APPENDIX F

HYDROLOGY & WATER QUALITY TECHNICAL REPORT

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CSU LONG BEACH BUILDING 9

HYDROLOGY & WATER QUALITY TECHNICAL REPORT

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County of Los Angeles, California

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1. INTRODUCTION

1.1. PROJECT DESCRIPTION

The California State University, Long Beach (CSULB) Technology Park Phase II, Building 9 project ("Project") is an industrial office/warehouse project located on approximately 9.7 acres in the City of Long Beach. The Project is located at the intersection of Pacific Coast Highway (PCH) and Cota Avenue. The Project site is bordered to the north by offices including the Long Beach Job Corps Head Start buildings, to the east by the Long Beach Police – West Division buildings and a McDonald's, to the west by existing CSULB office/warehouse buildings and to the south by a series of motels. The Project site is primarily vacant with a road (Technology Pl) that transects the property. In addition to the road, a few existing buildings are located in the middle and southern portions of the property.

The proposed Project will entail the demolition of the existing buildings and road and the construction of a two-story structure with office space and a warehouse. The buildings will contain a total of 33 dock high positions and 2 grade level drive in doors.

1.2. SCOPE OF WORK

As part of the Mitigated Negative Declaration (MND) for the Project, this report will describe the existing and proposed conditions of surface water hydrology and water quality at the Project site and immediate surrounding areas. The report will include summaries of the regulatory framework that impacts the Project, the environmental setting, and the methodologies and design specifications utilized to ensure local and regional hydrology and water quality regulations are met.

2. REGULATORY FRAMEWORK

2.1. SURFACE WATER HYDROLOGY

County of Los Angeles Hydrology Manual

The Los Angeles County (County) Department of Public Works Hydrology Manual is used as the basis of design for storm drainage facilities in the City of Long Beach. The Hydrology Manual requires that a storm drain conveyance system be designed for at least a 10-year storm event. Areas with sump conditions are required to have a storm drain conveyance system capable of conveying flow from a 50-year storm event (Capital Flood). All drainage facilities in developed areas not covered under the Capital Flood protection conditions must convey flow from a 25-year storm event (Urban Flood). The County also limits the allowable discharge into existing storm drain facilities based on the Municipal Separate Storm Sewer System (MS4) Permit, which is applicable to all new developments that discharge directly into the County's storm drain system. Any proposed drainage improvements of County owned storm drain facilities, such as catch basins and storm drain lines, require review and approval from the County Flood Control District department.

The Los Angeles County Department of Public Works Hydrology Manual (January 2006) establishes the Los Angeles County Department of Public Works' hydrologic design procedures based on historic rainfall and runoff data collected within the County. The hydrologic techniques in the manual apply tor the design of local storm drains, retention and detention basins, pump stations, and major channel projects.

The proposed Project is required to utilize the 2006 Hydrology Manual and accompanying hydrologic tools including HydroCalc Calculator to calculate existing and proposed discharges and volumes from the Project.

City of Long Beach

Any proposed drainage improvements within the street right-of-way or any other property owned by, to be owned by, or under the control of the City requires approval by the City of Long Beach Public Works Department. Storm drain installation plans, which include any connections to the City's storm drain system from a property line to a catch basin or storm drain pipe, are subject to review and approval by the City of Long Beach Public Works Department.

2.2. SURFACE WATER QUALITY

City of Long Beach MS4 Permit

The Los Angeles Regional Water Quality Control Board (LARWQCB or "LA Regional Board") regulates discharges from medium and large MS4s through the Los Angeles County, Long Beach, and Ventura County MS4 Permits. These permits are issued under the National Pollutant Discharge Elimination System (NPDES) Program. The City of Long Beach is subject to waste discharge requirements for its MS4 discharges and is currently governed under Order No. R4-2014-0024 NPDES Permit No. CAS004003. The permitted area includes approximately 47.7 square miles and approximately 180 linear miles of MS4 that flow into surface waters within the Los Angeles River Watershed, the Dominguez Channel and Greater Los Angeles/Long Beach Harbors Watershed Management Area, the Los Cerritos Channel and Alamitos Bay Watershed Management Area, and the San Gabriel River Watershed. The proposed Project is part

of the Dominguez Channel Watershed and ultimately discharges into the Dominguez Channel and the Los Angeles/Long Beach Inner and Outer Harbor.

City of Long Beach Low Impact Development Manual

The Low Impact Development (LID) Best Management Practices (BMP) Design Manual, 2nd Edition, reflects the LID requirements that were adopted by the City of Long Beach on November 16, 2010 as Chapter 18.74 of the Long Beach Municipal Code, approved by Ordinance No. ORD-10-0035, and amended on November 12, 2013 by ORD-13-0024. The purpose of the LID manual is to provide guidance on the requirements and implementation of LID standards and practices in future developments and redevelopments to encourage the beneficial use of rainwater and urban runoff. The LID standards and practices will ensure water quality is protected in receiving waters downstream of new developments and redevelopments. The proposed Project is required to include LID features in its design; additional details are provided in Section 5.2.

3. ENVIRONMENTAL SETTING

3.1. SURFACE WATER HYDROLOGY

3.1.1. Regional

The Project site is located in the Dominguez Channel Watershed ("Dominguez Watershed" or "Watershed") which is located within the southern portion of Los Angeles County, California. The Watershed encompasses approximately 133 square miles and approximately 81% of the watershed is developed. The dominant land use types in the watershed are residential (38%) and industrial (25%). Seventeen (17) cities in addition to the Port of Long Beach, the Port of Los Angeles, and other portions of Los Angeles County are a part of the Dominguez Channel Watershed.

