 6' 6' x 6'	50"	#5 @ 7" OCEW #5 @ 7" OCEW	1,210 1,270

** ASSUMED SOIL BEARING CAPACITY



ROUND OPTION PLAN VIEW



SQUARE OPTION PLAN VIEW

NOTES:

1. DESIGN IN ACCORDANCE WITH AASHTO, 17th EDITION AND ACI 350.
2. DESIGN LOAD HS25.
3. EARTH COVER = 1' MAX.
4. CONCRETE STRENGTH = 4,000 psi
5. REINFORCING STEEL = ASTM A615, GRADE 60.
6. PROVIDE ADDITIONAL REINFORCING AROUND OPENINGS EQUAL TO THE BARS INTERRUPTED, HALF EACH SIDE. ADDITIONAL BARS TO BE IN THE SAME PLANE.
7. TRIM OPENING WITH DIAGONAL #4 BARS, EXTEND BARS A MINIMUM OF 12" BEYOND OPENING, BEND BARS AS REQUIRED TO MAINTAIN BAR COVER.
8. PROTECTION SLAB AND ALL MATERIALS TO BE PROVIDED AND INSTALLED BY CONTRACTOR.
9. DETAIL DESIGN BY DELTA ENGINEERS, ARCHITECTS AND LAND SURVEYORS, ENDWELL, NY.
10. THIS DETAIL REFLECTS CAST-IN-PLACE MANHOLE CAP PROTECTION SLAB. PRECAST MANHOLE CAP AVAILABLE IN SELECT MARKETS UNDER SEPARATE SUBMITTAL & DESIGN.

CAST-IN-PLACE MANHOLE CAP DETAIL

NOT TO SCALE

MARK	DATE	REVISION DESCRIPTION	BY

CONTECH ENGINEERED SOLUTIONS LLC
A QUIKRETE COMPANY
9100 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH CMP DETENTION SYSTEMS

CONTECH PROPOSAL DRAWING

30"Ø UNDERGROUND DETENTION SYSTEM - 780630-015
VALLEY CENTRE WAREHOUSE - CITY OF INDUSTRY
LA PUENTE, CA
SITE DESIGNATION: BASIN B

PROJECT No.: 780630	SEQ. No.: 015	DATE: 5/21/25
DESIGNED: RLH		DRAWN: RLH
CHECKED:		APPROVED:
SHEET NO.: P5	OF	5

Preliminary Storm Drain Sizing

Project Description

Friction Method	Manning Formula
Solve For	Full Flow Capacity

Input Data

Roughness Coefficient	0.012
Channel Slope	0.005 ft/ft
Normal Depth	12.0 in
Diameter	12.0 in
Discharge	2.73 cfs

Diameter (in)	Normal Depth (in)	Discharge (cfs)	Velocity (ft/s)	Flow Area (ft²)	Wetted Perimeter (ft)	Top Width (ft)
6.0	6.0	0.43	2.19	0.2	1.6	0.00
12.0	12.0	2.73	3.47	0.8	3.1	0.00
18.0	18.0	8.05	4.55	1.8	4.7	0.00
24.0	24.0	17.33	5.52	3.1	6.3	0.00

Worksheet for Area A-Curb Inlet Hydraulics (Sump)

Project Description

Solve For	Spread

Input Data

Discharge	2.12 cfs
Gutter Width	2.00 ft
Gutter Cross Slope	0.020 ft/ft
Road Cross Slope	0.020 ft/ft
Curb Opening Length	3.5 ft
Opening Height	0.5 ft
Curb Throat Type	Horizontal
Local Depression	3.0 in
Local Depression Width	24.0 in
Throat Incline Angle	90.00 degrees

Results

Spread	12.8 ft
Depth	3.1 in
Gutter Depression	0.0 in
Total Depression	3.0 in

Worksheet for Area A-Curb Inlet Hydraulics (Sump)

Notes:

Total Drainage Area A = 2.64 acres
Roof tributary area = 1.88 acres

1.88/2.64=71.21 % (Percent of tributary area drains to roof pipe)
100-71.21=28.79 % (Percent of parking area to inlets of Area A)

Tributary Area flow rate:
Q50=7.38 CFS

Tributary flow on pavement:
7.38* (0.2879)=2.12 cfs
Use minimum 3.5' curb opening.

Worksheet for Area B-Curb Inlet Hydraulics (Sump)

Project Description

Solve For	Spread

Input Data

Discharge	2.87 cfs
Gutter Width	2.00 ft
Gutter Cross Slope	0.020 ft/ft
Road Cross Slope	0.020 ft/ft
Curb Opening Length	3.5 ft
Opening Height	0.5 ft
Curb Throat Type	Horizontal
Local Depression	3.0 in
Local Depression Width	24.0 in
Throat Incline Angle	90.00 degrees

Results

Spread	15.7 ft
Depth	3.8 in
Gutter Depression	0.0 in
Total Depression	3.0 in

Worksheet for Area B-Curb Inlet Hydraulics (Sump)

Notes:

Area B Tributary flow = 2.87 cfs
assuming 2.87 cfs as parking drainage, resulting depth of catch basin is acceptable.
Use minimum curb opening of 3.5'

Worksheet for Area C-Curb Inlet Hydraulics (Sump)

Project Description

Solve For	Spread

Input Data

Discharge	5.56 cfs
Gutter Width	2.00 ft
Gutter Cross Slope	0.020 ft/ft
Road Cross Slope	0.020 ft/ft
Curb Opening Length	3.5 ft
Opening Height	0.5 ft
Curb Throat Type	Horizontal
Local Depression	3.0 in
Local Depression Width	24.0 in
Throat Incline Angle	90.00 degrees

Results

Spread	24.4 ft
Depth	5.9 in
Gutter Depression	0.0 in
Total Depression	3.0 in

Worksheet for Area C-Curb Inlet Hydraulics (Sump)

Notes:

Area C Tributary flow = 5.56 cfs
Resulting depth of catch basin is acceptable.
Use minimum curb opening of 3.5'

APPENDIX E: GEOTECHNICAL REPORT AND REFERENCES

- Geotechnical Due Diligence Exploration by Leighton Consulting, Inc.
- Asbuilt drawing No. 40792-40795, Job No. 57346

GEOTECHNICAL DUE DILIGENCE EXPLORATION PROPOSED INDUSTRIAL WAREHOUSE DEVELOPMENT, 110 SOUTH 6TH AVENUE CITY OF INDUSTRY, LOS ANGELES COUNTY, CALIFORNIA

Prepared For **CAPROCK ACQUISITIONS III, LLC**
1300 DOVE STREET, SUITE 200
NEWPORT BEACH, CA 92660

Prepared By **LEIGHTON CONSULTING, INC.**
10532 ACACIA STREET, SUITE B-6
RANCHO CUCAMONGA, CA 91730

Project No. 13877.001

May 11, 2023

Leighton

Leighton Consulting, Inc.
A Leighton Group Company

May 11, 2023

Project No. 13877.001

CapRock Acquisitions III, LLC
1300 Dove Street, Suite 200
Newport Beach, California 92660

Attention: Mr. Christian Spence
Assistant Project Manager

Subject: Geotechnical Due Diligence Exploration
Proposed Industrial Warehouse Development
110 South 6th Avenue
City of Industry, Los Angeles County, California

In accordance with your authorization, Leighton Consulting, Inc. (Leighton) has performed geotechnical due diligence exploration for the proposed industrial warehouse development located at 110 South 6th Avenue in the City of Industry, Los Angeles County, California. Our understanding of this project is based on email correspondence with you and a review of the *Site Plan Option 06 - 2023-03-30*, prepared by Douglas Franz Architects, Inc., that you provided. The purpose of this study has been to collect surface and subsurface geotechnical data at the site with regard to the proposed development, to evaluate the proposed development with respect to site geotechnical conditions, and to provide geotechnical recommendations for design and construction of the proposed development.

Based on this geotechnical exploration, construction of the proposed warehouse development is feasible from a geotechnical standpoint. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, potentially compressible soils near the surface, and potentially liquefiable soil layers in the subsurface. Good planning and design of the project can limit the impact of these constraints. This report presents our findings, conclusions, and geotechnical recommendations for the project.

We appreciate the opportunity to work with you on the development of this project. If you have any questions regarding this report, please call us at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

Steven G. Okubo, CEG 2706
Associate Geologist

Jason D. Hertzberg, GE 2711
Principal Engineer

BM/SGO/JDH/rsm

Distribution: (1) Addressee

TABLE OF CONTENTS

<u>Figures (Rear of Text)</u>

<u>Appendices</u>

1.0 INTRODUCTION

1.1 Site Location and Description

The proposed warehouse will be situated on a roughly 4.5-acre site located southeast of 6th Street and approximately 400 feet southwest of Valley Boulevard. Review of aerial imagery indicates that the project site is currently occupied by 6 industrial/commercial buildings and asphalt-paved parking areas and drive aisles. Based on available topographic maps and the elevation model from Google Earth, elevations within the project site range from approximately 296 feet above mean sea level (msl) in the northeast to approximately 295 feet msl in the southwest.

Based on a review of available historical aerial imagery, the project site was used for agricultural purposes in 1948. By 1972, industrial buildings were erected to the west, southwest, and southeast, and by 1980 the current buildings onsite were built and remaining open parcels surrounding the site were constructed with industrial buildings.

1.2 Proposed Development

Our understanding of this project is based on email correspondence with you and a review of the *Site Plan Option 06 - 2023-03-30*, prepared by Douglas Franz Architects, Inc., you provided. We understand the development conceptually consists of the demolition of the buildings currently onsite and the construction a 93,920-square-foot warehouse with office space and 11 dock doors. No subterranean levels are indicated in the conceptual plan. Ancillary improvements include new drive aisles, parking stalls, and presumably underground utilities. Based on discussions with you, we understand potential infiltration systems are planned, but locations and invert depths are not available yet. A detailed site plan and structural loading were not available at the time of this report. We anticipate that the warehouse will be composed of concrete tilt-up walls.

1.3 Purpose of Investigation

The purpose of this study has been to evaluate the geotechnical conditions of this site with respect to the proposed development and to provide preliminary geotechnical recommendations for design and construction of the development.

1.4 Scope of Work

Our geotechnical due diligence exploration included hollow-stem auger soil borings, infiltration tests, surface observations, laboratory testing, and geotechnical analysis of collected information to evaluate geotechnical conditions relating to the proposed industrial warehouse and to provide preliminary conclusions and recommendations for earthwork and design for your due diligence assessment of this development. The scope of our study has included the following tasks:

- Background Review: We reviewed available, relevant geotechnical and geologic maps and reports and aerial photographs available from our in-house library, available online, or those provided by you.

- Utility Coordination: We contacted Dig Alert (811) prior to excavating borings so that utility companies could mark utilities onsite. Because this project site is developed, we had retained a private utility locator to assist in identifying shallowly buried utilities in the areas of our proposed borings.

- Field Exploration: Six (6) hollow-stem auger borings (LB-1 through LB-4, IT1, and IT-2) were logged and sampled onsite on April 12, 2023 to evaluate subsurface conditions onsite. These borings were drilled by a subcontracted rig to depths ranging from approximately 10 to 51½ feet below the ground surface (bgs). Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Modified split barrel sampler lined with rings. Standard Penetration Tests (SPT) were conducted at selected depths and samples were obtained at those intervals. Representative bulk soil samples were also collected at shallow depths from the borings.

  Boring LB-1 was drilled through asphalt pavement, and was backfilled with soil cuttings to approximately the bottom of the pavement section. The remainder of boring LB-1 was patched to the level of the surface of the surrounding asphalt with black-dyed concrete. The remainder of our borings were drilled in planter or lawn areas and were backfilled to the surface with soil cuttings. Logs of the geotechnical borings are presented in Appendix B. Approximate boring locations are shown on the accompanying Figure 2, *Geotechnical Map*.

We conducted well permeameter tests at two locations (IT-1 and IT-2) to evaluate general infiltration characteristics of the subsurface soils for planning purposes at the depths and locations tested. These test zones targeted soil layers most favorable for infiltration selected based on the soil types encountered in nearby borings LB-3 and LB-4. These well permeameter tests were conducted based on the USBR 7300-89 method and in general accordance with Los Angeles County guidelines. Testing consisted of constant head and falling head infiltration using a water trailer to transport water to each location. A 2-inch diameter, slotted PVC pipe was used within each boring's test zone, with sand backfilled around the slotted pipe. These tests were conducted at bottom depths of approximately 10 feet bgs in IT-1 and 25 feet bgs in IT-2. Infiltration test logs are included in Appendix B.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests are currently being conducted on selected relatively undisturbed and bulk soil samples obtained during our subsurface exploration to further evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation include:

  - Maximum dry density and optimum moisture content
  - In situ moisture content and density
  - Sieve analysis for grain-size distribution
  - Atterberg limits
  - Expansion Index
  - Collapse/swell-settlement
  - R-value
  - Water-soluble sulfate concentration in the soil
  - Resistivity, chloride content and pH

  Laboratory tests are provided in Appendix C, *Laboratory Test Results*.

- Engineering Analysis: Data obtained from our background review and from our field exploration was evaluated and analyzed to develop conclusions and provide geotechnical recommendations presented in this report.

- Report Preparation: Preliminary results of our geotechnical exploration have been summarized in this report, presenting our findings, conclusions and geotechnical recommendations for design and construction of the proposed development.

2.0 FINDINGS

2.1 Regional Geologic Conditions

The site is located within the San Gabriel Valley, an alluvial basin drained by the Rio Hondo and San Gabriel Rivers, in the Peninsular Ranges geomorphic province. The San Gabriel Valley is bound on the north by the San Gabriel Mountains and Cucamonga fault, on the west by the San Rafael Hills, on the east by the San Jose Hills, and on the south by the Puente Hills. Sediments within the Valley include about 6,000 feet of marine and non-marine sedimentary rocks of Quaternary age (Yerkes, et al 1965) overlain by more recent sediments deposited by the San Gabriel River consisting predominately of granitic sands, gravels, cobbles and boulders shed from the surrounding highlands.

Published regional geologic mapping has indicated that the project site is underlain by Holocene undifferentiated alluvial fan and valley deposits consisting of silt and clay (Tan, 2000). Figure 3, *Regional Geology Map,* depicts the site location in relation to the predominate geologic materials (alluvium) of the area. Figure 4, *Regional Fault and Historical Seismicity Map,* presents the site location in relation to active faults and epicenters of relatively large (> M_w 4.0) historical earthquakes.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by a mantle of artificial fill (afu) over native sediments consisting of Quaternary undifferentiated alluvial fan and valley deposits (Qyf). Artificial fill encountered within our borings onsite ranged in thickness between approximately 4 to 7 feet and consisted of loose to medium dense clayey sand (SC) with minor amounts of silty sand (SM) and sandy silt (ML). Because reporting of the engineering and placement of artificial fill onsite was not available for our review, we have characterized all artificial fill onsite as undocumented.

Native undifferentiated alluvial fan and valley deposits (alluvium) encountered below undocumented artificial fill in our borings generally consisted loose to dense silty sand (SM), clayey sand (SC), poorly graded sand (SP) and medium stiff to stiff sandy clay (CL). In situ moisture content in the upper 20 feet in native soils ranged from 3 to 16 percent.

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge. Based on the conditions encountered in our borings and our experience in the area, native soils are considered moderately compressible.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Laboratory tests performed on a representative soil sample indicated a collapse of 1.1%. Field standard penetration tests generally indicate that onsite granular soils are dense. Based on our overexcavation and compaction recommendations provided in Section 3.1 of this report, soil collapse and consolidation are not a significant issue at this site.

2.2.2 Expansive Soils

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subjected to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of building foundations and slabs-on-grade could result.

Based on the soils encountered in our borings and our experience in the area, onsite soils are anticipated to have low expansion potential.

2.2.3 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on American Concrete Institute (ACI) provisions, adopted by the 2019 CBC (CBC, 2019, Chapter 19, and ACI 318, 2014).

A near-surface soil sample collected during subsurface exploration for this project had been tested for soluble sulfate content. Based on the results of this testing, the sulfate exposure from onsite soils is expected to be negligible (Exposure Class S0). Recommendations for concrete in contact with these soils are provided in Section 3.11.

2.2.4 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content and pH. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested to screen for minimum resistivity, chloride content, and pH. Based on this laboratory testing, onsite soils are considered to be moderately corrosive to buried ferrous metals.

2.3 Groundwater

The site of the proposed industrial warehouse development is located within the 167-square-mile Main San Gabriel Basin.

The California Geological Survey (CGS) performed a groundwater elevation evaluation for the Baldwin Park Quadrangle within the alluvial soils in order to evaluate the liquefaction potential of the soils (CGS, 1998b). In the vicinity of the project site, the historically shallowest groundwater depth is reported to have been approximately 10 feet bgs (CGS, 1998b). CGS's historically highest groundwater level mapping was based on information including 1904 and 1944 groundwater contour maps and geotechnical boring data spanning from 1960 through 1997. Their evaluation indicated that shallow groundwater conditions in the area of the proposed industrial development were indicated by 1904 and 1944 mapping.

In the 1950's, rapid urbanization resulted in increased water consumption in the Main San Gabriel Basin, decreasing the water supply to those downstream of the basin. With the resulting lowering of the groundwater table, the municipal water District filed a complaint in 1968 to bring water producers in the basin under the control of one governing body. In 1973, in response to the complaint, the Main San Gabriel Basin Watermaster was formed to manage basin-wide water resources.

Since the San Gabriel Watermaster was formed, groundwater management appears to have focused on recharging replacement water to maintain groundwater levels. Groundwater levels had been stable since the Watermaster's formation until 2011, when drought conditions have lowered groundwater levels.

Measurements from State and County wells near the project site were reviewed to estimate historically highest groundwater levels in the region of the project site since the formation of the Watermaster.

Well No.	Distance from Site	Measurement Date Range	Historical Highest Groundwater Depth (ft.)	Historical Highest Groundwater Date
State Well No. 340334N1179796W001/ LA County Well ID 5036N	0.2 mile south of site	September 1966 to October 2022	33	April 2, 1970
State Well No. 340355N1179827W001	0.2 mile southwest of site	July 2011 to July 2022	64	January 16, 2012
LA County Well ID 3025N	0.5 mile north of site	October 1987 to December 2022	68	April 1, 1997
State Well No. 340454N1179830W001	0.7 mile northwest of site	July 2011 to July 2022	73	July 5, 2012
State Well No. 340469N1179761W001	0.8 mi north of site	July 2011 to July 2022	83	January 4, 2012

Additionally, groundwater was not encountered within our exploratory borings performed on April 12, 2022, which reached as deep as approximately 51½ feet bgs.

Based on the reviewed well and boring data, historically highest groundwater levels in the region of the site were approximately 33 feet bgs in 1970, about 3 years prior to the formation of the Main San Gabriel Basin Watermaster. Considering the Watermaster's efforts to maintain groundwater levels from pumping and overdraft by recharging replacement water, the likelihood of groundwater becoming shallower than 33 feet bgs in the future is very unlikely.

2.3.1 Regional Subsidence

Regional ground subsidence generally occurs due to rapid and intensive removal of subterranean fluids, typically water or oil. It is generally attributed to the consolidation of sediments as the fluid in the sediment is removed. The total load of the soils in partially saturated or saturated deposits is born by their granular structure and the fluid. When the fluid is removed, the load is born by the sediment alone and it settles.

The project site has been mapped by the U.S. Geological Society (2023) to be outside of an area of land subsidence from intense removals of significant quantities of water, peat, or oil extraction in the area. Based on this and no known reports indicating land subsidence of the site's area, the potential for ground subsidence is considered to be less than a significant impact.

2.4 Faulting and Seismicity

In general, primary seismic hazards for sites in the region include surface rupture along active faults and strong ground shaking. The potential for fault rupture and seismic shaking are discussed below.

2.4.1 Surface Faulting

Based on our research, no Earthquake Fault Zones or active faults appear to have been mapped on or trending towards the site. The closest mapped active or potentially active faults are presented in the following table.

Fault Name	Approximate Distance from Site
Whittier fault	3.4 miles to the southwest
San Jose fault	5.7 miles to the east
Upper Elysian Park fault	7.4 miles to the west

Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered low.

2.4.2 Seismic Design Parameters

The site has experienced and is anticipated to experience strong ground shaking during the life of the project resulting from an earthquake occurring along one or more of the major active or potentially active faults in southern California. Accordingly, the project should be designed in accordance with applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

The following seismic parameters should be considered for design under the 2022 edition of the California Building Code (CBC). The following table lists seismic design parameters based on the 2022 CBC and ASCE 7-16 methodology:

	Site Seismic Coefficients / Coordinates		Value (g)
Latitude: 34.0361		Longitude: -117.9785	
2022 CBC Parameters	Spectral Response – Class D (short), S_S		1.753
	Spectral Response – Class D (1 sec), S_1		0.63
	Max. Considered Earthquake Spectral Response Acceleration (short), S_{MS}		1.753
	Max. Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}		null
	5% Damped Design Spectral Response Acceleration (short), S_{DS}		1.169
	5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}		null

* See Section 11.4.8 of ASCE 7-16. A site-specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-16 is required for this site. **Per Supplement 3 to ASCE 7-16, a site-specific ground motion hazard analysis is not required where the value of the parameters SM_1 and SD_1 in the table are increased by 50%.**

** Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.

The project structural engineer should review the seismic parameters. Site-Specific analyses output is presented in Appendix D.