It is estimated that approximately 62% of the land is covered with impervious surfaces which represents the highest percentage for any watershed area in Los Angeles County. Since the majority of the watershed is urban, drainage is primarily conducted through an extensive network of underground storm drains. The majority (~60%) of the runoff drains to the Dominguez Channel, which is the largest drainage feature in the watershed. This man-made channel, which originates in the City of Hawthorne, is 15 miles long and empties into the Los Angeles/Long Beach Harbor. The remaining runoff drains to retention basins for groundwater recharge, to Wilmington Drain (which empties into Machado Lake), or to Los Angeles/Long Beach Harbor directly.

Much of the Watershed area consists of gently rolling plains between low hills, which the Dominguez Channel flows through. In the northern and eastern portions of the watershed, the Rosecrans and Dominguez Hills rise to about 61 meters (200 feet) elevation. In the southwest portion of the watershed, the Palos Verdes Hills rise to an elevation of 451 meters (1,480 feet) and contain numerous small streams and drainages. Isohyets for the 50-year design storm predict over an inch difference in rainfall between the eastern central portion of the watershed and the coast. Up to 5 to 6 inches of rain over a 24-hour period would result in flood flows that would be contained within the design of the flood control channel conveyance system.[1]

3.1.2. Project Specific

Under existing conditions, the surrounding areas of the Project site are built out with a high impervious condition; however, the majority of the Project site itself is vacant with a low impervious condition. The imperviousness of the site is approximately 20% in the existing condition.

The existing site stormwater runoff sheet flows to Technology Place, which transects the property from the northeast corner to the middle of the west boundary. The drainage is then conveyed along the curb and gutter of Technology Place (private street) in a southwesterly direction and into existing offsite private catch basins to the west of the Project. The catch basins connect to an existing 3'x7' reinforced concrete box (Line "A") that runs along Technology Place, which then discharges to an existing/private offsite detention basin located at the northwest corner of Pacific Coast Highway and Technology Place. The outflow from the detention basin flows into an existing 54" storm drain to the west and ultimately to the Dominguez Channel. There are no existing storm drain facilities onsite. Please refer to Appendix A for the existing drainage map.

[1] Dominguez Watershed Master Plan (2004). Found here: https://dpw.lacounty.gov/wmd/watershed/dc/DCMP/masterplan.cfm

The existing offsite storm drain system and detention basin are both privately owned have been designed to convey the commercial development associated with and including the Project. The hydrology that includes this proposed development is presented in the "Hydrology & Hydraulics Report for Tentative Tract No. 52467", dated September 30, 1999 and prepared by RBF for the CSULB Foundation.

Table 1 below provides the 25-year and 50-year storm frequency analysis for the Project Site's existing conditions. Output calculations are provided in Appendix B.

Table 1 Existing Condition 25-year and 50-year Storm Event Hydrology

Drainage Area	Acreage	% Imperviousness	Q_{25} (cfs)	V_{25} (ac-ft)	Q_{50} (cfs)	V_{50} (ac-ft)
1A	9.68	20%	15.72	1.37	19.58	1.62
Notes: See Appendix A for the existing hydrology exhibit and Appendix B for existing hydrology calculations.						

3.1.3. FEMA

The Project is within Panel 1962 of 2350 (Map Number 06037C1962F, dated September 26, 2008) on the Federal Emergency Management Agency's (FEMA's) Flood Insurance Rate Map (FIRM). Based on the FIRM, the project is within Zone X, which depicts areas determined to be outside of the 0.2% (500-year) annual chance floodplain. Therefore, the processing of a CLOMR/LOMR, through FEMA, will not be required for this project.

3.2. SURFACE WATER QUALITY

3.2.1. Regional

As mentioned above, the Project site is located in the Dominguez Watershed and specifically discharges into the Dominguez Channel Estuary. Discharges ultimately end up in the Los Angeles/Long Beach Inner and Outer Harbors. Existing efforts to address the constraints of the watershed are included in the Dominguez Watershed Management Master Plan (DWMMP). The DWMMP was developed by the Dominguez Watershed Advisory Council (DWAC) which consists of local governmental representatives, environmental groups, regulating agencies, members of business and industry, water and sewer service providers, and private citizens. The DWMMP can be used in the development of project concepts for the Dominguez Watershed to protect and enhance water quality; to conserve, reuse, and recharge water supply; to protect, enhance, and restore native habitats and biological resources; and to promote public awareness and involvement.[2]

3.2.1.1. Receiving Water Beneficial Uses

The Dominguez Channel Estuary and the Los Angeles/Long Beach Inner and Outer Harbors existing and potential beneficial uses are described below. The DWMMP described above is aimed at protecting and improving the environment and beneficial uses of the watershed.

Table 2 Receiving Water Beneficial Uses

Dominguez Channel	
NAV – Navigation*	WILD – Wildlife Habitat
COMM – Commercial and Sport Fishing	RARE – Rare, Threatened, or Endangered Species
EST – Estuarine Habitat	MIGR – Migration or Aquatic Organisms
MAR – Marine Habitat	SPWN – Spawning, Reproduction and/or Early Development
Los Angeles/Long Beach Inner Harbor	
REC1 – Water Contact Recreation*	REC2 – Non-contact Recreation
Los Angeles/Long Beach Outer Harbor	
REC1 – Water Contact Recreation	REC2 – Non-contact Recreation
Notes: *Potential beneficial use	

3.2.1.2. Receiving Water 303(d) Impairments and TMDLs

CWA 303(d) List of Water Quality Impairments

Under Section 303(d) of the Clean Water Act (CWA), states are required to identify water bodies that do not meet their water quality standards. Biennially, the LARWQCB prepares a list of impaired waterbodies in the region, referred to as the 303(d) list. The 303(d) list outlines the impaired waterbody and the specific pollutant(s) for which it is impaired. All waterbodies on the 303(d) list are subject to the development of a Total Daily Maximum Load (TMDL).

[2] Los Angeles Department of Public Works – Dominguez Watershed. Found here: https://dpw.lacounty.gov/wmd/watershed/dc/

Stormwater runoff from the Project discharges into the Dominguez Channel Estuary and into the Los Angeles/Long Beach Inner and Outer Harbors. According to the 303(d) list of Limited Water Quality Segments, the Dominguez Channel Estuary and the Los Angeles/Long Beach Inner and Outer Harbors are listed as impaired by the constituents in Table 3 below.

Table 3 Receiving Water 303(d) Water Quality Impairments

Water Body	303(d) Impairment
Dominguez Channel Estuary	Ammonia, Benthic Community Effects, Benzo(a)pyrene (3, 4-Benzopyrene-7-d), Benzo(a)anthracene, Chlordane, Chrysene (C1-C4), Coliform Bacteria, DDT, Dieldrin, Lead, PCBs, Phenanthrene, Pyrene, Sediment Toxicity, Zinc
Los Angeles/Long Beach Inner Harbor	Beach Closures, Benthic Community Effects, benzo(a)pyrene (3, 4-Benzopyrene-7-d), Chrysene (C1-C4). Copper, DDT, PCBs, Sediment Toxicity
Los Angeles/Long Beach Outer Harbor	DDT, PCBs, Sediment Toxicity
Notes: Source: Final 2010 Integrated Report (CWA Section 303(d) List / 305(b) Report). Found here: http://www.waterboards.ca.gov/water_issues/programs/tmdl/2010state_ir_reports/category5_report.shtml	

Total Maximum Daily Loads (TMDLs)

Once a water body has been listed as impaired on the 303(d) list, a TMDL for the constituent of concern (pollutant) must be developed for that water body. A TMDL is an estimate of the daily load of pollutants that a water body may receive from point sources, non-point sources, and natural background conditions (including an appropriate margin of safety), without exceeding its water quality standard. The facilities and activities that are discharging into the water body, collectively, must not exceed the TMDL. In general terms, municipal, small MS4, and other dischargers within each watershed are collectively responsible for meeting the required reductions and other TMDL requirements by the assigned deadline.

TMDLs have been established for certain constituents for the Dominguez Channel Estuary and the Los Angeles/Long Beach Inner and Outer Harbors as shown in Table 4 below. The other 303(d) impairments for these three receiving waters are scheduled to have TMDLs established in the next few years as shown on the State Water Resources Control Board website[3].

Table 4 Receiving Water Established TMDLs

Water Body	Established TMDL
Dominguez Channel Estuary	Coliform Bacteria (2007)
Los Angeles/Long Beach Inner Harbor	Beach closures (2004), Sediment Toxicity (2009), Zinc (2008)
Los Angeles/Long Beach Outer Harbor	Sediment Toxicity (2008)
Notes: Source: Final 2010 Integrated Report (CWA Section 303(d) List / 305(b) Report). Found here: http://www.waterboards.ca.gov/water_issues/programs/tmdl/2010state_ir_reports/category5_report.shtml	

[3] 2010 California 303(d) list of water quality limited segments. Found here:
http://www.waterboards.ca.gov/water_issues/programs/tmdl/2010state_ir_reports/category5_report.shtml

3.2.2. Project Specific

Under existing conditions, the surrounding areas of the Project site are built out with a high impervious condition. However, the majority of the Project site itself is vacant with a low impervious condition. Therefore, the development of the Project will increase the imperviousness of the site from approximately 20% in the existing condition to approximately 90% in the proposed condition. Consequently, when it rains, there will be an increase in the rate that stormwater discharges from the site and the potential to degrade water quality in receiving waters from the new proposed land uses. Anticipated pollutants consistent with parking lots, building areas and landscaping include total suspended solids (TSS), oil/grease, heavy metals, nutrients, pesticides and trash. Additional details are provided in Appendix G on anticipated pollutants and BMPs. As there are no existing water quality BMPs on the Project site, the implementation of LID BMPs will be required as part of the Project's design. The proposed controls on stormwater flow rates associated with the Project are described in Section 5.1, and the proposed LID BMPs are described in more detail in Section 5.2.

4. METHODOLOGY

4.1. SURFACE WATER HYDROLOGY

The study was prepared in conformance with the Los Angeles County Department of Public Works' Hydrology Manual. According to the County's 2006 Hydrology Manual, storm drains associated with the Project must carry flow from at least the 10-year frequency design storm and areas with sump conditions are required to convey flow from the 50-year frequency design storm. However, the existing offsite storm drain infrastructure receiving the flows from the Project is designed for the 25-year storm event. The runoff for 25-year and 50-year frequency design storms was calculated for this report. This study was prepared using HydroCalc 1.0.2 software in conformance with the County's Hydrology Manual (2006). The HydroCalc program uses the Modified Rational Method to calculate the required time of concentration and designed flowrates for the various storm events. The peak runoff for a drainage area is calculated using the formula Q= CIA, where

- Q= flowrate (cfs)
- C= runoff coefficient (unit less)
- I=rainfall intensity (in/hr)
- A= basin area (acres)

The HydroCalc calculator is supported by the County's online GIS system. This database is used to locate the Project Site's 50-year isohyet rainfall frequency as well as relevant soil type (please refer to Appendix B). The data collected is then used in the HydroCalc program to calculate peak stormwater runoff values.

As, mentioned previously in this report, the drainage of the project is currently conveyed to an existing 3'x7' reinforced concrete box which is referred to as Line "A" in the RBF drainage masterplan report. Calculations were performed to confirm that the existing Line "A" has capacity to receive the Project's proposed stormwater flows. Hydraulic analysis was prepared using the Bentley FlowMaster V8i software.

4.2. SURFACE WATER QUALITY

The water quality design components associated with the proposed Project will meet the requirements of the Long Beach Development Services LID BMP Design Manual ("LID Manual"). Stormwater runoff must be infiltrated and/or captured and used through stormwater management techniques to the maximum extent feasible. Projects that have demonstrated they cannot manage 100% of the water quality design volume onsite through infiltration and/or capture and use BMPs may manage the remaining volume through the use of a high removal efficiency biofiltration/biotreatment BMP. The BMPs will be designed to handle the volume generated from a 0.75-inch storm event.