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 7.7 (M_W) at a distance on the order of 8 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years), with a corresponding peak ground acceleration of 0.79g.

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landslides, and earthquake-induced flooding. The potential for secondary seismic hazards at the site is discussed below.

2.5.1 Liquefaction Potential

Liquefaction is the loss of soil strength due to a buildup of excess pore-water pressure during strong and long-duration ground shaking. Liquefaction is associated primarily with loose (low density), saturated, relatively uniform fine- to medium-grained, clean cohesionless soils. As shaking action of an earthquake progresses, soil granules are rearranged and the soil densifies within a short period. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, soil shear strength reduces abruptly and temporarily behaves similar to a fluid. For liquefaction to occur there must be:

(1) loose, clean granular soils,
(2) shallow groundwater, and
(3) strong, long-duration ground shaking

The site is within a liquefaction hazard zone as mapped by the State of California (CGS, 1999).

We have performed an analysis based on the modified Seed Simplified Procedure as detailed by Youd et al. (2001) and Martin and Lew (1999), which compares the seismic demand on a soil layer (Cyclic Stress Ratio, or CSR) to the capacity of the soil to resist liquefaction (Cyclic Resistance Ratio, or CRR), (Youd el al., 2001). A minimum required factor of safety of 1.3 was used in our analysis, with factor of safety defined as CRR/CSR. As required, our analysis assumes that the design earthquake would occur while the groundwater is at its estimated design level (historically highest level). Parameters utilized in our analysis include Standard Penetration Test (SPT) results from the hollow-stem auger borings, visual descriptions of soil samples retrieved, and geotechnical laboratory test results, including sieve and hydrometer analysis, Atterberg limits, and moisture content. Soil susceptibility to liquefaction is estimated based on several factors, including relative density, fines content, plasticity, and moisture content.

Based on our analysis, several layers between 40 to 50 feet below the ground surface are considered potentially susceptible to liquefaction when considering a design groundwater depth of 33 feet bgs. The liquefiable layers consists of silty sands with an interbedded clay layer at 45 feet bgs.

A key aspect of liquefaction is what effect it may have on the proposed improvements in terms of surface manifestations, and seismic settlement. These are addressed in the following sections. With this analysis, the potential for surface manifestations of liquefaction, such as bearing failures and sand boils, exists, based on Ishihara (1995).

A summary of the liquefaction analysis is included in Appendix D.

2.5.2 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed, and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). Design/historic high groundwater levels of 33 feet below ground surface were used in the analysis. Based on our analysis, a potential for approximately 6.1 inches of seismic settlement is estimated at the site; however, based on our overexcavation recommendations presented later in this report, the maximum estimated potential seismic settlement is reduced to approximately 5.7 inches. Results of our seismic settlement analysis is presented in Appendix D.

If the potential differential settlement is estimated as half of the total seismic settlement over a horizontal distance of 30 feet, this would result in a maximum 2.9 inches differential settlement in 30 feet, or angular distortion of 0.0081L, considering the recommended overexcavation. The structural engineer should determine Structure Type and Risk Category and evaluate whether the differential settlement estimates described above are tolerable. A copy of ASCE 7-16 Table 12.13-3 is provided as follows for reference.

Table 12.13-3 Differential Settlement Threshold

Structure Type	Risk Category		
	I or II	III	IV
Single-story structures with concrete or masonry wall systems	0.0075L	0.005L	0.002L
Other single-story structures	0.015L	0.010L	0.002L
Multistory structures with concrete or masonry wall systems	0.005L	0.003L	0.002L
Other multistory structures	0.010L	0.006L	0.002L

2.5.3 Bearing Failures/Surface Manifestations

We performed an analysis of the potential for bearing failures/structural damage due to liquefaction (surface manifestations) based on the work of Ishihara (1995) and as described in Martin and Lew (1999). This method is based on empirical data and considers the thickness of non-liquefiable soil below the ground surface and foundations, compared to the thickness of underlying liquefiable soils. Our analysis based on this method does not indicate that there is a potential for structural damage due to liquefaction, due to the significant depth to potentially liquefiable soils.

2.6 Infiltration Testing

Two well permeameter tests (IT-1 and IT-2) were conducted to estimate the infiltration characteristics at specific locations of the site. Locations and invert depths of future infiltration systems for this development were not available for this study. We assumed two locations for assessing feasibility of onsite infiltration and targeted test depths based on soils encountered in borings LB-4 and LB-3, which were located adjacent to test locations IT-1 and IT-2, respectively. Test IT-1 was located southwest of the proposed building near the lowest elevation onsite and next to 6th Avenue, and test IT-2 was located towards the southeastern portion of the site, which is also at a relatively low elevation onsite. Infiltration tests were conducted at bottom depths of approximately 10 feet bgs in IT-1 and 25 feet bgs in IT-2.

Well permeameter tests are useful for field measurements of soil infiltration rates, and are suited for testing when the design depth of the basin or chamber is deeper than current existing grades. It should be noted that this is a clean-water, small-scale test, and that correction factors need to be applied. A test consists of excavating a boring to the depth of the test (or deeper as long as it is partially backfilled with soil and a bentonite plug with a thin soil covering is placed just below the design test elevation). A layer of clean sand or gravel is then placed in the boring bottom to temporarily support a perforated well casing pipe system. Once the well casing pipe has been installed, coarse sand or gravel is poured in the annular space outside of the well casing within the test zone to prevent the boring from caving/collapsing or spalling when water is added. Water is added into the boring to an initial water height, as water within the boring infiltrates into the soil, measurements are taken of the height of the water column within the boring at equally timed intervals (known as a falling head test). The infiltration rate as measured during intervals of the test is defined as the flow rate of water infiltrated, divided by the surface area of the infiltration interface. The test was conducted based on the USBR 7300-89 test method.

Small-scale infiltration rates as summarized in the table below. Results of the infiltration testing are provided in Appendix B.

Boring	Test Depth (ft)	Soil Classification	Raw Infiltration Rates (in./hr)[1]
IT-1	5 to 10	Silty Sand (15% to 30% fines, estimated)	1.0
IT-2	20 to 25	Silty Sand (5% to 25% fines, estimated)	1.7

[1] Factor of safety should be applied to raw rates

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed warehouse development is feasible from a geotechnical standpoint. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed warehouses. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, potentially compressible soils, and moderate seismic settlement. Good planning and design of the project can limit the impact of these constraints. Remedial recommendations for these and other geotechnical issues are provided in the following sections.

Undocumented fill onsite, which was found to be as thick as approximately 7 feet, should be completely removed and properly compacted during earthwork construction. We did not evaluate environmental conditions as part of this study.

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically amended below, or by future recommendations based on final development plans.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of debris, which should be disposed of offsite. Any underground obstructions should be removed. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted.

3.1.2 Removal of Undocumented Artificial Fill

Prior to overexcavation and recompaction of onsite alluvial soil, uncontrolled artificial fill should be completely removed and may be used as compacted fill for the project, provided any oversized rock is suitably handled and any deleterious materials are removed. Undocumented fill was found in the boring drilled during subsurface exploration to be as thick as 7 feet.

3.1.3 Overexcavation and Recompaction

To reduce the potential for adverse total and differential settlement of the proposed structures, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved.

All undocumented artificial fill within the proposed building pad should be removed and reused as compacted fill.

Based on our seismic settlement analysis, we recommend that onsite soils in the proposed building pad area and site walls taller than 8 feet be overexcavated to a minimum depth of 8 feet bgs, or a depth of 3 feet below the bottoms of proposed footings, whichever is deeper.

Where possible, the removal bottom should extend horizontally a minimum of 5 feet from the outside edges of the building footprint and footings (including columns connected to the buildings), or a distance equal to the depth of overexcavation below the footings, whichever is farther. Where this is not achievable, this should be reviewed on a case-by-case basis.

During overexcavation, the soil conditions should be observed by Leighton to further evaluate these recommendations based on actual field conditions encountered. A firm removal bottom should be established across the building footprint to provide uniform foundation support for the proposed structure. Leighton should observe and test the removal bottom prior to placing fill. Deeper overexcavation and recompaction may be recommended locally if undocumented artificial fill is still exposed and until a firm removal bottom is achieved.

Areas outside of proposed structures and planned for new asphalt or concrete pavement (such as parking areas or fire lanes), flatwork (such as sidewalks), site walls up to 8 feet tall and retaining walls retaining up to 3 feet of soil (taller walls should be overexcavated per the recommendations for buildings), areas to receive fill, and other improvements, should be overexcavated to a minimum depth of 3 feet below existing grade or 2 feet below proposed subgrade (including the footing subgrade for walls), whichever is deeper.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D1557 laboratory maximum density.

3.1.4 Mitigation of Potential Seismic Settlement

The potential total settlement resulting from seismic loading is considered moderate (up to 6.1 inches) for this site, assuming the historic high groundwater level and design level earthquake. Differential settlement resulting from seismic loading is generally assumed to be one-half of the total seismically induced settlement over a distance of 30 feet. We recommend that the potential for damaging seismic settlement be reduced by 1) overexcavating the near-surface soils to a depth of 8 feet as described in the previous section, 2) placing 2 geogrid layers within the compacted fill under the proposed footings, and 3) providing foundations as described in Section 3.5.

In order to mitigate differential settlements to tolerances that meet project requirements, we recommend the use of a geogrid-reinforced granular mat to provide support to shallow foundation footings. We recommend that the footings of the proposed structure be underlain by two layers of a Tensar TriAx TX160 triaxial geogrid. The first layer of geogrid should be placed on the recompacted removal bottom approximately 8 feet below existing grade. The first layer of geogrid should be laid at bottom of the overexcavation, extending a minimum lateral distance equal to its depth below footing feet beyond the footing edge and extending up the sides of the excavation with enough geogrid to allow for a 10-foot fold-over return one foot above the second layer of geogrid. A 1-foot-thick layer of aggregate base should be placed over the first layer of geogrid; the aggregate base should be compacted to a minimum of 95 percent relative compaction per ASTM D1557. The second layer of geogrid and another 1-foot-thick layer of aggregate base should be placed over the initial base layer and extended to the edge of the footing excavation fill. The 10-foot return should then be placed over the second layer of compacted base. The remaining overexcavation backfill using onsite soils should then continue to design grade, compacted to a minimum of 95 percent relative compaction per ASTM D1557.

Additional geogrid construction considerations are presented in Section 3.10 of this report.

3.1.5 Fill Placement and Compaction

Onsite soil to be used for compacted structural fill should also be free of organic material debris and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and possibly tested by Leighton.

All fill soil should be placed in thin, loose lifts, moisture conditioned, as necessary to at least 2 percentage points above optimum moisture content, and compacted to a minimum 90 percent relative compaction. However, the compacted fill under building foundations and the upper 24 inches of fill under the building slab should be compacted to a minimum of 95 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction.

3.1.6 Import Fill Soil

Import soil to be placed as fill should be geotechnically accepted by Leighton. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil samples. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.1.7 Shrinkage and Subsidence

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. This value does not factor in removal of debris or other materials. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site, the measured in-place densities of soils encountered and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage	Approximately 10 ± 5 percent
Subsidence (overexcavation bottom processing)	Approximately 0.2 foot

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

3.1.8 Rippability and Oversized Material

Oversized material (rock or rock fragments greater than 8 inches in dimension) was not observed during our investigation. Oversized material should not be used within structural fill areas.

3.2 Shallow Foundation Recommendations

Overexcavation and recompaction of the footing subgrade should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a low expansion potential.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment per code requirements, with a minimum width of 24 and 12 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 2,500 pounds-per-square-foot (psf) may be used, based on an assumed embedment depth of 18 inches and minimum width described above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,500 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

3.2.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.35. The passive resistance may be computed using an allowable equivalent fluid pressure of 250 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

The allowable bearing pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

3.2.5 Settlement Estimates

The above recommended allowable bearing pressure is generally based on a total allowable, post-construction total settlement of 1 inch, for column loads and wall loads not exceeding 50 kips and 3 kips per foot, respectively, for dead plus sustained live loads. Differential settlement due to static loading is generally estimated at ½ inch over a horizontal distance of 30 feet. Once developed by the Structural Engineer, we can review total dead and sustained live loads for each column including plan location and span distance, to evaluate if differential settlements between dissimilarly loaded columns will be tolerable. Excessive differential settlement can be mitigated with the use of reduced bearing pressures, deeper footing embedment, possibly changing overexcavation schemes and using imported base material under spread footings, or possibly other methods. Assuming that all existing fill soils are completely removed and properly recompacted, mitigation measures for potential seismic settlement are implemented as described in Section 3.1.4 below the proposed building, and geotechnical recommendations presented in this report are incorporated into the design by the structural engineer, dynamic differential settlement in dense sands is expected to be within acceptable limits.

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soil with a low expansion potential and considering the potential for liquefaction and seismic settlement. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the expansion index of near-surface subgrade soils. In addition, slabs-on-grade should have the following minimum recommended components:

- Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 2 percentage points above optimum moisture content to a minimum depth of 12 inches prior to placing the moisture vapor retarder, steel or concrete.

- Moisture Retarder: A minimum of 10-mil moisture retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a capillary break should be placed under the vapor retarder and whether or not a sand blotter layer should be placed over the vapor retarder. The moisture barrier may be placed directly on subgrade provided gravel or other protruding objects that could puncture the moisture retarder are removed from the subgrade prior to placement. A heavier vapor retarder (such as 15 mil Stego Wrap) placed directly on prepared subgrade may also be used. Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Institute, ASTM International, and California Building Code requirements and guidelines.

  Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

- Concrete Thickness and Reinforcement in Warehouse/Industrial Areas: Warehouse/industrial slabs-on-grade should be designed by the structural engineer based on anticipated wheel, equipment, and storage loads. Considering the site conditions, we recommend a minimum slab thickness of 6 inches. Crack control joints should be provided at a maximum spacing of 14 feet on center.

  The structural engineer should consider the following parameters. Provided that the slab subgrade soils are compacted to a minimum of 95 percent relative compaction at 2 percentage points above optimum (as measured by ASTM D 1557), an average subgrade spring constant (modulus of subgrade reaction, k) of 100 pci (with linear deflections up to ¾ inch and a non-linear response for larger deflections) may be assumed for analysis of loading on slabs-on-grade. This value should not be used for estimation of actual settlements, but is intended to estimate shears, moments, and local distortions. An alternate check may be used by assuming an allowable bearing pressure of 1,000 psf (though the modulus of subgrade reaction method is the preferred method). If soils are allowed to dry out prior to placing concrete, the upper 9 inches should be scarified, moisture conditioned to 2 percentage points above optimum moisture content, and recompacted to a minimum of 95 percent relative compaction (based on ASTM D1557) prior to placing steel or concrete.

- <u>Concrete Thickness--Office Areas</u>: Slabs-on-grade for office space should be at least 4 inches thick (this is referring to the actual minimum thickness, not the nominal thickness). Reinforcing steel should be designed by the structural engineer, but as a minimum (for conventionally reinforced, 4-inch-thick slabs) should be No. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab. Crack control joints should be provided at a maximum spacing of 15 feet on center for office areas.

Minor cracking of the concrete as it cures, due to drying and shrinkage, is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

3.4 Seismic Design Parameters

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the current CBC. The CBC seismic design parameters listed in Section 2.4.2 of this report should be considered for the seismic analysis of the subject site.

3.5 Retaining Walls

We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 5 (rear of text). Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Static Equivalent Fluid Weight (pcf)	
Condition	Level Backfill
Active	40 pcf
At-Rest	60 pcf
Passive	250 pcf (allowable) (Maximum of 3,500 psf)

The above values do not contain an appreciable factor of safety unless noted, so the structural engineer should apply the applicable factors of safety and/or load factors during design, as specified by the California Building Code.

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.35 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

For retaining walls with a retained height of more than 6 feet, an incremental seismic load applied as a uniform additive pressure of 32 pcf should be considered for a cantilever (unrestrained) wall with level backfill, and 42 pcf for a basement wall (restrained) with level backfill. This pressure is in addition to the static active earth pressures presented above. Earthquake and at-rest earth pressures need not be combined for analyses.

A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

3.6 Preliminary Pavement Design

Based on design procedures outlined in the 2017 Caltrans *Highway Design Manual* and an R-value 20 for compacted subgrade soils, preliminary flexible pavement sections were calculated for the Traffic Indices (TIs) tabulated, and are listed below:

Hot Mixed Asphalt (HMA) Pavement Sections

Assumed Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0 (automobile parking, driveways)	3.0	7.5
6.0 (truck traffic)	3.5	9.5
7.0 (roadways and heavy truck traffic)	5.0	10.0

If asphalt pavement is to be constructed prior to construction, the full pavement thickness should be placed to support heavy construction traffic.

In areas where rigid concrete pavement is planned and trucks may drive on this pavement, we recommend a minimum of 7 inches of Portland Cement Concrete (PCC) with a 28-day compressive strength of 4,000 psi over 4 inches of aggregate base placed on prepared subgrade soil (see Section **Error! Reference source not found.**). Reinforcement should be specified by the structural engineer, but should be a minimum of #3 rebar at 18 inches on center each way. The PCC pavement sections should be provided with crack-control joints spaced no more than 14 feet on center each way. If sawcuts are used, they should have a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement. We recommend that sections be as nearly square as possible.

PCC sidewalks should be at least 4 inches thick over prepared subgrade soil, with construction joints no more than 8 feet on center each way, with sections as nearly square as possible. Use of reinforcing will help reduce severity of cracking.

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled. Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 95 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

3.7 Infiltration Recommendations

In general, our geotechnical exploration encountered layers of sediment deposits that appear relatively continuous between our borings with varying soil types between these layers. In general, we found native alluvium just below undocumented artificial fill and to a depth of approximately 15 feet bgs to consist of sandy clay (CL), clayey sand (SC), and silty sand (SM), in our borings with a lens of poorly graded sand (SP) in the western portion of the site. At depths of about 15 feet bgs to 20 feet bgs, we encountered a layer of clayey sand (CL) with minor amounts of sandy silty clay (CL). Below approximately 20 feet bgs, we encountered a layer of silty sand (SM) about 5 feet thick. A layer of sandy clay (CL) and lean clay (CL) was encountered in Borings LB-1 and IT-2 at a depth of about 25 feet bgs. At our test locations, sieve analysis tests performed on soil samples from the infiltration test zones. In the field, the soils at depths between 5 feet and 10 feet bgs in boring IT-1, located in the southwestern portion of the property, were estimated in the field to have a percent fines (% silt and clay) ranging from 15 to 30 percent; and the soils at depths between 20 feet and 25 feet bgs in boring IT-2, located in the southwestern portion of the property, were estimated in the field to have a percent fines ranging from 5 to 25 percent.

Based on our infiltration testing and field observations, raw infiltration rates have been estimated in soils between 5 and 10 feet bgs to be 0.8 in./ hr. After applying factors of safety and considering that Los Angeles County guidelines (Los Angeles

County, 2021) indicate that design infiltration rated should be 0.3 in./hr or greater for proposed infiltration systems, infiltration does not appear to be feasible onsite at depths of 5 to 10 feet bgs. However, infiltration does appear to be feasible at depths between 20 and 25 feet bgs based on results from test IT-2. For planning purposes, a raw infiltration rate of 1.0 inch per hour (prior to application of a correction factor) may be assumed for infiltration system design at depths between 20 and 25 feet bgs. However, as site layout and infiltration system design becomes available, supplemental infiltration testing should be performed at locations and invert depths to further refine soil infiltration characteristics to support design.

We recommend that a correction factor/safety factor be applied to the infiltration rate in conformance with Los Angeles County guidelines, since monitoring of facility performance has shown that actual infiltration rates are lower than measured from small-scale tests. Infiltration basins are subject to siltation, which can result in reduced infiltration rates. *This small-scale infiltration rate should be divided by a design factor of at least 3 for buried chambers and at least 4 for open basins; although the design/safety factor may be higher based on project-specific aspects.* It should be noted that during periods of prolonged precipitation, underlying soils tend to become saturated to greater depths/extent. Therefore, infiltration rates tend to decrease with prolonged rainfall.

Some design considerations are presented in the following paragraphs:

- **Adjacent Structure Impact**: As infiltrating water can seep within soil strata partially horizontally, it is important to consider impact that infiltration facilities can play on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. Any such nearby features should be identified and evaluated as to whether infiltrating water can impact these facilities. Infiltration facilities should not be constructed adjacent to or under buildings. Setbacks should be discussed with Leighton during the planning process, but a building setback of at least 15 feet horizontally is initially suggested.

- **Infiltration Basins Type and Geometry**: Further testing may be required depending on final design of infiltration facilities. Infiltration rates are anticipated to vary based on location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. We should review all infiltration plans, including locations and depths of proposed facilities. Further testing may be required depending on infiltration facilities design details, particularly considering type, depth and location.