The LID Manual states that BMPs shall be designed to manage and capture stormwater runoff. Infiltration systems are the first priority type of BMP improvements as they provide for percolation and infiltration of the stormwater into the ground, which not only reduces the volume of stormwater runoff entering the MS4 but also contributes to groundwater recharge in some areas. The second priority BMP is capturing and reusing stormwater onsite for either landscape irrigation or toilet flushing. Projects that cannot infiltrate or harvest/reuse the water quality volume may implement biofiltration BMPs. Biofiltration BMPs shall be sized to adequately capture 1.5 times the volume not managed through infiltration and/or capture and reuse.

A geotechnical investigation performed by Southern California Geotechnical was performed on May 12, 2016 to determine the potential to implement infiltration BMPs on the Project site. Groundwater was encountered at depths of 9 to 11± feet below existing site grades, and additional research indicates

historic high groundwater is 8 feet below existing grade. Also, the results of the liquefaction analysis identified potentially liquefiable soils at the site. Therefore, based on the shallow depth to groundwater and the potentially liquefiable soils, as stated in the LID Manual (Table 4.1: Infiltration Feasibility Screening), infiltration is deemed infeasible for this project.

As infiltration is deemed infeasible due to site conditions and grading elevations, a harvest and reuse feasibility screening was performed following Table 4.2: Capture and Use Feasibility Screening in the LID Manual. After analyzing the landscaping type and coverage (~10% pervious) and the Estimated Total Water Use (ETWU) at the Project site, it was determined that capture and use BMPs could not be designed and maintained to ensure adequate capacity to capture and disperse the stormwater design volume within three (3) days of capture. Therefore, biofiltration BMPs are proposed to be implemented to satisfy water quality requirements at the Project site.

Biofiltration BMPs are landscaped facilities that capture and treat stormwater runoff through a variety of physical and biological treatment processes. Facilities normally consist of a ponding area, mulch layer, planting soils, plants, and in some cases, an underdrain. Stormwater passes through biofiltration BMPs for pollutant removal and ultimately connects into the public storm drain system/receiving water body. The Project includes the implementation of biofiltration planters sized to handle 1.5x the water quality volume. See Section 5.2 below and Appendix C for BMP sizing calculations and details.

5. PROPOSED PROJECT EVALUATION

5.1. SURFACE WATER HYDROLOGY

Development of the Project would result in an increase of the site's imperviousness from approximately 20% in the existing condition to 91% in the proposed condition. The proposed imperviousness was obtained from Appendix D (Proportion Impervious Data) of the County's 2006 Hydrology Manual. The Hydrology Manual calls for a proposed imperviousness of 91% for an industrial land use. The 91% imperviousness also matches the average of the percent imperviousness being used for each water quality sub-area calculation in Table 8. Furthermore, the RBF drainage report also implements a 91% imperviousness in the calculations that were used to initially design and size downstream storm drain facilities (i.e. 3'x7' RCB and detention basin).

The Project is divided into seven drainage sub-areas as shown in the proposed hydrology exhibit in Appendix A. Five of the sub-areas (1A-5B), which cover 9.5 acres of the site, will drain into the proposed biofiltration planter boxes before being discharged into the onsite storm drain system. The runoff from the other two sub-areas (6C & 7D) totaling 0.2 acres will sheet flow offsite to the adjacent streets and ultimately connect to the 54" storm drain line parallel to Pacific Coast Highway. As described in Section 5.2, biofiltration planters for water quality treatment will be oversized to provide adequate capacity to treat an equivalent amount of stormwater runoff from drainage sub-areas 6C and 7D.

The majority of the drainage sub-area 8E, which includes the existing police department facility, drains onto Cota Avenue through at grade openings along the existing wall adjacent to the Project's easterly boundary. This existing condition offsite runoff then flows southerly along Cota Avenue and onto Pacific Coast Highway where it flows westerly to an existing offsite catch basin. The portion of Cota Avenue adjacent to the police department facility will be incorporated into the proposed Project. In the proposed condition, Cota Avenue will dead end near the southwest boundary of the police station. To match existing drainage conditions, the Project will propose a concrete channel along the easterly site boundary to collect and divert offsite run-on to the south and onto Cota Avenue via a proposed parkway drain. Therefore, drainage sub-area 8E will not have any comingling of stormwater flows within the Project boundary.

The proposed onsite system to convey runoff from drainage sub-areas 1A-5B is comprised of two 18-inch storm drain lines with one servicing the northerly portion and the other servicing the southerly portion of the Project site. Both lines converge at the proposed sump pump located near the middle portion of the Project's western boundary. The proposed sump pump is being used to drain out the collected site runoff. The Project flows are then conveyed westerly along Technology Place through a proposed 18-inch storm drain line and into an existing offsite catch basin located on Technology Place to the west of the Project. This existing catch basin and lateral ultimately outlet to the 3'x7' RCB referred previously as Line "A". Table 5 below provides an analysis of the 25-year and 50-year frequency design storm events following construction of the Project. See proposed drainage map in Appendix A and output calculations in Appendix B.

Table 5 Proposed Condition 25-year and 50-year Storm Event Hydrology

Proposed Conditions 25-year Storm Frequency				
Drainage Sub-Area	Acreage	% Imperviousness	Q_{25} (cfs)	V_{25} (ac-ft)
1A	0.90	91%	2.45	0.32
2A	2.90	91%	7.24	1.02

3A	0.75	91%	2.04	0.27
4B	2.35	91%	4.59	0.83
5B	2.60	91%	5.08	0.92
Total Site	**9.50**	**91%**	**21.40**	**3.36**
Proposed Conditions 50-year Storm Frequency				
Drainage Sub-Area	Acreage	% Imperviousness	Q_{50} (cfs)	V_{50} (ac-ft)
1A	0.90	91%	2.80	0.36
2A	2.90	91%	9.02	1.17
3A	0.75	91%	2.33	0.30
4B	2.35	91%	5.51	0.95
5B	2.60	91%	6.10	1.05
Total Site	**9.50**	**91%**	**25.76**	**3.83**

Notes:
See Appendix A for the proposed hydrology exhibit and Appendix B for proposed hydrology calculations.
cfs cubic feet per second ac-ft acre-feet

The Project site is included in the drainage masterplan report titled "Hydrology & Hydraulics Report for Tentative Tract No. 52467", prepared by RBF in September 1999. Per the report by RBF, the existing Line "A" storm drain design is based on a 25-year storm event and drains to a detention basin located at the corner of Pacific Coast Highway and Technology Place. Line "A" and the detention basin associated with RBF's report have since been constructed. The proposed Project will connect to the existing system as accounted for in the RBF report. Excerpts from the RBF report and associated storm drain plans are included in Appendix E of this report.

In comparing the existing and proposed flows, it shows that the Project's proposed stormwater flows significantly increase from the existing conditions. Since the Project is originally part of Tract 52467, it is presumed that the existing Line "A" storm drain system has already accounted for and has the capacity to convey this increased runoff quantity. Table 6 below provides a comparison summary of the existing and proposed hydrology.

Table 6 Comparison of Existing and Proposed Hydrology

Condition	Area	Q_{25} (cfs)	Q_{50} (cfs)
Existing	9.68	15.72	19.58
Proposed	9.50*	21.40	25.76
Difference	-0.18	+5.68	+6.18
% Change	-1.9%	+36.1%	+31.6%

Notes:
*Does not include drainage areas 6C and 7D (combined area of 0.20 acres) as they sheet flow offsite and do not connect to the existing detention basin.
cfs cubic feet per second

To verify that Line "A" has sufficient capacity to receive the additional stormwater runoff from the proposed Project, calculations were prepared using the Bentley FlowMaster V8i software. The results of the hydraulic analysis for the proposed 25-yr storm event demonstrates that Line "A", at the downstream portion, flows at 51% of full capacity in this proposed condition. This confirms that Line "A" has sufficient capacity to convey the proposed stormwater flows from the Project. See Appendix F for the Line "A" drainage map and output calculations.

5.2. SURFACE WATER QUALITY

As mentioned above, infiltration and harvest and reuse BMPs have been deemed infeasible to satisfy water quality requirements at the Project site. Therefore, the Project will include the implementation of biofiltration planters sized to handle 1.5x the water quality volume. The water quality volume for each drainage sub-area was calculated by utilizing HydroCalc software as shown in Appendix C. An impervious value of 91% was utilized for BMP sizing to be consistent with the hydrology analysis. It is likely that this imperviousness value is conservative and may change in the final design phases of the Project. See Table 7 below for water quality volumes per drainage sub-area.

Table 7 Water Quality Volume Calculation

Drainage Sub-Area	Acreage	% Imperviousness	Water Quality Volume x 1.5 (cu-ft)
1A	0.90	91%	3,019
2A	2.90	91%	9,725
3A	0.75	91%	2,518
4B	2.35	91%	7,881
5B	2.60	91%	8,719
6C*	0.15	91%	503
7D*	0.05	91%	168
Total Site	**9.70**	**91%**	**32,553**

Notes:
*Subareas 6C and 7D drain offsite without physically being routed to a planter box. However, Subarea 2A was oversized to accommodate for 6C and 7D runoff as an attempt to mitigate stormwater requirements to the maximum extent practicable.
See Appendix C for proposed water quality calculations.
cu-ft cubic feet

As mentioned, biofiltration planter BMPs will be implemented throughout the Project site to treat 1.5 times the water quality volume. The footprints were determined by assuming a maximum time of three (3) hours to reach the ponding depth, a filter media rate of 2.5 inches per hour, and a ponding depth of 12 inches. As shown in Table 8 below, the water quality exhibit in Appendix A, and the water quality calculations in Appendix C, the biofiltration planter footprints provided achieve the minimum footprints required. Drainage sub-areas 6C and 7D drain offsite and cannot be routed to biofiltration planters. Therefore, the planters in the other drainage sub-areas were oversized to provide adequate capacity to treat an equivalent amount of stormwater runoff. Additional details on the biofiltration sizing calculations are provided in Appendix C.

Table 8 Biofiltration Planter Calculations

Drainage Sub-Area	Acreage	Water Quality Volume x 1.5 (cu-ft)	Ponding Depth	Footprint Required (sf)	Footprint Provided (sf)	Water Quality Requirements Satisfied?
1A	0.9	3,019	12″	1,858	2,230	Yes
2A	2.9	9,725	12″	5,985	6,595	Yes
3A	0.75	2,518	12″	1,550	1,553	Yes
4B	2.35	7,881	12″	4,850	5,518	Yes
5B	2.60	8,719	12″	5,364	5,364	Yes
6C*	0.15	503	12″	310	N/A	Yes*
7D*	0.05	168	12″	103	N/A	Yes*
Total	**9.70**	**32,553**	**--**	**20,020**	**21,260**	**Yes**

Notes:
*Subareas 6C and 7D drain offsite without physically being routed to a planter box. However, Subarea 2A was oversized to accommodate for 6C and 7D runoff as an attempt to mitigate stormwater requirements to the maximum extent practicable.
See Appendix A for the proposed water quality exhibit and Appendix C for proposed water quality calculations.