- **Siltation and Soil Changes**: These infiltration rates are for a clean, un-silted infiltration surface in native, sandy alluvial soil. These values may be reduced over time as silting of the basin or chamber occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of soil particles, gradation (uniform versus well graded), particle shape, fines content and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill. For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

- **De-silting Weir/Facilities**: Periodic flow of water carrying sediments into the basin or chamber, plus deposition of fine wind-blown sediments and sediments from erosion of basin side walls, will eventually cause the basin bottom or chamber to accumulate a layer of silt, which has the potential to significantly reducing the overall infiltration rate of the basin or chamber. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within stormwater, especially during construction of the project and prior to achieving a mature landscape onsite. We recommend that an easily maintained, robust silt/sediment removal system be installed to pretreat storm water before it enters the infiltration facility. Infiltration facilities should be constructed with spillways or other appropriate means that would prevent overfilling that could damage the facility or adjacent improvements.

- **Drainage/Infiltration Time Cycle**: In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with prolonged periods of infiltration. As such, water typically infiltrates much faster near the beginning of and/or immediately after storm events than at times well after a storm when the water level in the facility has receded, since the infiltration rate is then slower due to both lower head and longer overall duration of infiltration. In open basins with compacted or silty bottoms, this could be problematic, in that even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time, creating prolonged open-water safety concern (such as potential for mosquitos and waterborne diseases, algae odor, etc.). In a buried/cover infiltration chamber, these conditions would be of less concern.

- **Maintenance**: Infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented if and as needed. Things to check for include removal of trash or dumping, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained as recommended by the manufacturer or designer. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed.

3.8 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Cantilever shoring should be designed based on an active equivalent fluid pressure of 40 pcf. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA, standards to evaluate soil conditions.

Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.9 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1 and 306-6 of the Standard Specifications for Public Works Construction, (“Greenbook”), latest edition. Utility trenches can be backfilled with onsite sandy material free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Sand:** A uniform, sand material that has a Sand Equivalent (SE) greater than or equal to 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer), placed and densified in accordance with the Greenbook, or

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction,* (“Greenbook”), latest edition.

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. Native and clean fill soils can be used as backfill over the pipe bedding zone, and should be placed in thin lifts, moisture conditioned above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

Gravel or rock should not be used for trench backfill without written approval by Leighton. If gravel or open-graded rock is approved and used as bedding or shading, it should be wrapped in Mirafi 140N filter fabric, or approved equivalent, to prevent surrounding soil from washing into the pore spaces in the gap-graded rock and causing settlement or voids at the surface or under structures.

Subsequent to pipe bedding and shading, backfill soils should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction (ASTMS D1557). The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction

(Greenbook). The upper 6 inches in pavement areas should be compacted to 95 percent compaction.

3.10 Geogrid Installation

All field handling and installation procedures should be performed in accordance with manufacturer's guidelines with a particular focus to ensure proper overlap between adjacent sheets.

The placement of fill soils to finish grade should include certain procedures and precautions to protect the geogrid reinforcement and achieve proper recompaction. Fill placement and compaction is recommend to comply with the following:

- Backfill material should be placed in thin lifts (4- to 6-inch thick) and compacted to a minimum of 95 percent per ASTM D1557. Actual lift thickness should be consistent with the equipment used for compaction.
- Backfill should be placed, spread and compacted in such a manner that minimizes the development of wrinkles/bends in and/or movement of the geogrid reinforcement.
- Care should be taken by the grading contractor that the fill soils and the grading equipment does not damage the integrity of the geogrid reinforcement during the construction process.
- Tracked construction equipment should not be operated directly upon the geogrid reinforcement. A minimum thickness of six (6) inches is required prior to operation of tracked vehicles over the geotextile reinforcement fabric. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and damaging the geogrid reinforcement.
- Rubber-tired equipment may pass over geogrid reinforcement at slow speeds, less than 5 mph. Sudden braking and sharp turning should be avoided.

If future excavations (such as utility trenching) will penetrate through the installed geogrid layer, then a cut geogrid section should be placed at the bottom of the utility trench extending the width of the existing geogrid. If both layers of geogrid are trenched through, a second layer should be placed close to the elevation of the adjacent top geogrid layer. Additional recommendations and considerations are provided within the Tensar TriAx Geogrid Installation Guide.

3.11 Surface Drainage

Inadequate control of runoff water and/or poorly controlled irrigation can cause the onsite soils to expand and/or shrink, producing heaving and/or settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems.

Positive surface drainage should be designed to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes.

Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the building. We recommend that unpaved landscaped areas adjacent to the buildings be avoided. Roof runoff should be carried to suitable drainage outlets by watertight drain pipes or over paved areas.

3.12 Sulfate Attack and Corrosion Protection

Based on our observations and experience in the area, concrete structures in contact with the onsite soil are anticipated to have negligible exposure to water-soluble sulfates in the soil. Therefore, common Type II cement may be used for concrete construction. The concrete should be designed in accordance with Table 19.3.2.1 of the American Concrete Institute ACI 318-14 provisions (ACI, 2014). We will update our assessment of corrosion potential of onsite soils once results of laboratory testing become available.

The onsite soil are anticipated to be corrosive to ferrous metals. We recommend that buried pipe be made of non-ferrous material, or that ferrous pipe be protected by dielectric tape, polyethylene sleeves and/or other methods, with recommendations from a corrosion engineer. Corrosion information presented in this report should be provided to your underground utility subcontractors. Additional testing and evaluation by a corrosion engineer may be warranted if metallic utilities are planned.

3.13 Additional Geotechnical Services

The preliminary geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our supplemental geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided:

- After completion of site clearing.
- During overexcavation of compressible soil.
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was based in part on data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton Consulting, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of CapRock Acquisitions III, LLC, for application to the design of the proposed warehouse building development in accordance with generally accepted geotechnical engineering practices at this time in California.

See the GBA insert on the following page for important information about this geotechnical engineering report.

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

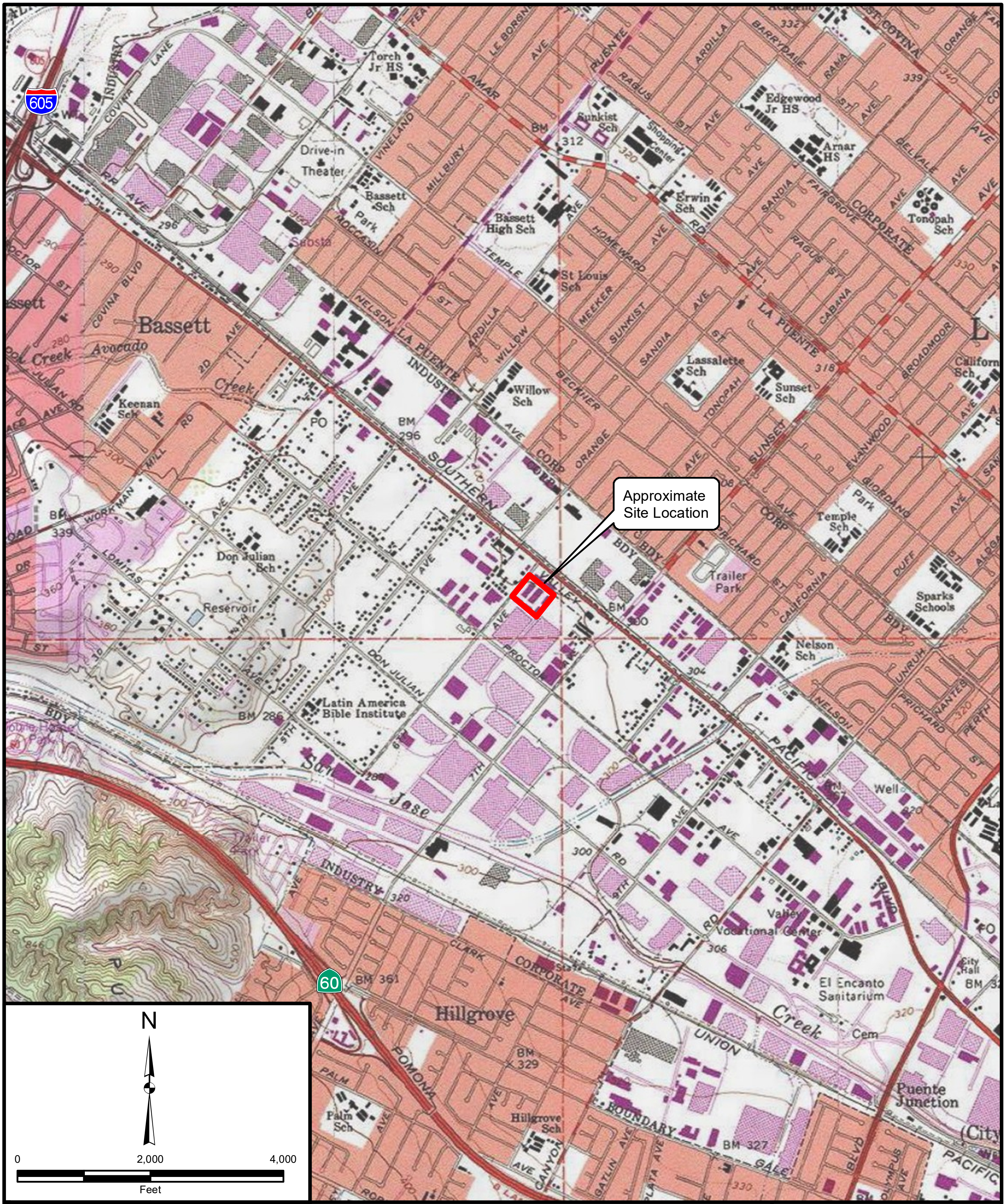
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org



Project: 13877.001	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: May 2023

Reference: Copyright:© 2013 National Geographic Society, i-cubed

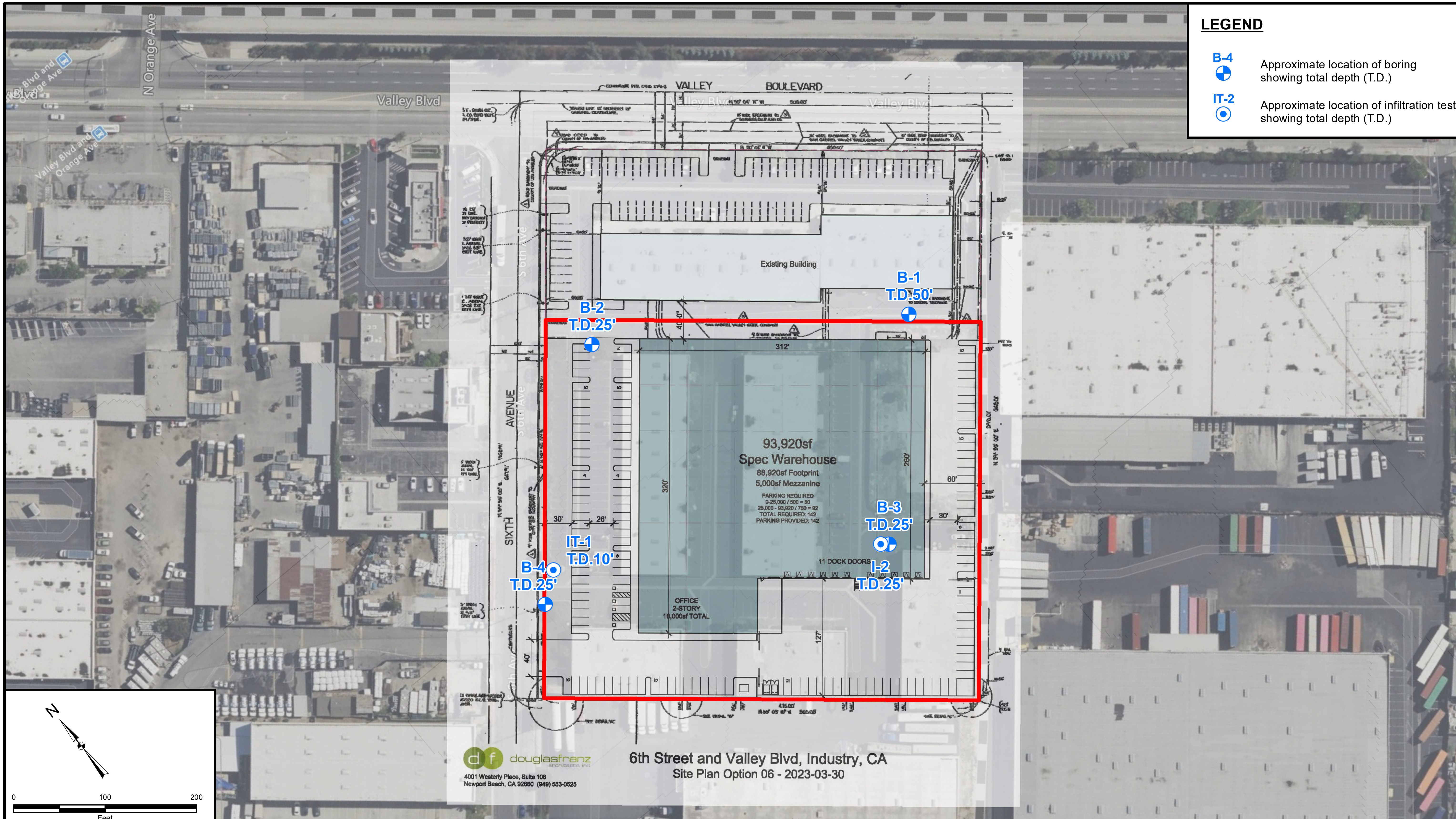
SITE LOCATION MAP

Proposed Industrial Warehouse
6th Avenue and Valley Boulevard
City of Industry, California

FIGURE 1

Leighton

Map Saved as J:\Drafting\13877\001\Maps\13877-001_F01_SLM_2023-04-25.mxd on 4/25/2023 9:58:13 AM Author: KVM (btran)



Project: 13877.001	Eng/Geol: JDH/SGO
Scale: 1 " = 100 '	Date: May 2023
Reference: As Shown	

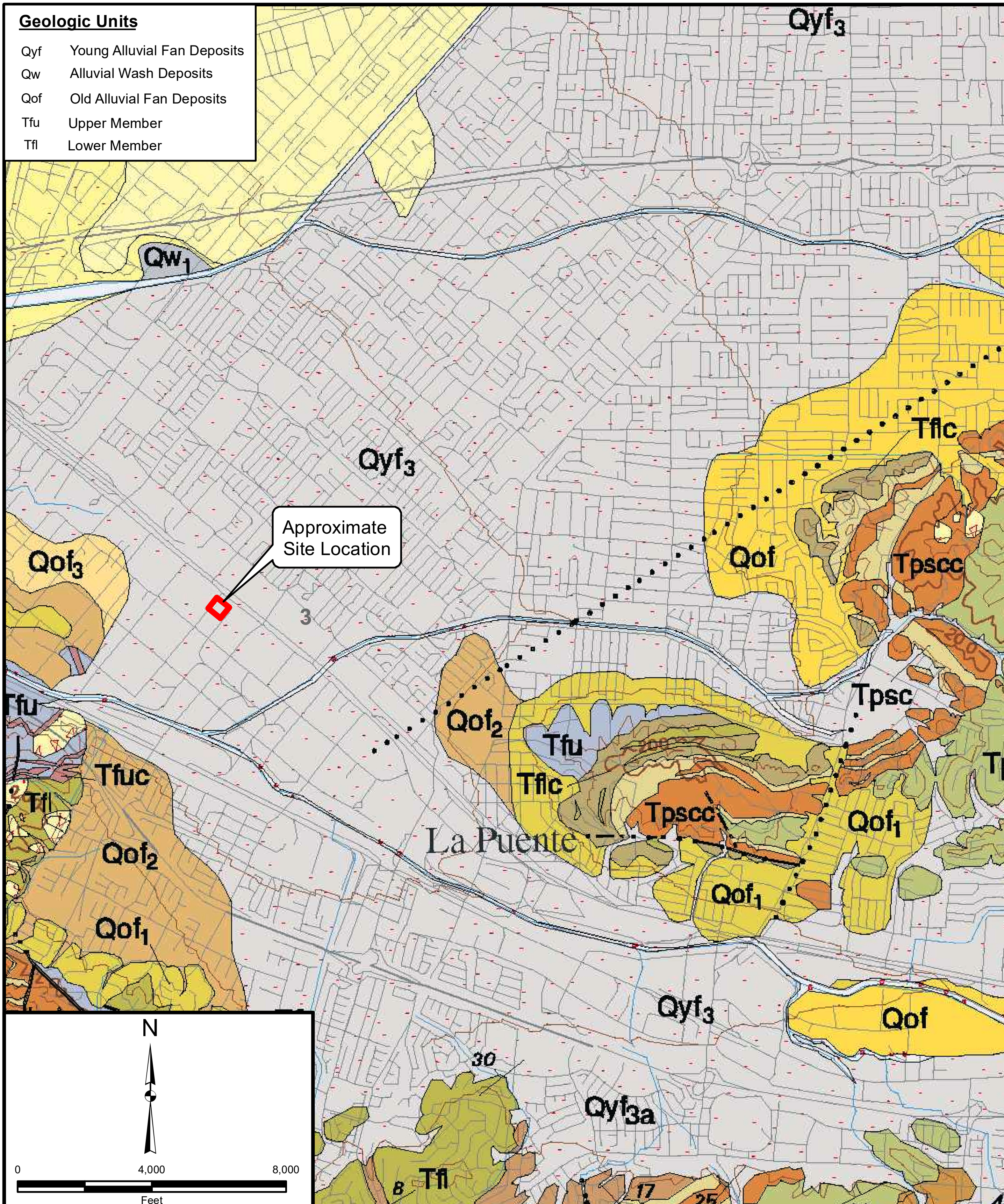
EXPLORATION LOCATION MAP

Proposed Industrial Warehouse
6th Avenue and Valley Boulevard
City of Industry, California

FIGURE 2

Leighton

Map Saved as J:\Drafting\13877\001\Maps\13877-001_F02_ELM_2023-04-25.mxd on 4/25/2023 12:27:22 PM Author: KVM (btran)



Project: 13877.001	Eng/Geol: JDH/SGO
Scale: 1 " = 4,000 '	Date: May 2023

Reference: Geologic Map of The San Bernardino and Santa Ana Quadrangles, California by Douglas M. Morton and Fred K. Miller, 2006

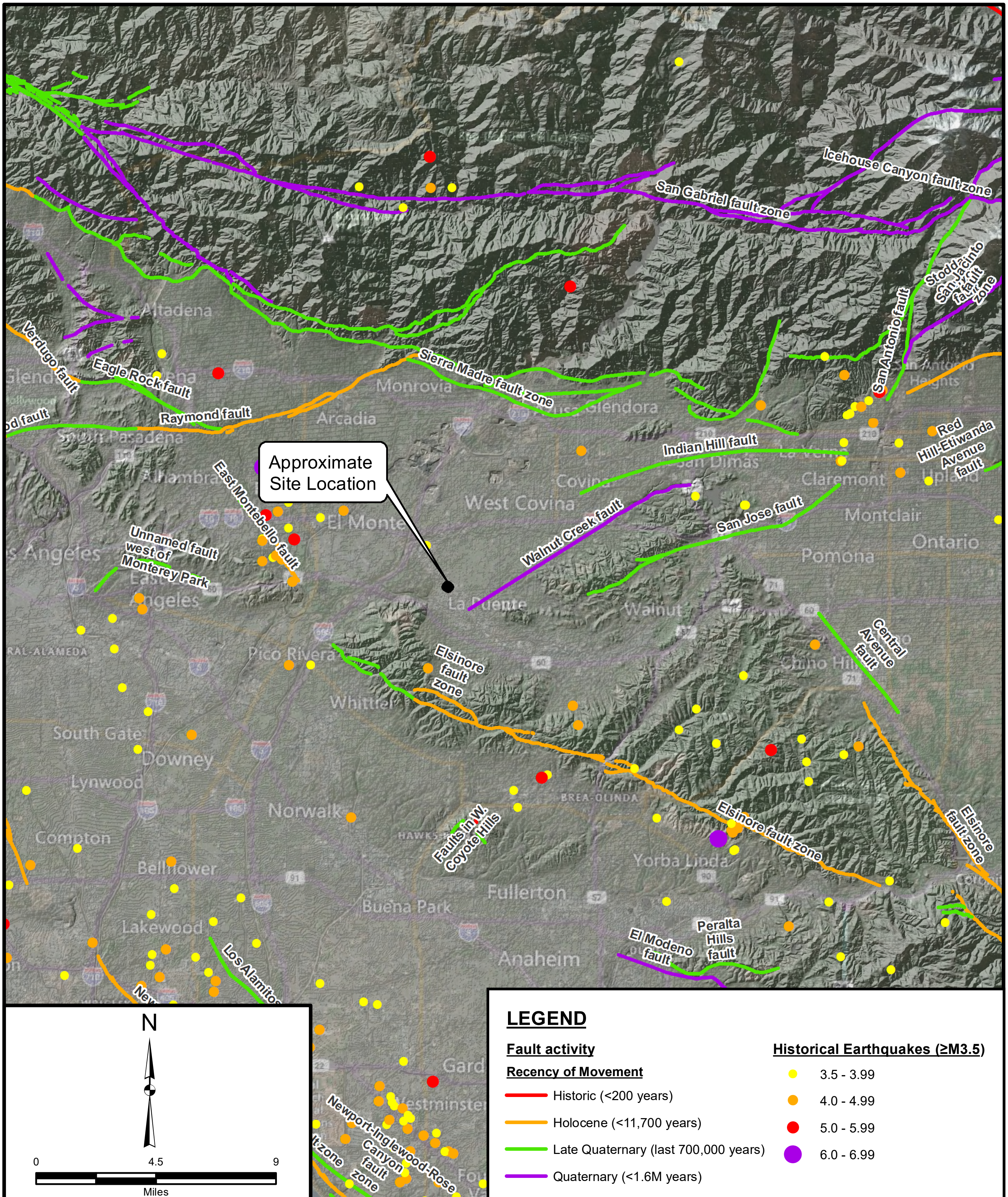
REGIONAL GEOLOGY MAP

Proposed Industrial Warehouse
6th Avenue and Valley Boulevard
City of Industry, California