As shown above in Table 8 as well as in Appendices A and C, the proposed LID BMP design will provide enough area to biofilter 1.5 times the volume associated with a 0.75-inch storm event at the Project site. Once the water quality volume is met, the "higher flows" will enter the risers which then drain stormwater to the main storm drain system. The planter boxes will be comprised of a layer of engineered soil media, a layer of rock and an underdrain system. Treated flows will drain back to the main storm drain system.

6. APPENDICES

Appendix A – Existing and Proposed Hydrology/Water Quality Exhibits

Appendix B – Hydrology Calculations

Appendix C – Water Quality Calculations

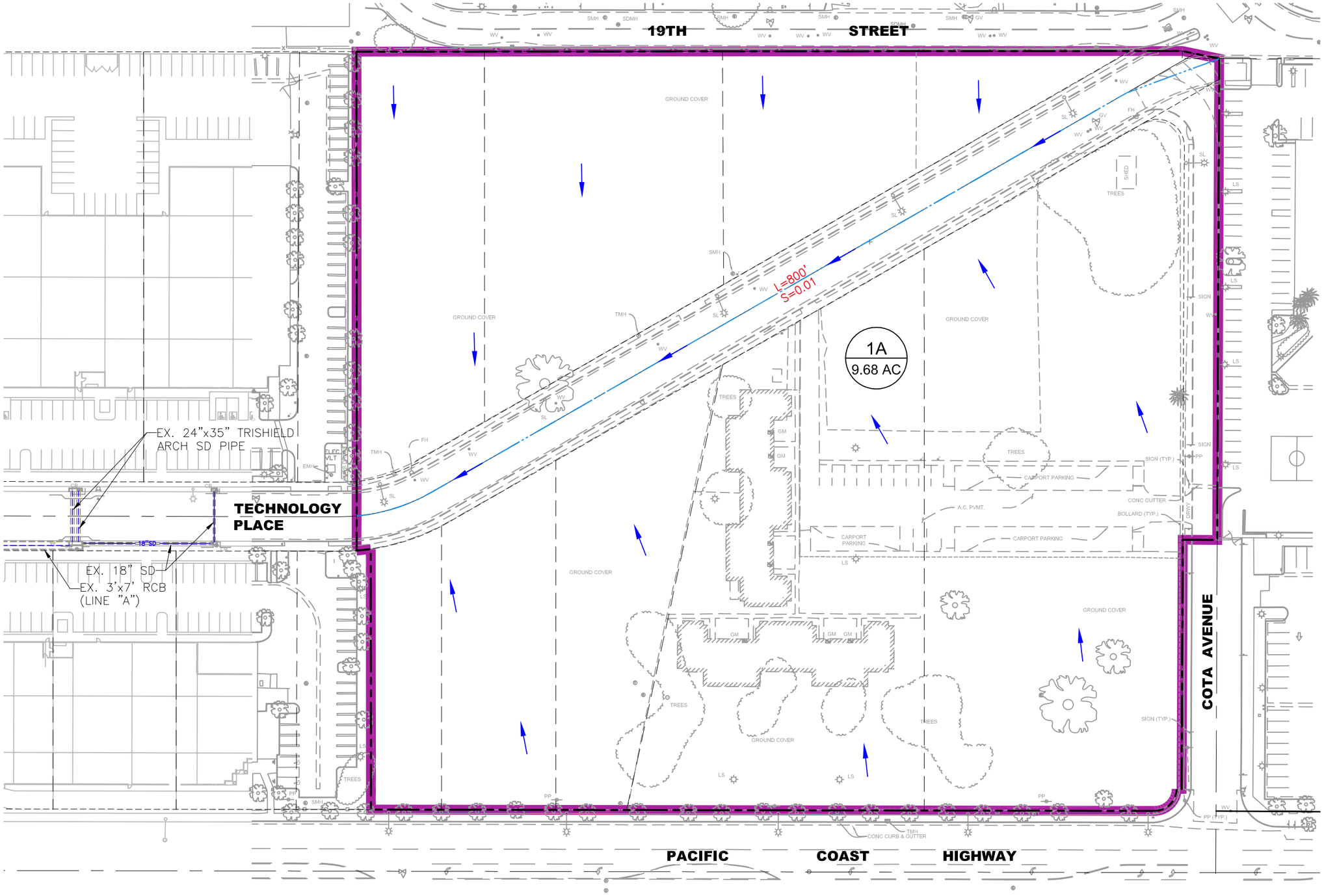
Appendix D – Geotechnical Findings

Appendix E – RBF "Hydrology & Hydraulics Report for Tract 52467" Excerpts

Appendix F – Line "A" Exhibit and Calculations

APPENDIX A

EXISTING AND PROPOSED HYDROLOGY/WATER QUALITY EXHIBITS



LEGEND

- EXISTING STORM DRAIN
- 1A / 9.68 — AREA DESIGNATION / ACRES
- SITE HYDROLOGIC BOUNDARY
- SUB-AREA FLOWPATH
- DIRECTION OF FLOW

80' 0' 40' 80'

SCALE: 1" = 80'