FIGURE 3

Leighton

Map Saved as J:\Drafting\13877\001\Maps\13877-001_F03_RGM_2023-04-25.mxd on 4/25/2023 10:20:21 AM(btran)



Project: 13877.001	Eng/Geol: JDH/SGO	**REGIONAL FAULT AND HISTORIC SEISMICITY MAP** Proposed Industrial Warehouse 6th Avenue and Valley Boulevard City of Industry, California	**FIGURE 4**
Scale: 1 " = 5 miles	Date: May 2023		Leighton

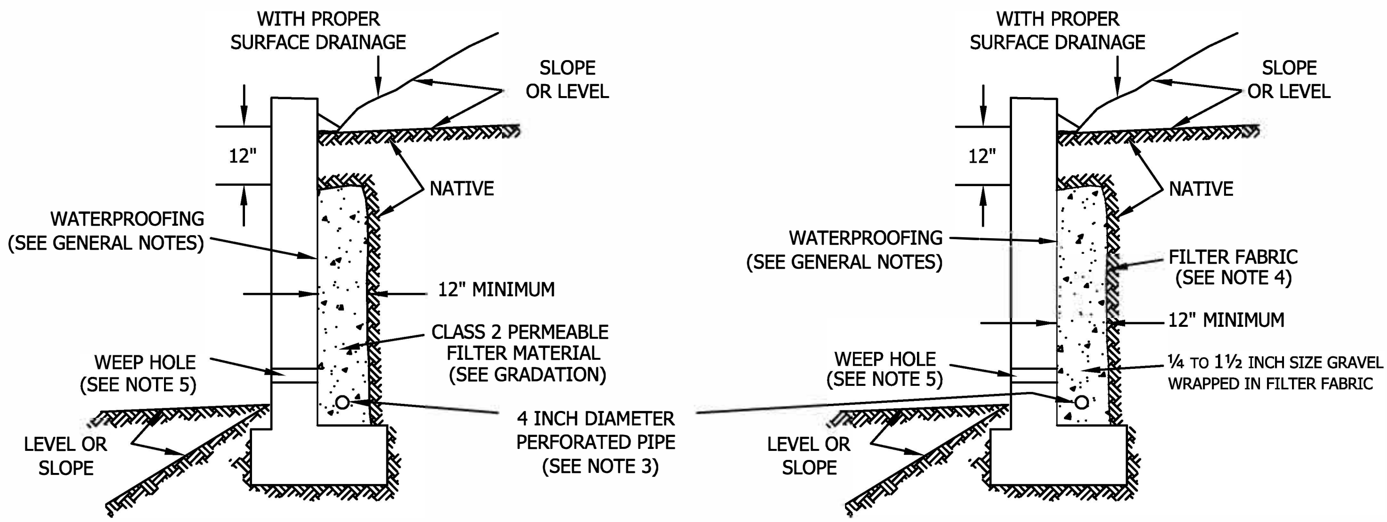
Basemap Reference: © 2023 Microsoft Corporation
Earthstar Geographics SIO © 2023 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

Map Saved as J:\Drafting\13877\001\Maps\13877-001_F04_RFHSM_2023-04-25.mxd on 4/25/2023 10:22:58 AM Author: KVM (btran)

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum

*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)

*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:

1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 5

P:Drafting\templates\details\retain-wall-backfill-and subdrain.dwg (7/00)

APPENDIX A

REFERENCES

APPENDIX A

References

American Concrete Institute (ACE), 2014, Building Code Requirements for Structural Concrete (ACE 318-14) and Commentary (ACE 318R-14), an ACE Standard.

California Building Standards Commission, 2019 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 1 and 2 of 2, Based on 2018 International Building Code, Effective January 1, 2020.

California Department of Water Resources (CDWR), 2023, California Statewide Groundwater Elevation Monitoring (CASGEM).

California Geologic Survey (CGS), 1999, Earthquake Zones of Required Investigation, Baldwin Park Quadrangle, Released: March 25, 1999, scale 1:24,000.

California Geologic Survey (CGS), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, Revised and Re-Adopted on September 11, 2008, Laguna Beach, California.

Los Angeles County, 2021, Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration, GS200.1, June 30, 2021.

Main San Gabriel Basin Watermaster, Overview.

Main San Gabriel Basin Watermaster, 2021-2022 Annual Report.

Martin, G. R., and Lew, M., ed., 1999, “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California,” Southern California Earthquake Center, dated March 1999.

Office of Statewide Health Planning and Development (OSHPD) and Structural Engineers Association of California (SEAOC), 2020, Seismic Design Maps web tool, <https://seismicmaps.org/>.

Public Works Standard, Inc., 2018, Greenbook, Standard Specifications for Public Works Construction: BNI Building News, Anaheim, California.

Tan, S.S., 2000, Geologic Map of the Baldwin Park 7.5 Quadrangle, Los Angeles County, California, California Division of Mines and Geology Open-File Report OFR-98-30, scale 1:24,000.

Tokimatsu, K., Seed, H. B., 1987, “Evaluation of Settlements in Sands Due to Earthquake Shaking,” *Journal of the Geotechnical Engineering*, American Society of Civil Engineers, Vol. 113, No. 8, pp. 861-878.

United States Geological Survey (USGS), 2011, Ground Motion Parameter Calculator, Seismic Hazard Curves and Uniform Hazard Response Spectrum, Java Application, Version 5.1.0, February 10, 2011, downloaded from http://earthquake.usgs.gov /hazards/designmaps/ javacalc.php

United States Geological Survey (USGS), 2023, Areas of Land Subsidence in California, website https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html, accessed September 1, 2022.

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E., and Vedder, J.G., 1965, Geology of the Los Angeles Basin California - an Introduction: U.S. Geological Survey Profession Paper 420-A, 57 p.

Youd, T. L., Hanson C. M., and Bartlett, S. F., 1999, Revised MLR Equations for Predicting Lateral Spread Displacement, Proceedings of the 7th U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, November 19, 1999, pp. 99-114.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.C., Marcuson, W.F. III, Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., Stokoe, K.H. II, 2001, “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils”, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001.

APPENDIX B

GEOTECHNICAL LOGS

APPENDIX B

FIELD EXPLORATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration. Approximate exploration locations are shown on Figure 2, *Geotechnical Map*.

Borings: On September 15 and 16, 2022, 12 hollow-stem-auger borings (LB-1 through LB-10, LI-1 and LI-2) were drilled, logged and sampled to depths ranging from 20 feet to 30 feet below the ground surface. Encountered soils were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Relatively undisturbed soil samples were obtained at selected intervals within these borings using both a Modified California ring-lined and Standard Penetration Test (SPT) split-spoon sampler. Standard Penetration Test (SPT) resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch outside diameter split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D 1586). In addition, 2.4-inch inside diameter brass ring samples were obtained using a Modified California sampler driven into the soil with the 140-pound hammer. Near surface bulk soil samples were also collected from the borings. Representative earth-material samples obtained from these subsurface explorations were transported to our geotechnical laboratory for evaluation and appropriate testing.

GEOTECHNICAL BORING LOG LB-1

Project No.	13877.001	**Date Drilled**	4-13-23
Project	CapRock 6th Ave Industry Warehouse	**Logged By**	BTM
Drilling Co.	Martini Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	290'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
290	0								**Artificial Fill, undocumented (Afu)** @Surface: 4-inches Asphalt Concrete over 6-inches Base	
				B-1				SC	@0.83': Clayey SAND, gray and brown, moist, low plasticity	
				R-1	9 16 19	113	15		@2.5': Clayey SAND, medium dense, dark gray, moist, fine sand, low plasticity, brick and asphalt bits	
285	5			R-2	5 8 10	106	14	SM	@5': Silty SAND, medium dense, olive brown, moist, fine sand, trace gravel and construction debris	
				R-3	4 4 8	102	16	SM	**Young Alluvial Fan Deposits (Qyf_3)** @7.5': Silty SAND, loose, olive brown, moist, fine sand, low plasticity	CO
280	10			R-4	7 9 13	111	3			-200
								SP	@11': Poorly-graded SAND, medium dense, tan, moist, fine sand, trace medium to coarse sand, trace silt, slightly oxidized, friable, 36% fines.	
275	15			S-5	2 4 5			SC	@15': Clayey SAND, loose, olive brown, moist, fine to medium sand, 30% fines, low plasticity, trace coarse sand	-200
270	20			R-6	5 9 13	97	11	SM	@20': Silty SAND, medium dense, olive brown, moist, fine sand, nonplastic, slightly oxidized, micaceous	
265	25			S-7	2 3 4			SC	@25': Sandy CLAY, medium stiff, olive brown and gray, moist to very moist, low plasticity, laminated, pockets of higher sand content, 78% fines	-200, AL
260	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-1

Project No.	13877.001	**Date Drilled**	4-13-23
Project	CapRock 6th Ave Industry Warehouse	**Logged By**	BTM
Drilling Co.	Martini Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	290'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
260	30			R-8	15 30 39	111	2	SP	@30': Poorly-graded SAND, dense, tan, slightly moist, fine to coarse sand, some very fine granitic gravel, friable	
255	35			S-9	8 14 18				@35': Poorly-graded SAND, dense, tan, slightly moist, fine to coarse sand, some fine granitic gravel, friable	
250	40			R-10	6 8 13	91	31	SM-ML	@40': Silty SAND to Sandy SILT, medium dense to stiff, brown and orange, moist, fine sand, 45% fines, heavily oxidized, heavily micaceous, slightly laminated	-200, AL
245	45			S-11	3 4 5			SC	@45': Sandy Lean CLAY, stiff, dark brown, fine sand, 57% fines, low plasticity, oxidation veins, slightly micaceous	-200, AL
240	50			R-12	9 17 24	98	12	SM	@50': Silty SAND, medium dense, medium brown, moist, fine sand, 18% fines, some oxidation staining, slightly micaceous, friable	-200
235	55								**Total Depth: 51.5' bgs** **No groundwater encountered during drilling** **Boring backfilled with soil cuttings and patched at surface with black-dyed concrete.**	
230	60									

SAMPLE TYPES:
- **B** BULK SAMPLE
- **C** CORE SAMPLE
- **G** GRAB SAMPLE
- **R** RING SAMPLE
- **S** SPLIT SPOON SAMPLE
- **T** TUBE SAMPLE

TYPE OF TESTS:
- **-200** % FINES PASSING
- **AL** ATTERBERG LIMITS
- **CN** CONSOLIDATION
- **CO** COLLAPSE
- **CR** CORROSION
- **CU** UNDRAINED TRIAXIAL
- **DS** DIRECT SHEAR
- **EI** EXPANSION INDEX
- **H** HYDROMETER
- **MD** MAXIMUM DENSITY
- **PP** POCKET PENETROMETER
- **RV** R VALUE
- **SA** SIEVE ANALYSIS
- **SE** SAND EQUIVALENT
- **SG** SPECIFIC GRAVITY
- **UC** UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-2

Project No.	13877.001	**Date Drilled**	4-13-23
Project	CapRock 6th Ave Industry Warehouse	**Logged By**	BTM
Drilling Co.	Martini Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	290'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
290	0			B-1				SC	**Artificial Fill, undocumented (Afu)** @Surface: Grass over Clayey SAND, dark brown, moist, fine to medium sand, low plasticity	
				R-1	5 5 5	105	10	SM	@2.5': Silty SAND, loose, brown, moist, fine sand, few medium to coarse sand, low plasticity, rootlets	
285	5			R-2	1 4 3	90	13		@5': Silty SAND, loose, brown, very moist, fine sand, few medium to coarse sand, low plasticity, rootlets	
				R-3	5 11 17	112	13	SM	**Young Alluvial Fan Deposits (Qyf_3)** @7.5': Silty SAND, medium dense, brown, moist, mostly fine sand, few medium to coarse sand, low plasticity, trace fine gravel	
280	10			R-4	5 12 19	106	15	SP	@10': Poorly-graded SAND, medium dense, light brown, moist, fine sand, trace medium sand, trace silt, friable	
275	15			R-5	5 7 13	112	16	SC	@15': Clayey SAND, medium dense, olive brown, moist to very moist, fine sand, trace medium to coarse sand, low plasticity, slightly micaceous	
270	20			S-6	4 6 8			SM	@20': Silty SAND, medium dense, olive brown, moist, fine sand, low to no plasticity, trace medium to coarse sand	
265	25			R-7	4 12 19	112	16		@25': Silty SAND, medium dense, olive brown, moist, fine sand, low plasticity, oxidation staining, slightly micaceous, friable	
									Total Depth: 26.5' bgs **No groundwater encountered during drilling** **Boring backfilled with soil cuttings to surfce.**	
260	30									

SAMPLE TYPES:
- **B** BULK SAMPLE
- **C** CORE SAMPLE
- **G** GRAB SAMPLE
- **R** RING SAMPLE
- **S** SPLIT SPOON SAMPLE
- **T** TUBE SAMPLE

TYPE OF TESTS:
- **-200** % FINES PASSING
- **AL** ATTERBERG LIMITS
- **CN** CONSOLIDATION
- **CO** COLLAPSE
- **CR** CORROSION
- **CU** UNDRAINED TRIAXIAL
- **DS** DIRECT SHEAR
- **EI** EXPANSION INDEX
- **H** HYDROMETER
- **MD** MAXIMUM DENSITY
- **PP** POCKET PENETROMETER
- **RV** R VALUE
- **SA** SIEVE ANALYSIS
- **SE** SAND EQUIVALENT
- **SG** SPECIFIC GRAVITY
- **UC** UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-3

Project No.	13877.001	**Date Drilled**	4-13-23
Project	CapRock 6th Ave Industry Warehouse	**Logged By**	BTM
Drilling Co.	Martini Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	290'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
290	0			B-1				SC	**Artificial Fill, undocumented (Afu)** @Surface: Grass over Clayey SAND, dark brown, moist, fine sand, 45% fines (field estimate), low plasticity, rootlets	-200, AL, MD
								ML	@2':Silty SAND, tan to olive brown, moist, fine sand, rootlets, trace construction debris	CR, EI
285	5			R-1	3 6 9	104	19	CL	**Young Alluvial Fan Deposits (Qyf_3)** @5': Sandy CLAY, stiff, gray brown, moist, fine sand, low plasticity, trace rootlets, pinhole pores, massive	
				R-2	4 11 14	111	16	SC-SM	@7.5': Clayey Silty SAND, medium dense, olive brown, moist, fine sand, low plasticity, slightly micaceous, friable	
280	10			R-3	5 11 19	112	17	SM	@10': Silty SAND, medium dense, olive brown, moist, fine sand, low plasticity, slightly micaceous, friable	
275	15			R-4	5 10 15	118	15	SC	@15': Clayey SAND, medium dense, brown to dark brown, moist, fine to medium sand, low plasticity, slightly micaceous	
270	20			S-5	3 4 6			SM	@20': Silty SAND, loose, tan to olive brown, moist, fine sand, low plasticity, slightly micaceous	
265	25			R-6	9 10 9	90	5		@25': Silty SAND, medium dense, tan to olive brown, moist, fine sand, nonplastic, slightly micaceous, some oxidation staining	
									Total Depth: 26.5' bgs **No groundwater encountered during drilling** **Boring backfilled with soil cuttings to surface.**	
260	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-4

Project No.	13877.001	**Date Drilled**	4-13-23
Project	CapRock 6th Ave Industry Warehouse	**Logged By**	BTM
Drilling Co.	Martini Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	289'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SC	**Artificial Fill, undocumented (Afu)** @Surface: Grass over Clayey SAND, dark brown, moist, fine sand, low plasticity, rootlets, trace fine gravel	
				R-1	5 6 7	106	10		@2.5': Clayey SAND, loose, very dark brown, moist, fine sand, few medium sand, low plasticity, trace rootlets	
285										
	5			R-2	5 10 11	103	13	SM	**Young Alluvial Fan Depostis (Qyf$_3$)** @5': Silty SAND, medium dense, olive brown, moist, mostly fine sand, few medium to coarse sand, low plasticity, trace rootlets, trace fine gravel	
				R-3	6 11 19	113	9		@7.5': Silty SAND, medium dense, olive brown, moist, mostly fine sand, few medium to coarse sand, low plasticity, slight oxidation staining	
280										
	10			R-4	9 10 17	106	17	SP	@10': Poorly-graded SAND, medium dense, tan, moist, mostly fine sand, few medium to coarse sand, trace fine gravel, friable, 49% fines	-200
275										
	15			S-5	1 2 3			CL	@15': Sandy Silty CLAY, medium stiff, olive brown, moist, fine sand, 54% fines, low plasticity, massive, slightly oxidized	-200
								SC	@16': Clayey SAND, loose, olive brown, moist, fine sand, , slightly laminated, slightly micaceous, friable	
270										
	20			R-6	4 8 14	96	19	SM	@20': Silty SAND, medium dense, olive brown, moist, fine sand, low plasticity; grading coarser toward bottom of sample, oxidation staining	
								SP-SM		
265										
	25			S-7	5 6 11				@25': Poorly-graded SAND with silt to Silty SAND, medium dense, tan, moist, fine sand, slightly micaceous, friable	
260									**Total Depth: 26.5' bgs** **No groundwater encountered during drilling** **Boring backfilled with soil cuttings to surface**	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG IT-1

Project No.	13877.001	**Date Drilled**	4-13-23
Project	CapRock 6th Ave Industry Warehouse	**Logged By**	BTM
Drilling Co.	Martini Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	289'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
289	0							SC	**Artificial Fill, undocumented (Afu)** @Surface: Grass over Clayey SAND, dark brown, moist, fine sand, 25-35% fines (field estimate), low plasticity, trace fine gravel, rootlets	
285										
	5			S-1	4 4 6			SC	**Young Alluvial Fan Deposits (Qyf_3)** @5': Clayey SAND, loose, olive brown, moist, fine sand, low plasticity, trace rootlets, few medium to coarse sand	
280				S-2	5 6 7			SM	@8.5: Silty SAND, medium dense, olive brown, moist, fine sand, trace medium to coarse sand, 25% fines, slight oxidation staining, slightly micaceous	-200
	10								**Total Depth: 10' bgs** **No groundwater encountered during drilling** **Temporary percolation well installed; screened 5-10' bgs** **Upon completion of infiltration testing, boring backfilled with soil cuttings to surface**	
275										
	15									
270										
	20									
265										
	25									
260										
	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

GEOTECHNICAL BORING LOG IT-2

Project No.	13877.001	**Date Drilled**	4-13-23
Project	CapRock 6th Ave Industry Warehouse	**Logged By**	BTM
Drilling Co.	Martini Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	290'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
290	0									
285	5									
280	10									
275	15									
270	20				2 4 5			SM	**Young Alluvial Fan Deposits (Qyf_3)** @20': Silty SAND, loose, olive brown, moist, fine sand, low plasticity, slightly micaceous	
					4 6 5			SP-SM	@23.5': Silty SAND to Poorly-graded SAND with silt, medium dense, tan to olive brown, moist, fine sand, few medium sand, 6% fines	-200
265	25							CL	@25': Lean CLAY, olive brown, moist, low plasticity, laminated, oxidation staining	
									Total Depth: 26.5' bgs **No groundwater encountered during drilling** **Temporary percolation well installed; screened 15-25' bgs** **Upon completion of infiltration testing, boring backfilled with soil cuttings to surface.**	
260	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

Leighton

Results of Falling Head Infiltration Test

Project: 13877.001

Exploration #/Location: *IT-1*
Depth Boring drilled, bgs (ft): 10
Tested by: BTM
USCS Soil Type in test zone: SM
Weather (start to finish): Overcast
Water Source/pH: H2O
Measured boring diameter: 8 in. 4 in. Well Radius
Depth to GW or aquitard, bgs: 100 ft
Well Prep: Drilled to 10', place sand at bottom, slotted 2" pipe for bottom 5 feet, sand around slotted section

Initial estimated Depth to Water Surface (in.): 85
Average depth of water in well, "h" (in.): 35
approx. h/r: 8.7
Tu (Fig. 8) (ft): 92.9
Tu>3h?: yes, OK

Cross-sectional area for flow calcs based on Δh
Well pack sand porosity: 0.3
Casing outer diameter, in.: 2.3
Casing inner diameter, in.: 2.1
Cross-sectional area, in.^2: 17.3

Use of Barrels: No
Use of Flow Meter: No
Test Type: Falling Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surf	10.2 ft	0. in.	122
Casing stickup measured above top of auger (or ground surface) (+	0. ft	3. in.	3
Depth to top of sand from top of casing	4. ft	2. in.	

Depth of well bottom below top of casing (in): 125

Field Data / **Calculations**

Date	Time	Data from Flow Meter: Reading (gallons)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of casing) ft	in.	Water Temp (deg F)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	from Δh	Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date	Start time:	Gallons																				
4/13/2023	8:10																					
4/13/23	8:16			6.81					6	78.7	43.7											
4/13/23	8:19			7.12				3	9	82.4	40.0	-3.72	42	0	64	64	21	1287		0.9	0.25	1.08
4/13/23	8:22			7.49				3	12	86.9	35.5	-4.44	38	0	77	77	26	1536		0.9	0.36	1.42
4/13/23	8:25			7.75				3	15	90.0	32.4	-3.12	34	0	54	54	18	1079		0.9	0.29	1.10
4/13/23	8:28			7.99				3	18	92.9	29.5	-2.88	31	0	50	50	17	996		0.9	0.31	1.11
4/13/23	8:31			8.22				3	21	95.6	26.8	-2.76	28	0	48	48	16	955		0.9	0.35	1.16
4/13/23	8:34			8.43				3	24	98.2	24.2	-2.52	26	0	44	44	15	872		0.9	0.37	1.16
4/13/23	8:37			8.58				3	27	100.0	22.4	-1.8	23	0	31	31	10	623		0.9	0.29	0.90
4/13/23	8:40			8.72				3	30	101.6	20.8	-1.68	22	0	29	29	10	581		0.9	0.31	0.90
4/13/23	8:58			5.5					48	63.0	59.4											
4/13/23	9:00			5.89				2	50	67.7	54.7	-4.68	57	0	81	81	40	2428		0.9	0.29	1.51
4/13/23	9:02			6.14				2	52	70.7	51.7	-3	53	0	52	52	26	1557		0.9	0.20	1.03
4/13/23	9:04			6.31				2	54	72.7	49.7	-2.04	51	0	35	35	18	1059		0.9	0.15	0.74
4/13/23	9:06			6.59				2	56	76.1	46.3	-3.36	48	0	58	58	29	1744		0.9	0.27	1.28
4/13/23	9:08			6.78				2	58	78.4	44.0	-2.28	45	0	39	39	20	1183		0.9	0.20	0.92
4/13/23	9:10			6.95				2	60	80.4	42.0	-2.04	43	0	35	35	18	1059		0.9	0.19	0.86
4/13/23	9:15			7.41				5	65	85.9	36.5	-5.52	39	0	95	95	19	1146		0.9	0.26	1.02
4/13/23	9:20			7.77				5	70	90.2	32.2	-4.32	34	0	75	75	15	897		0.9	0.25	0.91
4/13/23	9:25			8.12				5	75	94.4	28.0	-4.2	30	0	73	73	15	872		0.9	0.30	1.00
4/13/23	9:30			8.45				5	80	98.4	24.0	-3.96	26	0	68	68	14	822		0.9	0.36	1.08
4/13/23	9:35			8.69				5	85	101.3	21.1	-2.88	23	0	50	50	10	598		0.9	0.31	0.89
4/13/23	9:40			8.9				5	90	103.8	18.6	-2.52	20	0	44	44	9	523		0.9	0.33	0.88
4/13/23	9:45			9.1				5	95	106.2	16.2	-2.4	17	0	42	42	8	498		0.9	0.38	0.94
4/13/23	9:54			5.5					104	63.0	59.4											
4/13/23	9:59			6.13				5	109	70.6	51.8	-7.56	56	0	131	131	26	1569		0.9	0.21	1.00
4/13/23	10:04			6.66				5	114	76.9	45.5	-6.36	49	0	110	110	22	1320		0.9	0.21	0.96
4/13/23	10:09			7.09				5	119	82.1	40.3	-5.16	43	0	89	89	18	1071		0.9	0.21	0.87
4/13/23	10:14			7.5				5	124	87.0	35.4	-4.92	38	0	85	85	17	1021		0.9	0.24	0.94
4/13/23	10:19			7.85				5	129	91.2	31.2	-4.2	33	0	73	73	15	872		0.9	0.25	0.91
4/13/23	10:24			8.19				5	134	95.3	27.1	-4.08	29	0	71	71	14	847		0.9	0.31	1.00
4/13/23	10:29			8.5				5	139	99.0	23.4	-3.72	25	0	64	64	13	772		0.9	0.35	1.04
4/13/23	10:34			8.71				5	144	101.5	20.9	-2.52	22	0	44	44	9	523		0.9	0.28	0.79
4/13/23	10:39			8.92				5	149	104.0	18.4	-2.52	20	0	44	44	9	523		0.9	0.34	0.89
4/13/23	10:40			5.5					150	63.0	59.4											
4/13/23	10:45			6.16				5	155	70.9	51.5	-7.92	55	0	137	137	27	1644		0.9	0.22	1.05
4/13/23	10:50			6.67				5	160	77.0	45.4	-6.12	48	0	106	106	21	1270		0.9	0.21	0.92
4/13/23	10:55			7.11				5	165	82.3	40.1	-5.28	43	0	91	91	18	1096		0.9	0.22	0.90
4/13/23	11:00			7.52				5	170	87.2	35.2	-4.92	38	0	85	85	17	1021		0.9	0.25	0.95
4/13/23	11:05			7.89				5	175	91.7	30.7	-4.44	33	0	77	77	15	922		0.9	0.27	0.97
4/13/23	11:10			8.23				5	180	95.8	26.6	-4.08	29	0	71	71	14	847		0.9	0.31	1.01
4/13/23	11:15			8.52				5	185	99.2	23.2	-3.48	25	0	60	60	12	722		0.9	0.33	0.98
4/13/23	11:20			8.73				5	190	101.8	20.6	-2.52	22	0	44	44	9	523		0.9	0.28	0.80
4/13/23	11:25			8.95				5	195	104.4	18.0	-2.64	19	0	46	46	9	548		0.9	0.36	0.94
																				Minimum Rate:		0.74
																				Raw Rate for design, prior to application of adjustment factors:		**1.00**

Results of Falling Head Infiltration Test

Project: 13877.001

Exploration #/Location: IT-2

Depth Boring drilled, bgs (ft): 25

Tested by: BTM

USCS Soil Type in test zone: SP-SM

Weather (start to finish): Overcast

Water Source/pH: H2O

Measured boring diameter: 8 in. 4 in. Well Radius

Depth to GW or aquitard, bgs: 100 ft

Well Prep: Drilled to 25', place sand at bottom, slotted 2" pipe for bottom 10 feet, sand around slotted section

Initial estimated Depth to Water Surface (in.): 251

Average depth of water in well, "h" (in.): 52

approx. h/r: 13.1

Tu (Fig. 8) (ft): 79.1

Tu>3h?: yes, OK

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity 0.3

Casing outer diameter, in. 2.3

Casing inner diameter, in. 2.1

Cross-sectional area, in.^2 17.3

Use of Barrels: No

Use of Flow Meter: No

Test Type: Falling Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surf	25.4 ft	0. in.	305
Casing stickup measured above top of auger (or ground surface) (+	0. ft	1. in.	1
Depth to top of sand from top of casing	12. ft	4. in.	

Depth of well bottom below top of casing (in): 306

Field Data / **Calculations**

Date	Time	Data from Flow Meter: Reading (gallons)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of casing) ft	Depth to WL in Boring in.	Water Temp (deg F)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	Vol Change from Δh	Vol Change Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 4/13/2023	Start time: 12:35	Gallons																				
4/13/23	12:42	4885.78		21.27					7	254.2	50.6											
4/13/23	12:44	4886.17		21.26				2	9	254.1	50.7	0.12	51	90	-2	88	44	2640	1322	0.9	0.35	1.84
4/13/23	12:46	4886.56		21.26				2	11	254.1	50.7	0	51	90	0	90	45	2703	1324	0.9	0.36	1.88
4/13/23	12:48	4886.95		21.26				2	13	254.1	50.7	0	51	90	0	90	45	2703	1324	0.9	0.36	1.88
4/13/23	12:50	4887.34		21.25				2	15	254.0	50.8	0.12	51	90	-2	88	44	2640	1326	0.9	0.35	1.84
4/13/23	12:55	4888.29		21.2				5	20	253.4	51.4	0.6	51	219	-10	209	42	2509	1335	0.9	0.32	1.73
4/13/23	13:00	4889.26		21.11				5	25	252.3	52.5	1.08	52	224	-19	205	41	2465	1356	0.9	0.31	1.68
4/13/23	13:05	4890.25		21.08				5	30	252.0	52.8	0.36	53	229	-6	222	44	2670	1374	0.9	0.33	1.79
4/13/23	13:10	4891.23		21.06				5	35	251.7	53.1	0.24	53	226	-4	222	44	2667	1381	0.9	0.33	1.78
4/13/23	13:15	4892.2		21.04				5	40	251.5	53.3	0.24	53	224	-4	220	44	2639	1387	0.9	0.32	1.75
4/13/23	13:20	4893.16		21.03				5	45	251.4	53.4	0.12	53	222	-2	220	44	2636	1392	0.9	0.32	1.75
4/13/23	13:25	4894.12		21.01				5	50	251.1	53.7	0.24	54	222	-4	218	44	2611	1396	0.9	0.31	1.72
4/13/23	13:30	4895.04		21.02				5	55	251.2	53.6	-0.12	54	213	2	215	43	2575	1398	0.9	0.31	1.70
4/13/23	13:40	4896.9		21.02				10	65	251.2	53.6	0	54	430	0	430	43	2578	1396	0.9	0.31	1.70
4/13/23	13:50	4898.77		21.01				10	75	251.1	53.7	0.12	54	432	-2	430	43	2579	1398	0.9	0.31	1.70
4/13/23	14:00	4900.66		21.02				10	85	251.2	53.6	-0.12	54	437	2	439	44	2632	1398	0.9	0.32	1.74
4/13/23	14:10	4902.56		21.06				10	95	251.7	53.1	-0.48	53	439	8	447	45	2683	1390	0.9	0.33	1.78
4/13/23	14:20	4904.48		21.08				10	105	252.0	52.8	-0.24	53	444	4	448	45	2686	1381	0.9	0.33	1.79
4/13/23	14:30	4906.41		21.1				10	115	252.2	52.6	-0.24	53	446	4	450	45	2700	1375	0.9	0.34	1.81
4/13/23	14:40	4908.29		21.11				10	125	252.3	52.5	-0.12	53	434	2	436	44	2618	1371	0.9	0.33	1.76
4/13/23	14:50	4910.17		21.11				10	135	252.3	52.5	0	52	434	0	434	43	2606	1369	0.9	0.33	1.75
4/13/23	15:00	4912.02		21.13				10	145	252.6	52.2	-0.24	52	427	4	432	43	2589	1366	0.9	0.33	1.75
4/13/23	15:10	4913.88		21.13				10	155	252.6	52.2	0	52	430	0	430	43	2578	1363	0.9	0.32	1.74
4/13/23	15:20	4915.67		21.12				10	165	252.4	52.4	0.12	52	413	-2	411	41	2468	1365	0.9	0.31	1.67
4/13/23	15:30	4917.57		21.11				10	175	252.3	52.5	0.12	52	439	-2	437	44	2621	1368	0.9	0.33	1.77
4/13/23	15:40	4919.47		21.1				10	185	252.2	52.6	0.12	53	439	-2	437	44	2621	1371	0.9	0.33	1.76
																					Minimum Rate:	1.67
																					Raw Rate for design, prior to application of adjustment factors:	**1.70**

APPENDIX C

LABORATORY TEST RESULTS

APPENDIX C

GEOTECHNICAL LABORATORY TESTING

The geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

In-Situ Moisture and Density: The natural water content (ASTM D 2216) and in-situ dry density (ASTM D 2937) were determined for recovered relatively undisturbed ring-lined barrel drive samples, from our subsurface explorations. Results of these tests are shown on the logs at the appropriate sample depths, in Appendix B.

Sieve Analysis: Sieve analyses (ASTM D 422) were performed on selected subsurface soil samples. These tests were performed to assist in the classification of the soil. Results of these tests are presented on the "*Particle Size Analysis of Soils*" figures.

Collapse Potential: Collapse potential tests were performed on selected soil samples in general accordance with ASTM Standard Test Method D 5333. Test results are presented on the "*One Dimensional Swell or Settlement*" figure.

Modified Proctor Compaction Curve: A laboratory modified Proctor compaction test (ASTM D 1557) was performed on a bulk soil sample to determine maximum laboratory dry density and optimum moisture content. Result of this test is presented on the following "*Modified Proctor Compaction Test*" plot in this appendix.

Percent Passing No. 200 Sieve: Percent fines (silt and clay) passing the No. 200 U.S. Standard Sieve was determined for soil samples in accordance with ASTM D1140 Standard Test Method. Samples were dried and passed through a No. 4 sieve, then a No. 200 sieve. Result of grain size analyses, as percent by dry weight passing the No. 200 U.S. Standard Sieve, is tabulated in this appendix and entered on our boring logs.

R-value Test: One R-value test was performed on collected bulk soil sample to evaluate pavement support characteristics of the near-surface soils. R-value test was performed in accordance with Caltrans Standard Test Method 301. The test result is presented in this appendix.

Corrosivity Tests: To evaluate the corrosion potential of the subsurface soils at the site, we tested representative bulk samples collected during our subsurface investigation for pH, resistivity and soluble sulfate and chloride content testing. Results of these tests are presented at the end of this appendix.

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	CapRock/6th Avenue/GE	Tested By:	M. Vinet	Date:	04/26/23
Project No.:	13877.001	Input By:	M. Vinet	Date:	04/27/23
Boring No.:	LB-3	Depth (ft.):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Clayey Sand (SC), Dark Brown.				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft^3) 0.03340 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5402	5513	5572	5534		
Weight of Mold (g)	3522	3522	3522	3522		
Net Weight of Soil (g)	1880	1991	2050	2012		
Wet Weight of Soil + Cont. (g)	856.2	831.4	863.3	729.6		
Dry Weight of Soil + Cont. (g)	815.0	783.2	812.9	676.6		
Weight of Container (g)	328.2	327.7	415.0	326.1		
Moisture Content (%)	8.5	10.6	12.7	15.1		
Wet Density (pcf)	124.1	131.4	135.3	132.8		
Dry Density (pcf)	114.4	118.8	120.1	115.4		

Maximum Dry Density (pcf) **120.2** **Optimum Moisture Content (%)** **12.3**

PROCEDURE USED

[ ] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[X] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

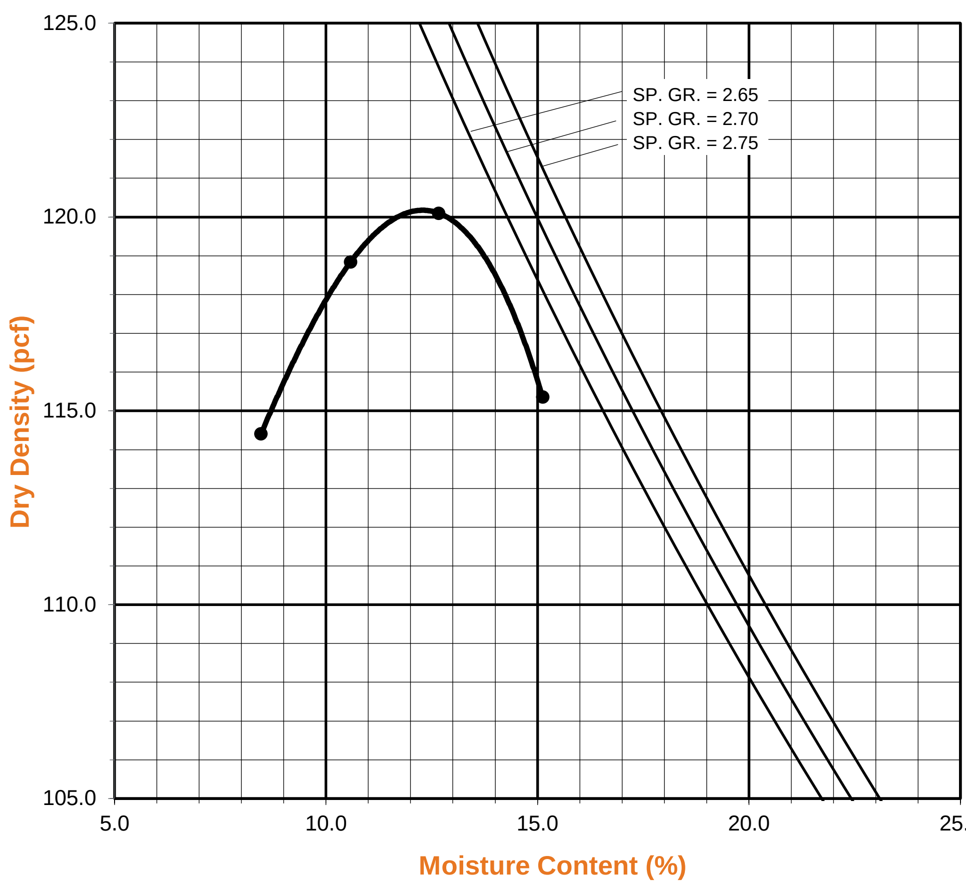
Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

28:20:8

LL,PL,PI



Compaction; LB-3, B-1 (04-03-23)

Boring No.	LB-1	LB-1	LB-1	LB-1	LB-1	LB-1	LB-4	LB-4
Sample No.	R-4	S-5	S-7	R-10	S-11	R-12	R-4	S-5
Depth (ft.)	10.0	15.0	25.0	40.0	45.0	50.0	10.0	15.0
Sample Type	RING	SPT	SPT	RING	SPT	RING	RING	SPT
Soil Classification	SM	SC	SM	SM	s(CL)	SM	SM	s(CL)
Soak Time (min)	10	10	10	10	10	10	10	10
Moisture Correction								
Wet Weight of Soil + Container (gm.)	604.5	677.3	654.2	553.5	629.5	650.8	626.4	654.6
Dry Weight of Soil + Container (gm.)	576.7	641.5	566.1	487.5	566.7	616.5	578.3	595.7
Weight of Container (gm)	277.8	279.6	276.0	279.0	276.5	277.8	281.8	278.4
Moisture Content (%)	9.3	9.9	30.4	31.7	21.6	10.1	16.2	18.6
Container No.:	K2	B1	R2	BL	XW	MA	LA	WO
Sample Dry Weight Determination								
Weight of Sample + Container (gm.)	576.7	641.5	566.1	487.5	566.7	616.5	578.3	595.7
Weight of Container (gm.)	277.8	279.6	276.0	279.0	276.5	277.8	281.8	278.4
Weight of Dry Sample (gm.)	298.9	361.9	290.1	208.5	290.2	338.7	296.5	317.3
Container No.:	K2	B1	R2	BL	XW	MA	LA	WO
After Wash								
Dry Weight of Sample + Container (gm)	470.4	533.0	338.5	394.3	400.8	554.9	433.5	424.0
Weight of Container (gm)	277.8	279.6	276.0	279.0	276.5	277.8	281.8	278.4
Dry Weight of Sample (gm)	192.6	253.4	62.5	115.3	124.3	277.1	151.7	145.6
% Passing No. 200 Sieve	**36**	**30**	**78**	**45**	**57**	**18**	**49**	**54**
% Retained No. 200 Sieve	64	70	22	55	43	82	51	46

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: CapRock/6th Avenue/GE
Project No.: 13877.001
Client Name: CapRock Acquisitions III, LLC
Tested By: M. Vinet Date: 05/01/23

Boring No.	LB-3	IT-1	IT-2					
Sample No.	B-1	S-2	S-2					
Depth (ft.)	0 - 5.0	8.5	23.5					
Sample Type	BULK	SPT	SPT					
Soil Classification	SC	SM	SP-SM					
Soak Time (min)	10	10	10					
Moisture Correction								
Wet Weight of Soil + Container (gm.)	683.2	613.9	607.0					
Dry Weight of Soil + Container (gm.)	630.5	584.4	591.8					
Weight of Container (gm)	276.3	277.5	280.0					
Moisture Content (%)	14.9	9.6	4.9					
Container No.:	R2	AB	PO					
Sample Dry Weight Determination								
Weight of Sample + Container (gm.)	630.5	584.4	591.8					
Weight of Container (gm.)	276.3	277.5	280.0					
Weight of Dry Sample (gm.)	354.2	306.9	311.8					
Container No.:	R2	AB	PO					
After Wash								
Dry Weight of Sample + Container (gm)	471.7	508.9	573.4					
Weight of Container (gm)	276.3	277.5	280.0					
Dry Weight of Sample (gm)	195.4	231.4	293.4					
% Passing No. 200 Sieve	**45**	**25**	**6**					
% Retained No. 200 Sieve	55	75	94					

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: CapRock/6th Avenue/GE
Project No.: 13877.001
Client Name: CapRock Acquisitions III, LLC
Tested By: M. Vinet Date: 04/26/23

ATTERBERG LIMITS

ASTM D 4318

Project Name:	CapRock/6th Avenue/GE	Tested By:	M. Vinet	Date:	05/03/23
Project No. :	13877.001	Input By:	M. Vinet	Date:	05/03/23
Boring No.:	LB-1	Checked By:	M. Vinet		
Sample No.:	R-10	Depth (ft.)	40.0		
Soil Identification:	Silty Sand (SM), Dark Brown.				