EXISTING CONDITION STORM EVENTS

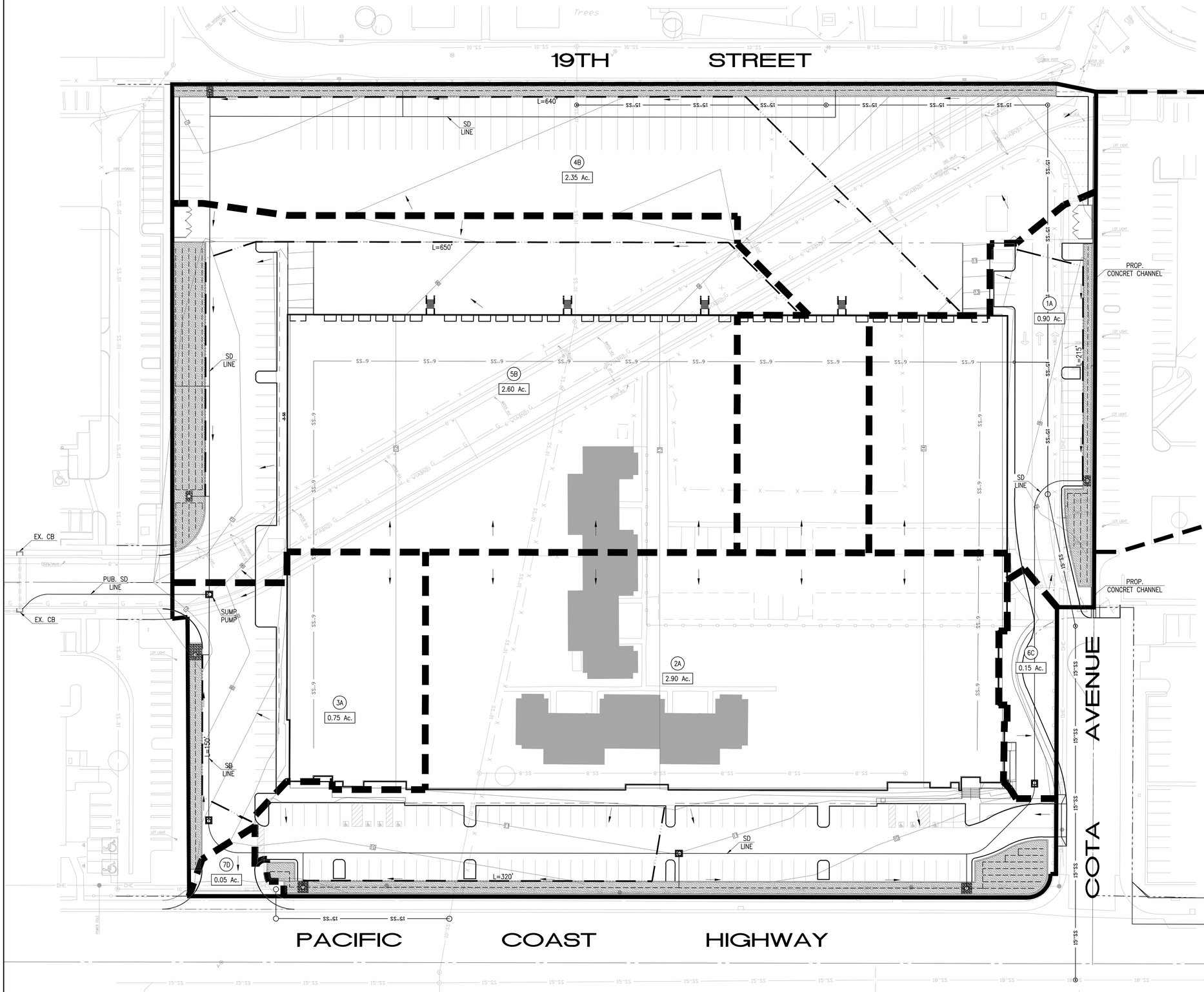
Drainage Area	Acreage	% Imperviousness	Q_{25} (cfs)	V_{25} (ac-ft)	Q_{50} (cfs)	V_{50} (ac-ft)
1A	9.68	20%	15.72	1.37	19.58	1.62

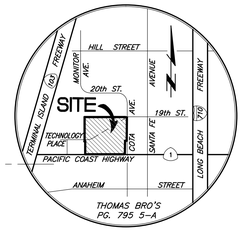
FUSCOE ENGINEERING
16795 Von Karman, Suite 100
Irvine, California 92606
tel 949.474.1960 • fax 949.474.5315
www.fuscoe.com

Exhibit Date: 08/07/2017

EXISTING CONDITION HYDROLOGY
CSU LONG BEACH BLDG 9
LONG BEACH, CA

\\FUSCOE.CORP\IRV\PANZURA\PROJECTS\309\019_SUPPORT FILES\REPORTS\EIR TECH REPORTS\MND\HYDROLOGY AND WQ REPORT\HYDRO EXHIBITS\309-019XH - CSULB BLDG 9 HYDRO-EX.DWG (08-07-17 4:40:15PM) Plotted by: Archie Damidon





VICINITY MAP
NOT TO SCALE

LEGEND

▬▬	PROJECT BOUNDARY
▬ ▬	SUBAREA BOUNDARY
— · · —	FLOW PATH
1.00 AC.	SUBAREA AREA
(1A)	SUBAREA NUMBER

SUBAREA DATA SUMMARY

SUBAREA	AREA (ACRES)	IMP. %	LENGTH (FEET)	SLOPE	Tc (MINUTES)	Q25 (CFS)	Q50 (CFS)
1A	0.90	0.91	215	0.009	5.0	2.5	2.8
2A	2.90	0.91	320	0.010	5.0	7.2	9.0
3A	0.75	0.91	150	0.026	5.0	2.0	2.3
4B	2.35	0.91	640	0.006	9.0	4.6	5.5
5B	2.60	0.91	650	0.005	9.0	5.1	6.1
TOTAL	9.50	0.91	--	--	8.0	19.5*	23.6*
8E (OFFSITE)	2.00	0.91	300	0.005	6.0	--	5.7

*TOTAL PEAK FLOW RATES BASED ON A HYDROCALC RUN FOR THE 9.50 ACRES DRAINING TO THE EXISTING STORM DRAIN SYSTEM IN TECHNOLOGY PLACE.