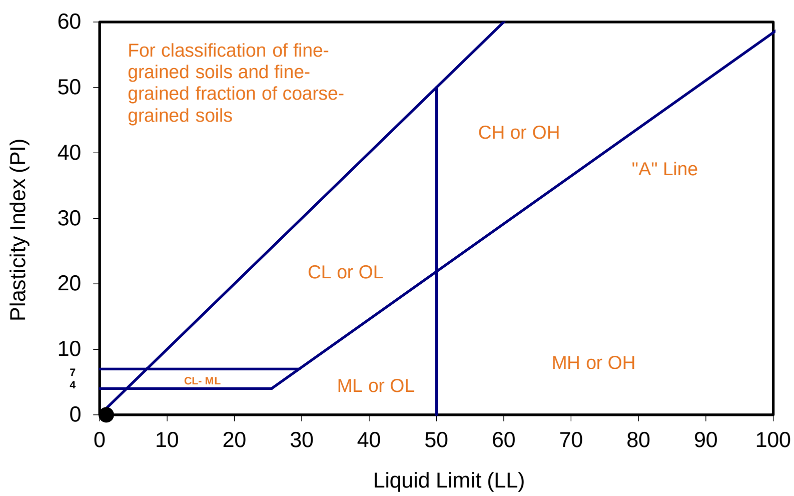
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]						
Wet Wt. of Soil + Cont. (g)	Non-Plastic (NP)		Non-Plastic (NP)			
Dry Wt. of Soil + Cont. (g)						
Wt. of Container (g)						
Moisture Content (%) [Wn]						

Liquid Limit	**NP**
Plastic Limit	**NP**
Plasticity Index	**NP**
Classification	**SM**

PI at "A" - Line = 0.73(LL-20) =

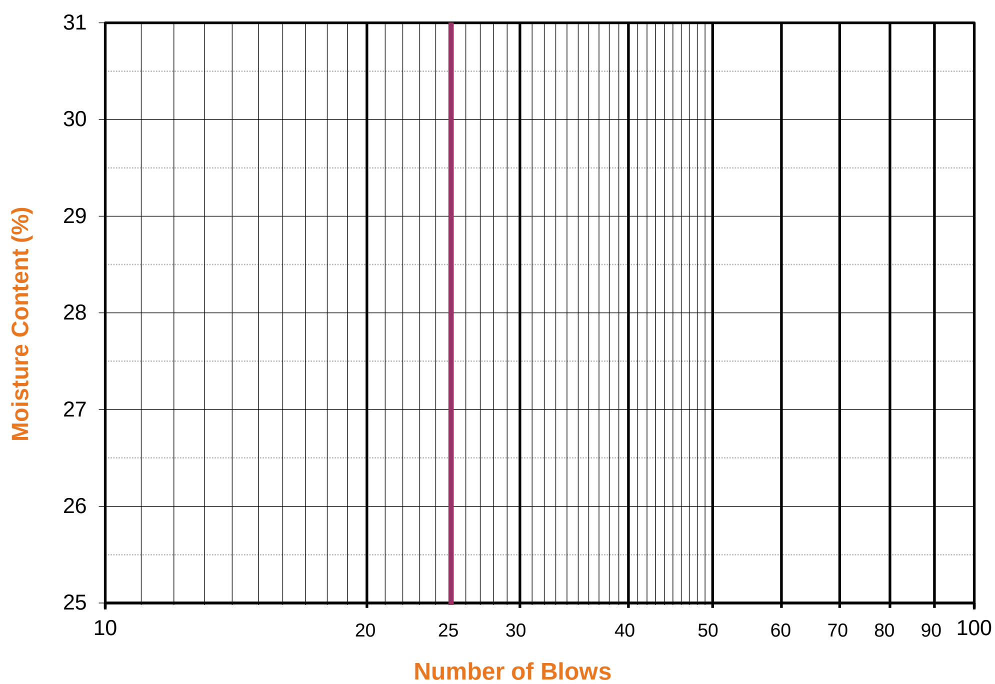
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	CapRock/6th Avenue/GE	Tested By:	M. Vinet	Date:	05/03/23
Project No. :	13877.001	Input By:	M. Vinet	Date:	05/03/23
Boring No.:	LB-1	Checked By:	M. Vinet		
Sample No.:	S-7	Depth (ft.)	25.0		
Soil Identification:	Silty Sand (SM), Dark Brown.				

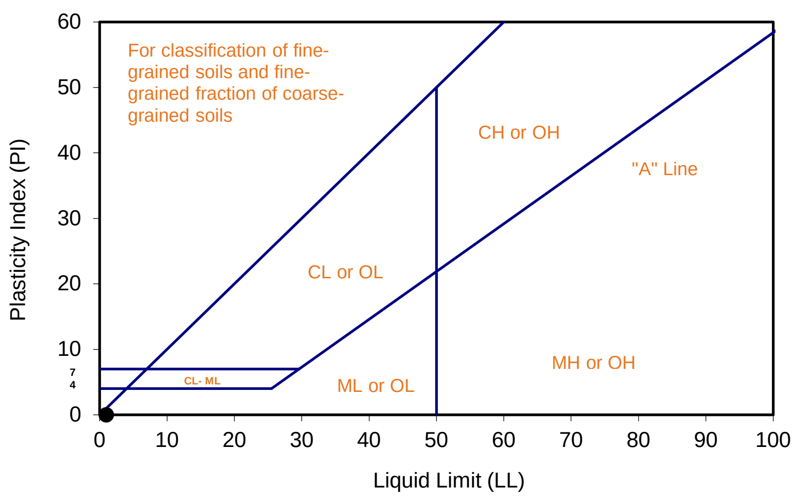
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]						
Wet Wt. of Soil + Cont. (g)	Non-Plastic (NP)		Non-Plastic (NP)			
Dry Wt. of Soil + Cont. (g)						
Wt. of Container (g)						
Moisture Content (%) [Wn]						

Liquid Limit	**NP**
Plastic Limit	**NP**
Plasticity Index	**NP**
Classification	**SM**

PI at "A" - Line = 0.73(LL-20) =

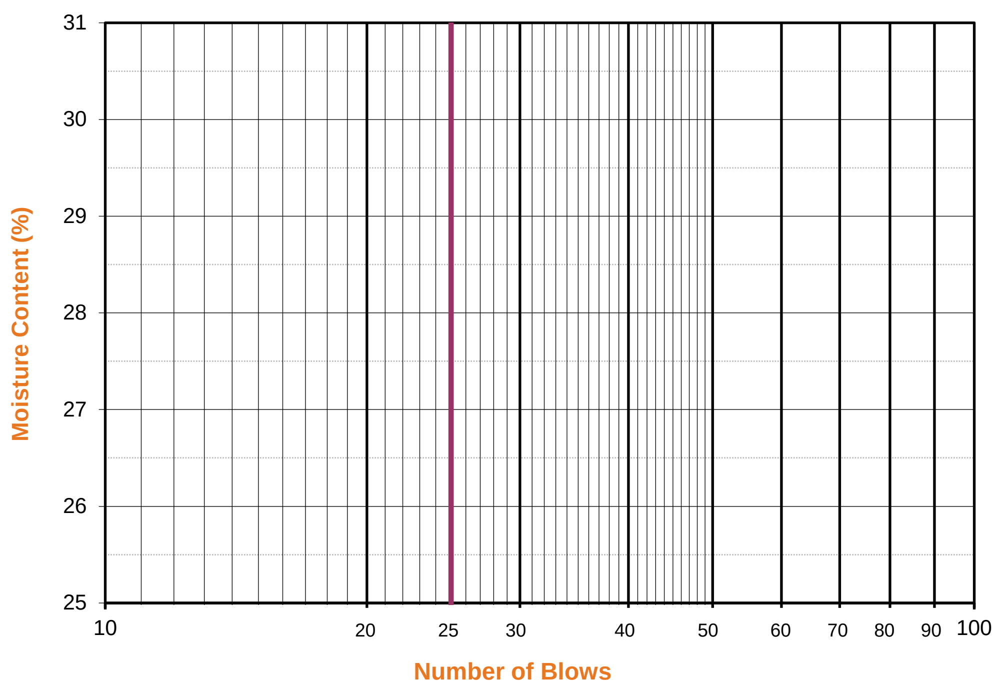
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation — Multipoint - Wet
- [x] Dry Preparation — Multipoint - Dry
- [x] Procedure A — Multipoint Test
- [ ] Procedure B — One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	CapRock/6th Avenue/GE	Tested By:	M. Vinet	Date:	05/03/23
Project No. :	13877.001	Input By:	M. Vinet	Date:	05/03/23
Boring No.:	LB-1	Checked By:	M. Vinet		
Sample No.:	S-11	Depth (ft.)	45.0		
Soil Identification:	Sandy Lean Clay s(CL), Dark Brown.				

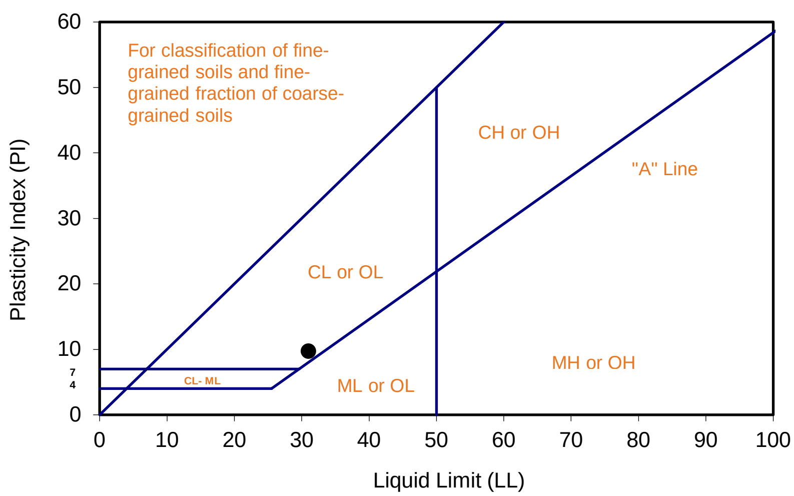
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			15	25	35	
Wet Wt. of Soil + Cont. (g)	21.67	23.22	24.22	25.09	30.42	
Dry Wt. of Soil + Cont. (g)	20.29	21.54	21.63	22.36	26.48	
Wt. of Container (g)	13.78	13.65	13.84	13.68	13.61	
Moisture Content (%) [Wn]	21.20	21.29	33.25	31.45	30.61	

Liquid Limit	**31**
Plastic Limit	**21**
Plasticity Index	**10**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 8.03

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



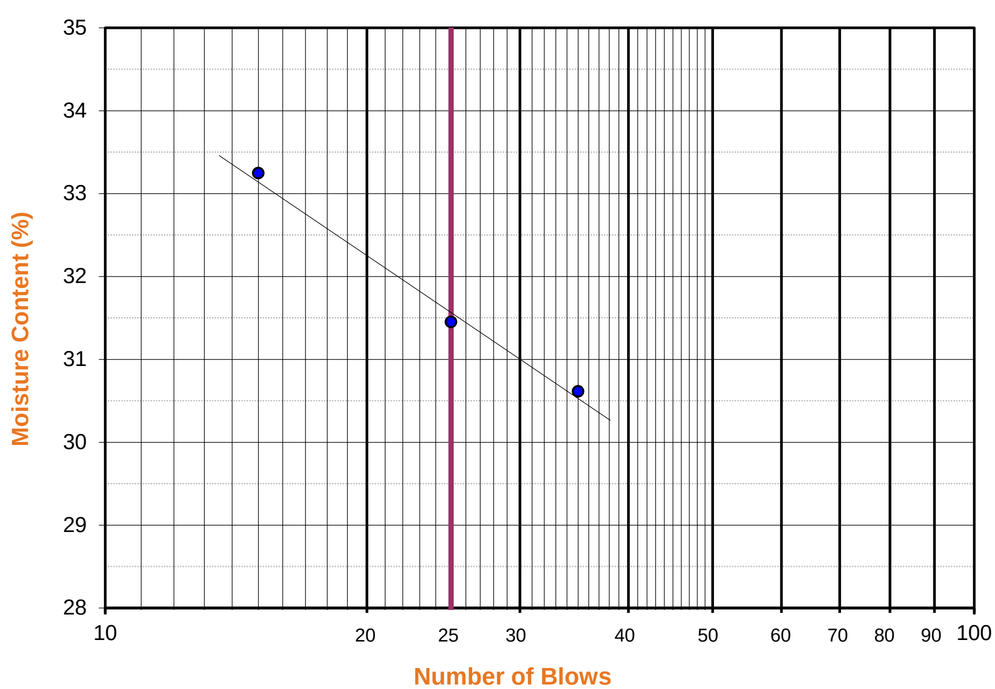
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



ATTERBERG LIMITS

ASTM D 4318

Project Name:	CapRock/6th Avenue/GE	Tested By:	M. Vinet	Date:	04/27/23
Project No. :	13877.001	Input By:	M. Vinet	Date:	04/27/23
Boring No.:	LB-3	Checked By:	M. Vinet		
Sample No.:	B-1	Depth (ft.)	0 - 5.0		
Soil Identification:	Clayey Sand (SC), Dark Brown.				

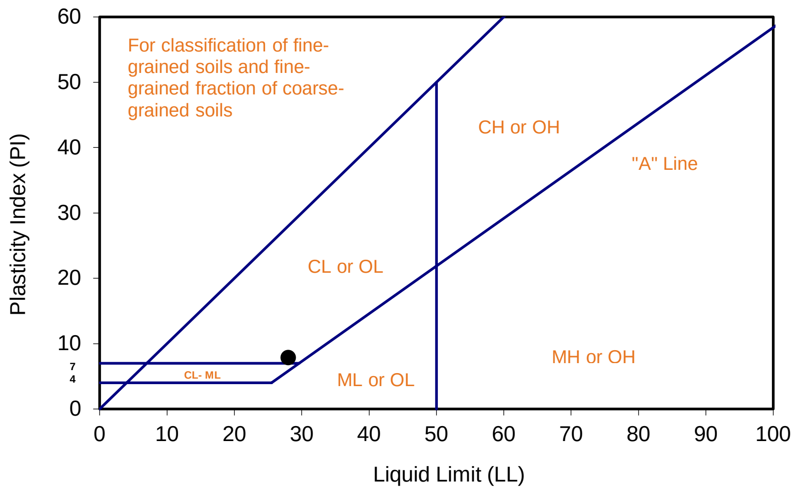
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			16	23	30	
Wet Wt. of Soil + Cont. (g)	22.14	20.46	27.12	26.21	27.06	
Dry Wt. of Soil + Cont. (g)	20.71	19.35	24.09	23.45	24.14	
Wt. of Container (g)	13.67	13.79	13.68	13.70	13.65	
Moisture Content (%) [Wn]	20.31	19.96	29.11	28.31	27.84	

Liquid Limit	**28**
Plastic Limit	**20**
Plasticity Index	**8**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 5.84

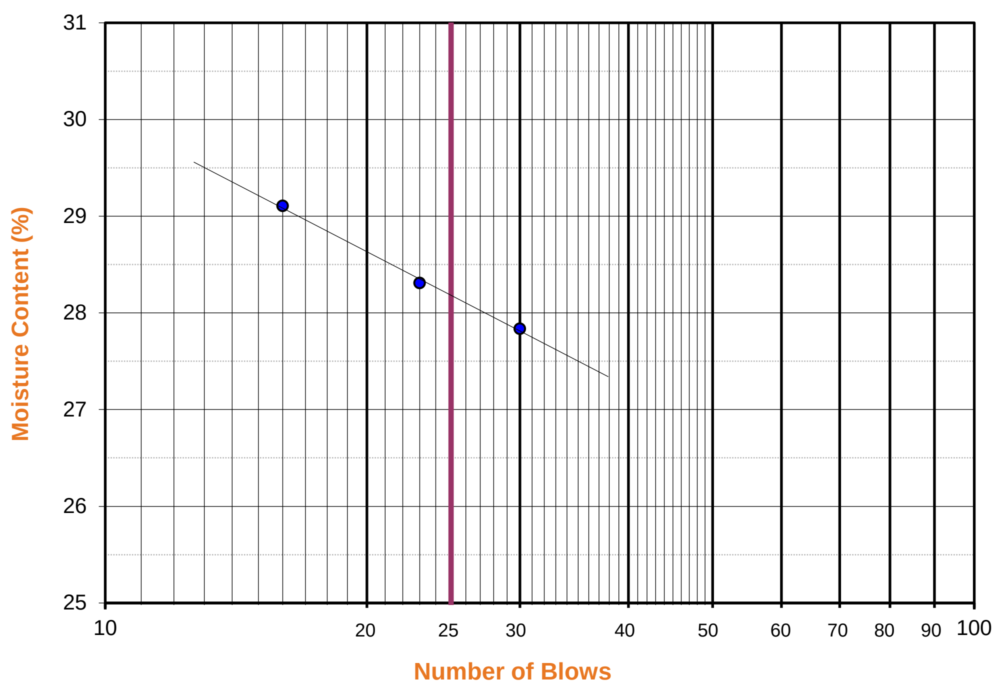
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [x] Dry Preparation
  Multipoint - Dry
- [x] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	CapRock/6th Avenue/GE	Tested By:	M. Vinet	Date:	4/26/23
Project No. :	13877.001	Checked By:	M. Vinet	Date:	4/27/23
Boring No.:	LB-3	Depth:	0 - 5.0		
Sample No. :	B-1	Location:	N/A		
Sample Description:	Clayey Sand (SC), Dark Brown.				

Dry Wt. of Soil + Cont. (gm.)	4016.2
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	4016.2
Weight Soil Retained on #4 Sieve	256.2
Percent Passing # 4	93.6

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0243
Wt. Comp. Soil + Mold (gm.)	572.0	606.4
Wt. of Mold (gm.)	190.7	190.7
Specific Gravity (Assumed)	2.70	2.70
Container No.	12	12
Wet Wt. of Soil + Cont. (gm.)	577.6	606.4
Dry Wt. of Soil + Cont. (gm.)	544.3	338.9
Wt. of Container (gm.)	277.6	190.7
Moisture Content (%)	12.5	22.6
Wet Density (pcf)	115.0	122.4
Dry Density (pcf)	102.2	99.8
Void Ratio	0.649	0.689
Total Porosity	0.394	0.408
Pore Volume (cc)	81.5	86.5
Degree of Saturation (%) [S meas]	**52.0**	**88.8**

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h.

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
4/26/23	10:30	1.0	0	0.5000
4/26/23	10:40	1.0	10	0.5000
	Add Distilled Water to the Specimen			
4/27/23	8:00	1.0	1280	0.5243
4/27/23	9:00	1.0	1340	0.5243

Expansion Index (EI meas) =	((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	24.3
Expansion Index (Report) =	Nearest Whole Number or Zero (0) if Initial Height is > than Final Heigh	**24**

Rev. 03-08

One-Dimensional Swell or Settlement Potential of Cohesive Soils

(ASTM D 4546) -- Method 'B'

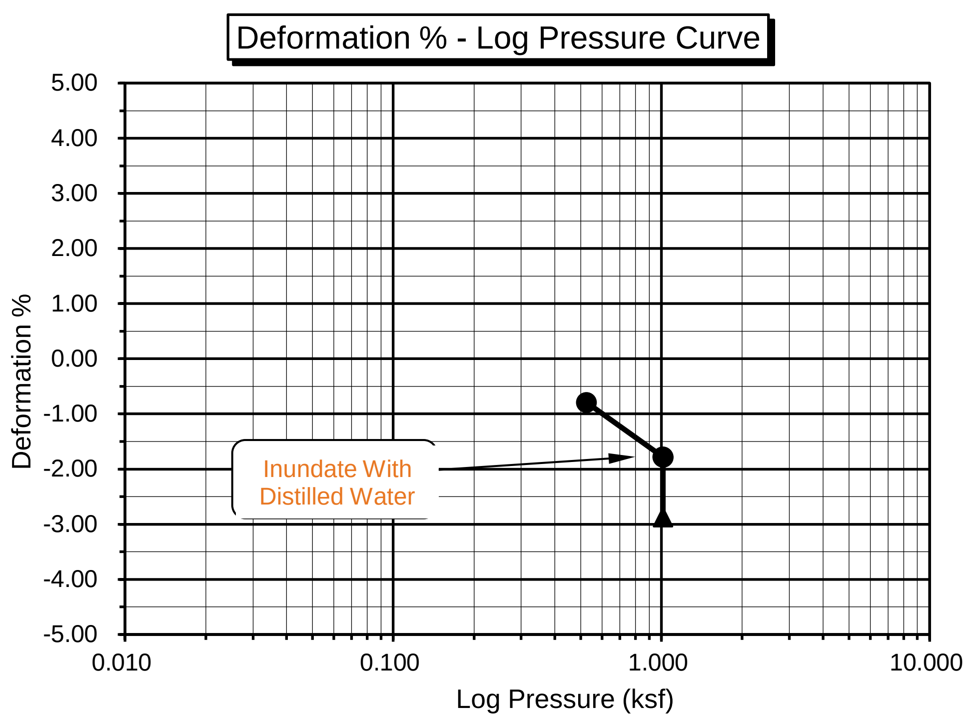
Project Name: CapRock/6th Avenue/GE
Tested By: M. Vinet Date: 4/26/23
Project No.: 13877.001
Checked By: M. Vinet Date: 4/27/23
Boring No.: LB-1
Sample Type: IN SITU
Sample No.: R-3
Depth (ft.) 7.5
Sample Description: Silty Sand (SM), Yellowish Brown.
Source and Type of Water Used for Inundation: Arrowhead (Distilled)
** Note: Loading After Wetting (Inundation) not Performed Using this Test Method.

Initial Dry Density (pcf):	97.3
Initial Moisture (%):	17.1
Initial Height (in.):	1.0000
Initial Dial Reading (in):	0.0000
Inside Diameter of Ring (in):	2.416

Final Dry Density (pcf):	100.2
Final Moisture (%) :	23.6
Initial Void ratio:	0.7316
Specific Gravity (assumed):	2.70
Initial Degree of Saturation (%):	63.2

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.525	0.0079	0.9921	0.00	-0.79	0.7179	-0.79
1.013	0.0178	0.9822	0.00	-1.78	0.7008	-1.78
H_2O	0.0287	0.9713	0.00	-2.87	0.6819	-2.87

Percent Swell / Settlement After Inundation = -1.11



Rev. 01-10

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: CapRock/6th Avenue/GE Tested By : M. Vinet Date: 04/27/23

Project No. : 13877.001 Data Input By: M. Vinet Date: 04/27/23

Boring No.	LB-3			
Sample No.	B-1			
Sample Depth (ft)	0 - 5.0			
Soil Identification:	Clayey Sand (SC)			
Wet Weight of Soil + Container (g)	100.0			
Dry Weight of Soil + Container (g)	100.0			
Weight of Container (g)	0.0			
Moisture Content (%)	0.0			
Weight of Soaked Soil (g)	100.0			

SULFATE CONTENT, Hach Kit Method

Dilution : 1	3			
Water Fraction (ml)	25			
Tube Reading	60			
PPM Sulfate	**180**			
% Sulfate	0.0180			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.6			
PPM of Chloride (C -0.2) * 100 * 30 / B	40			
PPM of Chloride, Dry Wt. Basis	**40**			

pH TEST, DOT California Test 643

pH Value	**7.60**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST
DOT CA TEST 643

Project Name:	CapRock/6th Avenue/GE	Tested By :	M. Vinet	Date:	04/27/23
Project No. :	13877.001	Data Input By:	M. Vinet	Date:	04/27/23
Boring No.:	LB-3	Depth (ft.) :	0 - 5.0		
Sample No. :	B-1				

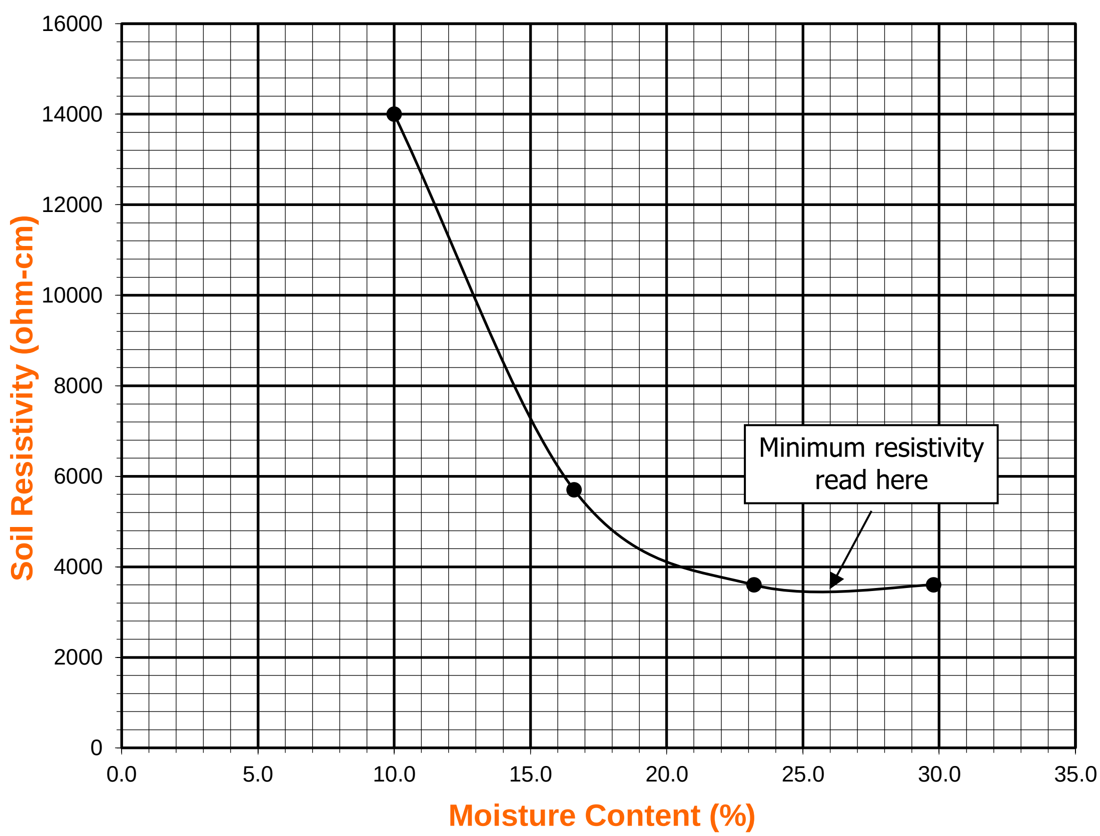
Soil Identification:* Clayey Sand (SC)

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	50	10.00	14000	14000
2	83	16.60	5700	5700
3	116	23.20	3600	3600
4	149	29.80	3600	3600
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	100.00
Dry Wt. of Soil + Cont. (g)	100.00
Wt. of Container (g)	0.00
Container No.	A
Initial Soil Wt. (g) (Wt)	500.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		Hach Kit	DOT CA Test 422	DOT CA Test 643	
3400	**26.0**	**180**	**40**	**7.60**	**21.0**



APPENDIX D

SUMMARY OF SEISMIC HAZARD ANALYSIS

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

34.0361

Longitude

Decimal degrees, negative values for western longitudes

-117.9785

Site Class

259 m/s (Site class D)

Spectral Period

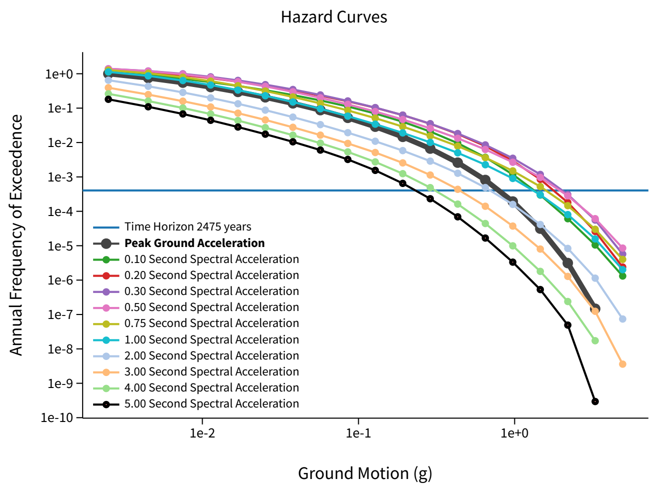
Peak Ground Acceleration

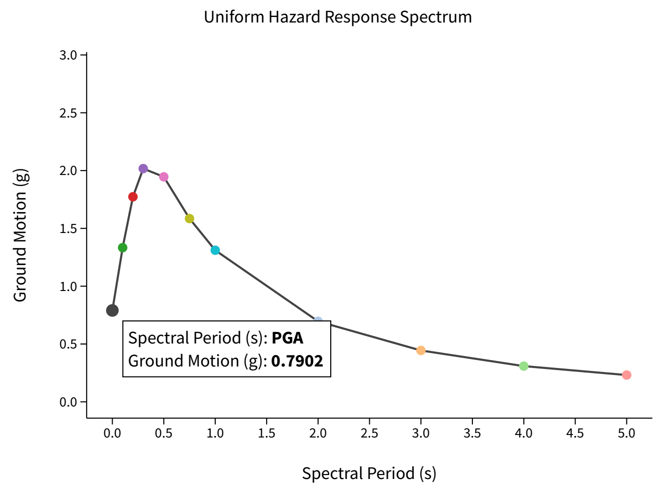
Time Horizon

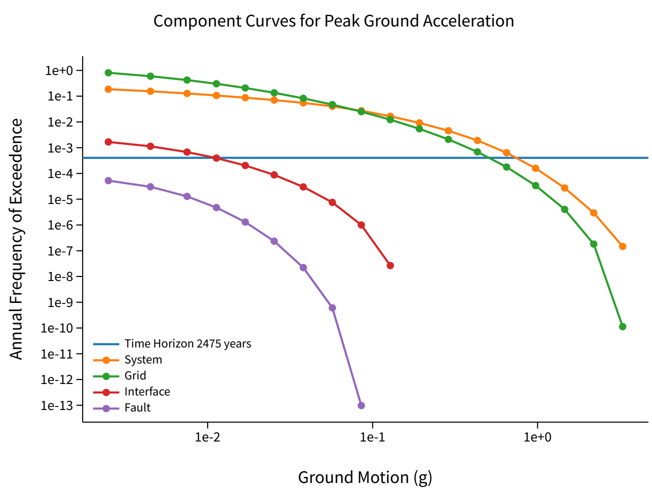
Return period in years

2475

Hazard Curve





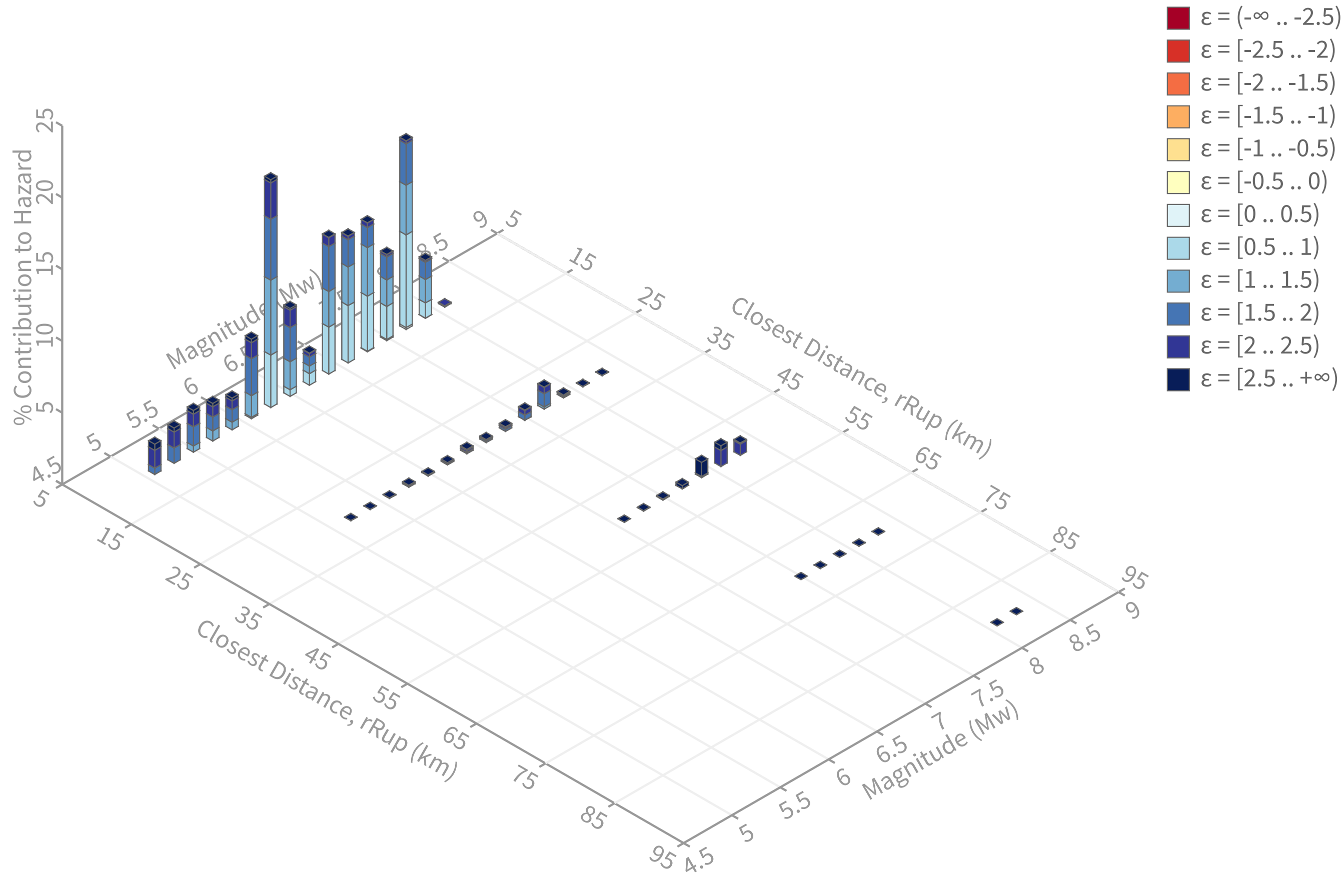


[View Raw Data]

Deaggregation

Component

Total



ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)
% Contribution to Hazard
Closest Distance, rRup (km)
Magnitude (Mw)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.79023608 g

Recovered targets

Return period: 2922.3872 yrs
Exceedance rate: 0.00034218601 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 6.86
r: 12.08 km
ε_0: 1.48 σ

Mode (largest m-r bin)

m: 6.27
r: 7.86 km
ε_0: 1.44 σ
Contribution: 15.87 %

Mode (largest m-r-ε_0 bin)

m: 7.74
r: 8 km
ε_0: 0.76 σ
Contribution: 6.46 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							42.15
Whittier alt 1 [7]		6.57	6.67	1.16	118.009°W	33.983°N	205.29	10.38
Puente Hills [1]		9.71	7.37	0.85	117.978°W	33.944°N	179.63	6.88
Whittier alt 1 [6]		6.90	6.39	1.28	117.990°W	33.975°N	189.03	3.24
San Jose [2]		9.15	7.05	1.10	117.881°W	34.043°N	85.12	3.00
Compton [1]		18.14	7.27	1.39	118.161°W	33.764°N	209.18	2.76
Elysian Park (Upper) [0]		12.54	6.51	2.04	118.097°W	34.077°N	292.71	2.63
Raymond [0]		13.75	7.30	1.74	118.046°W	34.145°N	332.84	2.49
Sierra Madre [2]		14.32	7.74	1.56	117.942°W	34.161°N	13.57	1.78
San Andreas (Mojave S) [11]		45.73	8.08	2.47	117.756°W	34.404°N	26.45	1.68
UC33brAvg_FM32	System							39.10
Whittier alt 2 [6]		6.76	7.19	1.03	118.010°W	33.983°N	205.96	8.97
Whittier alt 2 [5]		7.29	7.16	1.03	117.982°W	33.972°N	182.67	4.53
Compton [1]		18.14	7.33	1.37	118.161°W	33.764°N	209.18	2.95
San Jose [2]		9.15	7.03	1.11	117.881°W	34.043°N	85.12	2.93
Puente Hills (Santa Fe Springs) [0]		9.80	6.92	1.09	118.023°W	33.950°N	203.46	2.82
Raymond [0]		13.75	7.29	1.75	118.046°W	34.145°N	332.84	2.62
Puente Hills (LA) [0]		12.45	7.16	1.43	118.116°W	33.990°N	248.22	2.48
Sierra Madre [2]		14.32	7.77	1.55	117.942°W	34.161°N	13.57	1.80
San Andreas (Mojave S) [11]		45.73	8.08	2.47	117.756°W	34.404°N	26.45	1.68
Richfield [1]		14.06	6.17	1.85	117.870°W	33.882°N	149.70	1.57
Elysian Park (Upper) [0]		12.54	6.82	1.87	118.097°W	34.077°N	292.71	1.42
Puente Hills (Coyote Hills) [1]		10.26	7.12	0.87	118.024°W	33.910°N	196.74	1.38
UC33brAvg_FM32 (opt)	Grid							9.51
PointSourceFinite: -117.978, 34.077		6.82	5.64	1.65	117.978°W	34.077°N	0.00	2.07
PointSourceFinite: -117.978, 34.077		6.82	5.64	1.65	117.978°W	34.077°N	0.00	2.07
UC33brAvg_FM31 (opt)	Grid							9.24
PointSourceFinite: -117.978, 34.077		6.88	5.59	1.68	117.978°W	34.077°N	0.00	2.00
PointSourceFinite: -117.978, 34.077		6.88	5.59	1.68	117.978°W	34.077°N	0.00	2.00

Latitude, Longitude: 34.0361, -117.9785

Date	4/24/2023, 2:50:02 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.753	MCE_R ground motion. (for 0.2 second period)
S_1	0.63	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.753	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.169	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.748	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.823	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.753	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.934	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.297	Factored deterministic acceleration value. (0.2 second)
S1RT	0.63	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.698	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.734	Factored deterministic acceleration value. (1.0 second)
PGAd	0.919	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.748	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.907	Mapped value of the risk coefficient at short periods
C_{R1}	0.902	Mapped value of the risk coefficient at a period of 1 s
C_V	1.451	Vertical coefficient

DISCLAIMER

While the information presented on this website is believed to be correct, SEAOC /OSHPD and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in this web application should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. SEAOC / OSHPD do not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the seismic data provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the search results of this website.

Liquefaction Susceptibility Analysis: SPT Method

Leighton

Youd and Idriss (2001), Martin and Lew (1999)

Description: 110 6th Avenue Industry Warehouse; Case 4; PGAm 0.823; design GW 33; Overex./scarify 8
Project No.: 13877.001
Apr 2023

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve
LB-1	65	33		8	290	257
LB-2	65	33		8	290	257
LB-3	65	33		8	290	257
LB-4	65	33		8	289	256
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Y (ft)
580.64	71.294
288.15	268.71
402.75	-104.2
68.841	80.603

General Parameters:

a_{max} = 0.82g
M_W = 7.7
MSF eq: 1
MSF = 0.93
Hammer Efficiency = 84
C_E = 1.40
C_B = 1
C_S for SPT? TRUE
Unlined, but room for liner
Rod Stickup (feet) = 3
Ring sample correction = 0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Leighton

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: 110 6th Avenue Industry Warehouse; Case 4; PGAm 0.823; design GW 33; Overex./scarify 8

Project No.: 13877.001

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 3.8	2.5	3.8	OX	30	120	50	1	1.3	65.0	300	116.0	138.6	>Range	300	0.53	0.57	**NonLiq**	138.6	0.00		0.00	5.7
LB-1	3.8 to 6.3	5	2.5	OX	30	120	50	1	1.3	65.0	600	116.0	138.6	>Range	600	0.53	0.57	**NonLiq**	138.6	0.00		0.00	5.7
LB-1	6.3 to 8.0	7.5	1.8	OX	15	120	50	1	1.3	65.0	900	110.9	118.7	>Range	900	0.53	0.56	**NonLiq**	118.7	0.00		0.00	5.7
LB-1	8.0 to 8.8	7.5	0.8		15	120	12	2	1	7.8	900	13.3	16.4	0.175	900	0.53	0.56	**NonLiq**	16.4	1.06		0.10	5.7
LB-1	8.8 to 12.5	10	3.8		**36**	120	22	2	1	14.3	1200	22.4	31.9	>Range	1200	0.52	0.56	**NonLiq**	31.9	0.31		0.14	5.6
LB-1	12.5 to 17.5	15	5.0		**30**	120	9	1	1.13	10.2	1800	13.0	19.8	0.212	1800	0.52	0.55	**NonLiq**	19.8	1.06		0.64	5.5
LB-1	17.5 to 22.5	20	5.0		20	120	22	2	1	14.3	2400	17.7	22.8	0.253	2400	0.51	0.55	**NonLiq**	22.8	0.85		0.51	4.8
LB-1	22.5 to 27.5	25	5.0		**78**	120	7	1	1.1	7.7	3000	8.5	15.3	0.163	3000	0.50	0.54	**NonLiq**	15.3	1.72		1.03	4.3
LB-1	27.5 to 32.5	30	5.0		5	120	69	2	1	44.9	3600	47.8	47.8	>Range	3600	0.50	0.53	**NonLiq**	47.8	0.05		0.03	3.3
LB-1	32.5 to 33.0	35	0.5		5	120	32	1	1.3	41.6	4200	41.1	41.1	>Range	4075.2	0.49	0.52	**NonLiq**	41.1	0.06		0.00	3.2
LB-1	33.0 to 37.5	35	4.5		5	120	32	1	1.3	41.6	4200	41.1	41.1	>Range	4075.2	0.49	0.52	**NonLiq**	41.1			0.00	3.2
LB-1	37.5 to 42.5	40	5.0		**45**	120	21	2	1	13.7	4800	12.6	20.1	0.217	4363.2	0.50	0.53	**0.41**	14.6		1.94	1.16	3.2
LB-1	42.5 to 47.5	45	5.0		**57**	120	9	1	1.1	9.9	5400	8.6	15.3	0.164	4651.2	0.50	0.54	**0.30**	12.6		2.21	1.33	2.1
LB-1	47.5 to 52.0	50	4.5		**18**	120	41	2	1	26.7	6000	22.0	26.7	0.330	4939.2	0.50	0.53	**0.62**	23.0		1.4	0.76	0.8
LB-2	0 to 3.8	2.5	3.8	OX	15	120	50	1	1.3	65.0	300	116.0	124.1	>Range	300	0.53	0.57	**NonLiq**	124.1	0.00		0.00	1.2
LB-2	3.8 to 6.3	5	2.5	OX	15	120	50	1	1.3	65.0	600	116.0	124.1	>Range	600	0.53	0.57	**NonLiq**	124.1	0.00		0.00	1.2
LB-2	6.3 to 8.0	7.5	1.8	OX	15	120	50	1	1.3	65.0	900	110.9	118.7	>Range	900	0.53	0.56	**NonLiq**	118.7	0.00		0.00	1.2
LB-2	8.0 to 8.8	7.5	0.8		15	120	28	2	1	18.2	900	31.0	35.0	>Range	900	0.53	0.56	**NonLiq**	35.0	0.19		0.02	1.2
LB-2	8.8 to 12.5	10	3.8		5	120	31	2	1	20.2	1200	31.6	31.6	>Range	1200	0.52	0.56	**NonLiq**	31.6	0.31		0.14	1.2
LB-2	12.5 to 17.5	15	5.0		30	120	20	2	1	13.0	1800	16.7	23.9	0.272	1800	0.52	0.55	**NonLiq**	23.9	0.53		0.32	1.1
LB-2	17.5 to 22.5	20	5.0		15	120	14	1	1.21	16.9	2400	21.0	24.5	0.283	2400	0.51	0.55	**NonLiq**	24.5	0.80		0.48	0.7
LB-2	22.5 to 27.0	25	4.5		35	120	30	2	1	19.5	3000	21.6	31.0	>Range	3000	0.50	0.54	**NonLiq**	31.0	0.48		0.26	0.3
LB-3	0 to 6.3	5	6.3	OX	65	120	50	1	1.3	65.0	600	116.0	144.2	>Range	600	0.53	0.57	**NonLiq**	144.2	0.00		0.00	2.5
LB-3	6.3 to 8.0	7.5	1.8	OX	30	120	50	1	1.3	65.0	900	110.9	132.7	>Range	900	0.53	0.56	**NonLiq**	132.7	0.00		0.00	2.5
LB-3	8.0 to 8.8	7.5	0.8		30	120	25	2	1	16.3	900	27.7	36.7	>Range	900	0.53	0.56	**NonLiq**	36.7	0.18		0.02	2.5
LB-3	8.8 to 12.5	10	3.8		**45**	120	30	2	1	19.5	1200	30.6	41.7	>Range	1200	0.52	0.56	**NonLiq**	41.7	0.08		0.04	2.4
LB-3	12.5 to 17.5	15	5.0		25	120	25	2	1	16.3	1800	20.8	27.5	0.353	1800	0.52	0.55	**NonLiq**	27.5	0.43		0.26	2.4
LB-3	17.5 to 22.5	20	5.0		15	120	10	1	1.14	11.4	2400	14.2	17.3	0.184	2400	0.51	0.55	**NonLiq**	17.3	1.40		0.84	2.1
LB-3	22.5 to 27.0	25	4.5		10	120	19	2	1	12.4	3000	13.7	**14.9**	0.159	3000	0.50	0.54	**NonLiq**	14.9	2.43		1.31	1.3
LB-4	0 to 3.8	2.5	3.8	OX	20	120	50	1	1.3	65.0	300	116.0	128.9	>Range	300	0.53	0.57	**NonLiq**	128.9	0.00		0.00	2.3
LB-4	3.8 to 6.3	5	2.5	OX	15	120	50	1	1.3	65.0	600	116.0	124.1	>Range	600	0.53	0.57	**NonLiq**	124.1	0.00		0.00	2.3
LB-4	6.3 to 8.0	7.5	1.8	OX	10	120	50	1	1.3	65.0	900	110.9	114.2	>Range	900	0.53	0.56	**NonLiq**	114.2	0.00		0.00	2.3
LB-4	8.0 to 8.8	7.5	0.8		10	120	30	2	1	19.5	900	33.3	34.9	>Range	900	0.53	0.56	**NonLiq**	34.9	0.19		0.02	2.3
LB-4	8.8 to 12.5	10	3.8		**49**	120	27	2	1	17.6	1200	27.5	38.1	>Range	1200	0.52	0.56	**NonLiq**	38.1	0.27		0.12	2.3

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-4	12.5 to 17.5	15	5.0		**54**	120	5	1	1.1	5.5	1800	7.0	**13.5**	0.145	1800	0.52	0.55	**NonLiq**	13.5	1.96		1.18	2.2
LB-4	17.5 to 22.5	20	5.0		30	120	22	2	1	14.3	2400	17.7	25.2	0.296	2400	0.51	0.55	**NonLiq**	25.2	0.78		0.47	1.0
LB-4	22.5 to 27.0	25	4.5		10	120	17	1	1.23	20.9	3000	23.2	24.6	0.284	3000	0.50	0.54	**NonLiq**	24.6	0.98		0.53	0.5

Surface Manifestations of Liquefaction and Liquefaction Bearing Capacity Analysis

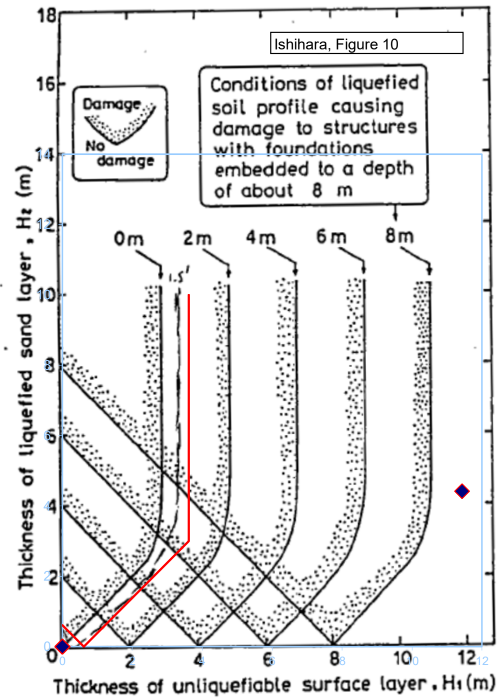
110 6th Avenue Industry Warehouse; Case 4; PGAm 0.823; design GW 33; Overex./scarify 8

13877.001

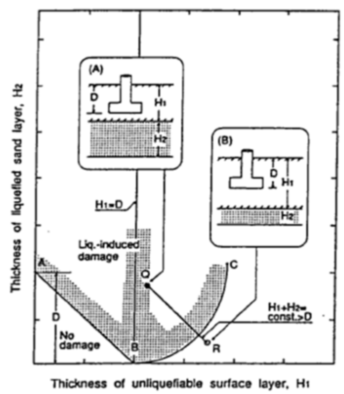
Leighton

Leighton

Boring No.	New Fill (raise grade) (ft)	Footing Depth (ft)	Ishihara, 1995, Surface Manifestisations of Liquefaction Analysis: Bot. Depth of Nonliq and Liq Layers Z1 (non) (ft)	Za (liq) (ft)	Zb (non) (ft)	Zc (liq) (ft)	Thickness H1 (ft)	Thickness H2 (ft)	Thickness H1 (m)	Thickness H2 (m)	Struct Damage/ Surface Manifestations? (Ishihara, 1995)	Amount of New Fill needed to mitigate (ft)	Or, Amount of Overex. needed to mitigate (ft)	Karamitros et al., 2013, Liqufaction Bearing Capacity: Assumed maximum Footing Width Square ftg (ft)	Strip ftg (ft)	In order to achieve critical thickness of Non-liquefiable upper clay crust (where additional thickness does not further increase FSliq of bearing capacity): Amount of New Fill Needed (ft) square ftg	Amount of New Fill Needed (ft) Strip ftg	Or, Amount of Overex. Needed (ft) square ftg	Or, Amount of Overex. Needed (ft) Strip ftg	Juang (2005) based on Iwasaki (1982), as presented in Tonkin & Taylor (2013), Liquefaction Potential Index (LPI): LPI = Sum[F1*W(z)*dz]	Risk of Liquefaction Damage Based on LPI
LB-1	0	2	37.5	52.0			37.5	14.5	11.4	4.4	**no**	0	0	10	4	0.0	0.0	0.0	0.0	0.0	Very Low
LB-2	0	2	27.0				27.0		8.2		**no**	0	0	10	4	0.0	0.0	0.0	0.0	0.0	Very Low
LB-3	0	2	27.0				27.0		8.2		**no**	0	0	10	4	0.0	0.0	0.0	0.0	0.0	Very Low
LB-4	0	2	27.0				27.0		8.2		**no**	0	0	10	4	0.0	0.0	0.0	0.0	0.0	Very Low



Footing Depth= 2 ft
0.61 m



LPI range:	Liquefaction Risk:
LPI=0	Very low
0<LPI<=5	Low
5<LPI<=15	High
LPI>15	Very High

References:

Ishihara, K., 1995, Effects of At-Depth Liquefaction on Embedded Foundations During Earthquakes, Proceedings of 11th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Vol. 2, 1995.

Iwasaki, T., Arakawa, T., and Tokida, K., 1982, Simplifed Procedures for Assessing Soil Liquefaction During Earthquakes Proc. Conference on Soil Dynamics and Earthquake Engineering. Southampton, 925-939

Juang, C.H, Yang, S.H, Yuan, H., and Fang, S.Y., 2005, Liquefaction in the Chi-Chi earthquake – effect of fines and capping non-liquefiable layers Journal of the Japanese Geotechnical Society of Soils and Foundations, Vol. 45 No. 6 pp 89-101

Karamitros, Bouckovalas, Chaloulos, and Andrianopoulos, 2013, Numerical analysis of liquefaction-induced bearing capacity degradation of shallow foundations on a two-layered soil profile, Soil Dynamics and Earthquake Engineering, Vol 44.

Tonkin & Taylor Ltd, 2013, Liquefaction Vulnerability Study, Earthquake Commission, T&T Ref

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The

Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety: The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.3 Lift Thickness: Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.4 Observation and Testing: The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

REVIEWED [signature] 1/31/79
DESIGN SECTION DATE

CURVE DATA

CURVE	RADIUS Ft.	CENTRAL ANGLE Deg.	CENTRAL ANGLE Min.	CENTRAL ANGLE Sec.	LENGTH Ft.	TANGENT Ft.	B.W.R. Ft.
a	22.5	59	59	45	23.56	-	-

NOTE: FOR GENERAL DRAINAGE NOTES SEE SHEET NO. 9

R.D.D. 328

DATE	BY	REVISIONS
09-23-82	H. Thind	Excepted Line 18 - 23 from Maintenance by L.A.C.F.C.D.

* Per Alhambra Foundry Co. Traffic Grate A-2022 or equal.

AS BUILT PLANS

DATE	BY	REVISIONS
10-27-81	Goldman	Prepared "As Built Plans"
3-7-79	J.T.	Changed Pipe offset on Sixth Ave to 5' from ℄.
2-22-79	J.T.	Changed "D" Loadings. Added * to round Grate

LOS ANGELES COUNTY ROAD DEPARTMENT
HIGHWAY DIVISION
JOB NAME SIXTH AVENUE
LIMITS VALLEY BLVD. TO DON JULIAN RD.
SCALE HORIZ. 1" = 40' VERT. 1" = 4'
JOB NO. 57346
SHEET 6 OF 9 SHTS.
DWG. NO. #40792

PROJECT ENGINEER [signature] C.E. No. 20816 · DESIGNER John H. [signature] · CHECKER W.H. · REFERENCES See Sheet No. 1


REVIEWED
1/31/79
Finished Surface
Exist. ℄ Profile
H.G.L.
66" R.C.P.
1000 D
Q=84 C.F.S.
S=0.0031
60" R.C.P.
1200 D
Q=64 C.F.S.
S=0.0109
Q=47.8 C.F.S.
S=0.0543
24" R.C.P.
1500 D
Q=21 C.F.S.
Not to be maintained by L.A.C.F.C.D.
L. A. COUNTY
NOTE: FOR GENERAL DRAINAGE NOTES SEE SHEET № 9
B.M. 6-A RDLB 1229-1406
Elev. 297.015 Puente 70'
SIXTH AVENUE
VALLEY BLVD.
CITY OF INDUSTRY
AS BUILT PLANS
25.3 cfs
47.8 cfs
21 cfs
47.8-21-25.3=1.5 cfs (Planned)
POC 1
Qplanned=1.5 cfs
Q50onsite= 7.73 cfs
Delta 7.73-1.5= + 6.23 cfs
POC2
64-47.8= 16.2 CFS (See profile above)
Q50 Prop.= 10.2 cfs (POC 2, from onsite)
Q50 Ex.= 6.91 cfs from onsite
Q50 Ex. = 11.24 cfs from onsite as gutter flow (Area A)
Q50 Prop. = 0 cfs from onsite
CURVE DATA
CURVE | RADIUS | CENTRAL ANGLE | LENGTH | TANGENT | B.W.R.
R.D.D. 328
LOS ANGELES COUNTY ROAD DEPARTMENT
DESIGN DIVISION
PROJECT NAME SIXTH AVENUE
LIMITS VALLEY BLVD. TO DON JULIAN RD.
PROJECT NUMBER 57346
SHEET 7 OF 9 SHTS.
DWG. NO. #40793
SCALE HORIZ. 1" = 40' VERT. 1" = 4'
REVISIONS
"PH040793"

REFERENCES	See Sheet No. 1
CHECKER	W.H.
DESIGNER	John H. Trammell
C.E. No.	20,416
PROJECT ENGINEER	James Blunty

REVIEWED 1/31/78

CURVE DATA

CURVE	RADIUS Ft.	CENTRAL ANGLE Deg.	Min.	Sec.	LENGTH Ft.	TANGENT Ft.	B.W.R. Ft.
e	45	59	59	25	47.12	25.98	—

DATE	BY	REVISIONS
09-23-82	H. Third	Identified RDD 329.
10-27-81	Goldman	Prepared "As Built Plans"
7-17-79	J.T.	Deleted 1-M.H. No. 4, 2-W.I. Inlets 1-J.S. No. 4, 42" R.C.P. Lateral & 1 Pipe Support on Proctor Ave.
7-5-79	J.T. J.K.	Changed S.D. Grade
3-7-79	J.T.	Changed Pipe offset from 8' to 5' from ℄
2-22-79	J.T.	Changed "D" Loadings. Added J.S. No. 4 at Sta. 40+79

PH040794

AS BUILT PLANS

CITY OF INDUSTRY

R.D.D. 328

LOS ANGELES COUNTY ROAD DEPARTMENT

DESIGN DIVISION

PROJECT NAME SIXTH AVENUE

LIMITS VALLEY BLVD. TO DON JULIAN RD.

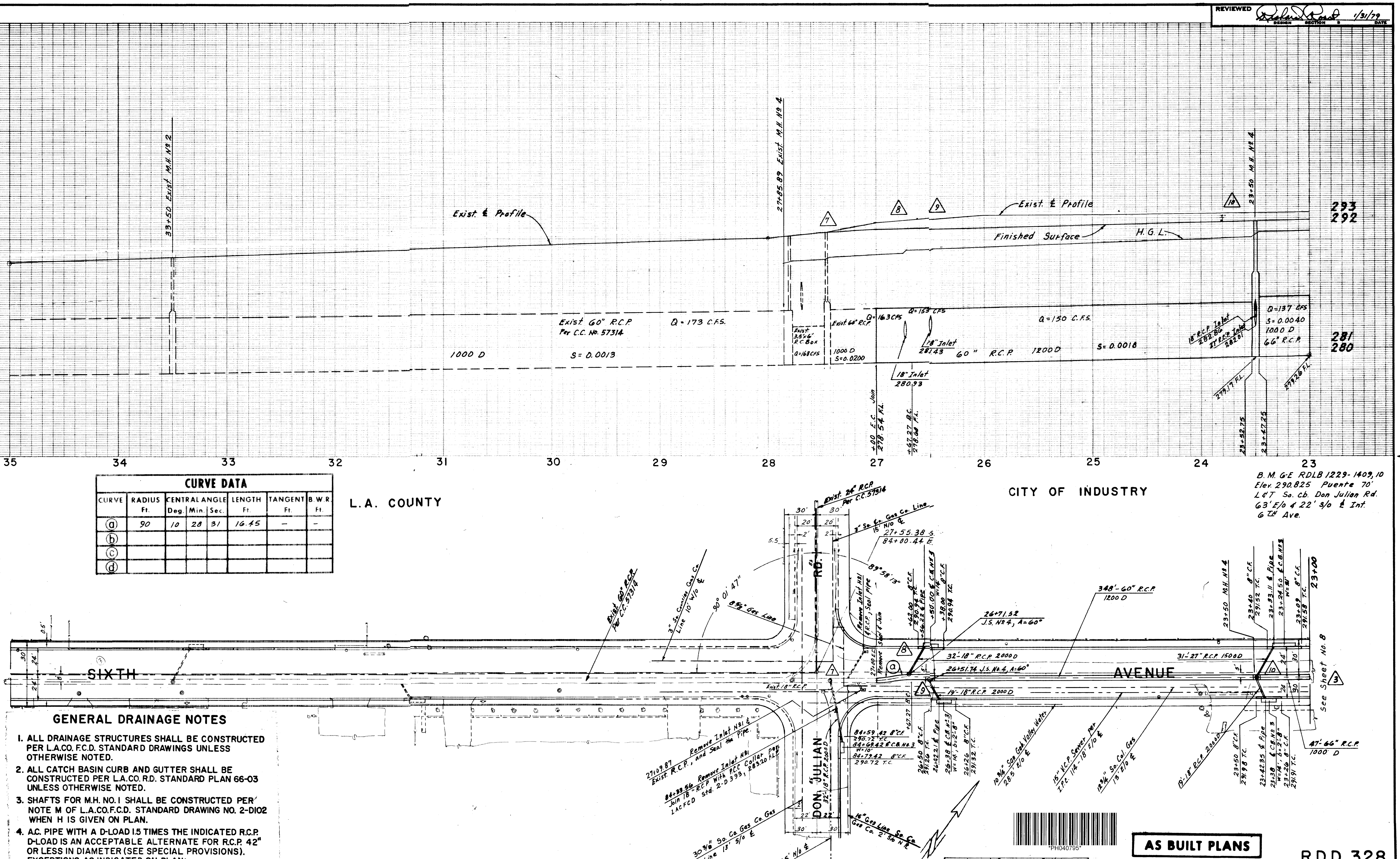
SCALE HORIZ. 1" = 40' VERT. 1" = 4'

PROJECT NUMBER 57346

SHEET 8 OF 9 SHTS.

DWG. NO. #40794

REVIEWED 1/31/79
DESIGN SECTION DATE



CURVE DATA

CURVE	RADIUS Ft.	CENTRAL ANGLE Deg.	Min.	Sec.	LENGTH Ft.	TANGENT Ft.	B.W.R. Ft.
a	90	10	28	31	16.45	–	–
b							
c							
d							

B. M. G-E RDLB 1229-1409,10
Elev. 290.825 Puente 70'
L&T So. Cb. Don Julian Rd.
63' E/o & 22' S/o ℄ Int.
6TH Ave.

GENERAL DRAINAGE NOTES

1. ALL DRAINAGE STRUCTURES SHALL BE CONSTRUCTED PER L.A.CO. F.C.D. STANDARD DRAWINGS UNLESS OTHERWISE NOTED.
2. ALL CATCH BASIN CURB AND GUTTER SHALL BE CONSTRUCTED PER L.A.CO. R.D. STANDARD PLAN 66-03 UNLESS OTHERWISE NOTED.
3. SHAFTS FOR M.H. NO. 1 SHALL BE CONSTRUCTED PER NOTE M OF L.A.CO.F.C.D. STANDARD DRAWING NO. 2-D102 WHEN H IS GIVEN ON PLAN.
4. A.C. PIPE WITH A D-LOAD 1.5 TIMES THE INDICATED R.C.P. D-LOAD IS AN ACCEPTABLE ALTERNATE FOR R.C.P. 42" OR LESS IN DIAMETER (SEE SPECIAL PROVISIONS). EXCEPTIONS AS INDICATED ON PLAN:
5. THE CONTRACTOR WILL NOTIFY THE L.A.C.F.C.D. AREA INSPECTOR AT 226-4206, 24 HOURS PRIOR TO COMMENCING CONSTRUCTION.

CITY OF INDUSTRY

DATE	BY	REVISIONS
03-23-82	H.Thind	Identified Exist. Storm Drains.

AS BUILT PLANS

R.D.D. 328

DATE	BY	REVISIONS
10-27-81	Goldman	Prepared "As Built Plans"
7-5-79	J.T.	Changed Grade of S.D. from 0.0040 to 0.0018
3-7-79	J.T.	Changed Pipe offset to 5' and added (a).
2-22-79	J.T.	Added Note at pipe Join Sta. 27+00

LOS ANGELES COUNTY ROAD DEPARTMENT
DESIGN DIVISION
PROJECT NAME SIXTH AVENUE
LIMITS VALLEY BLVD. TO DON JULIAN RD.
SCALE HORIZ. 1" = 40' VERT. 1" = 4'
PROJECT NUMBER 57346
SHEET 9 OF 9 SHTS.
DWG. NO. #10795

REFERENCES See Sheet No. 1
CHECKER W.H.
DESIGNER John H. Grammell
C.E. No. 20446
PROJECT ENGINEER