0 15 30 60 90
SCALE:1"=30'

Last Update: 8/4/17
O:\3500-3599\3572\3572HYD.dwg

CITY OF LONG BEACH
PUBLIC WORKS DEPARTMENT

HYDROLOGY MAP

CSULB TECHNOLOGY PARK III
1924 TECHNOLOGY PLACE

PREPARED FOR:
PROLOGIS
3546 CONCOURS STREET, SUITE 100
ONTARIO, CA 91764
PHONE: (909) 673-8700
FAX: (909) 673-8703

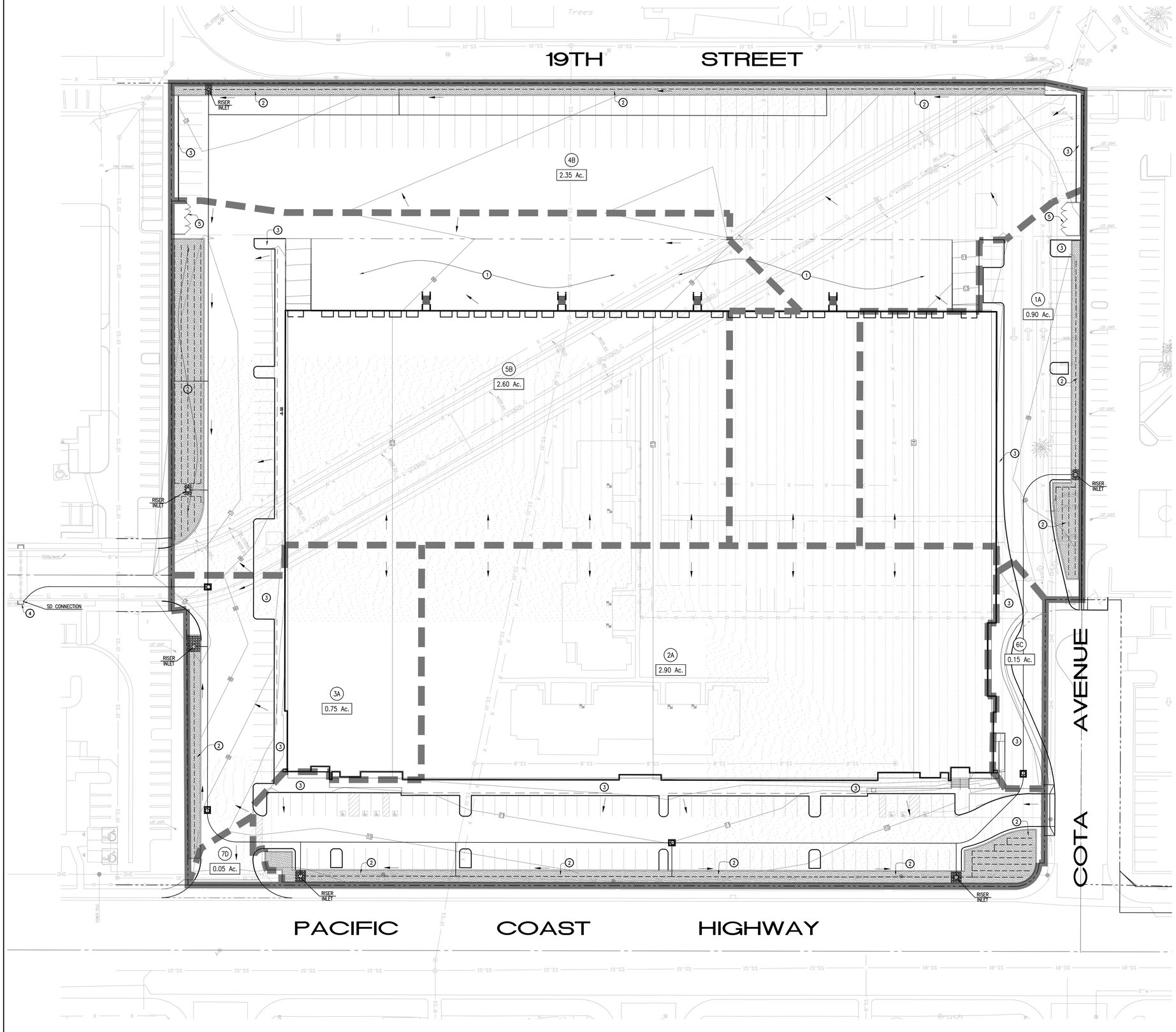
PREPARED BY:
Thienes Engineering, Inc.
CIVIL ENGINEERING • LAND SURVEYING
14349 FIRESTONE BOULEVARD
LA MIRADA, CALIFORNIA 90638
PH:(714)521-4811 FAX:(714)521-4173

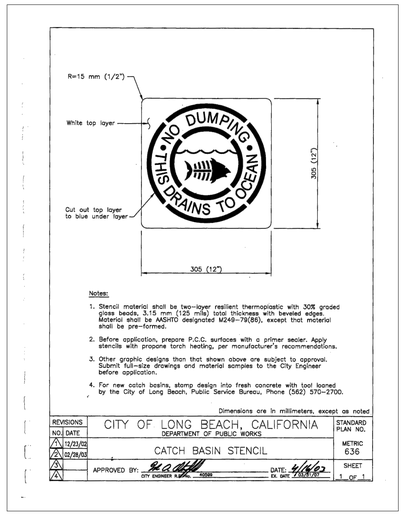
Designed by
Date
Checked by
Date
Designed by
Date
Checked by
Date

Approved by Date
Public Works Director R.C.E. XXXXX

Sheet 1 of 1 Sheets

3572/1 OF 1 SHEET



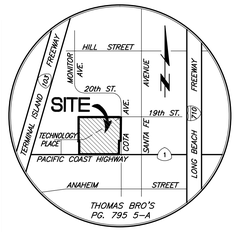


Notes:

1. Stencil material shall be two-layer resilient thermoplastic with 30% graded glass beads, 3.15 mm (125 mils) total thickness with beveled edges. Material shall be AASHTO designated M249-79(86), except that material shall be pre-formed.
2. Before application, prepare P.C.C. surfaces with a primer sealer. Apply stencils with propane torch heating, per manufacture's recommendations.
3. Other graphic designs than that shown above are subject to approval. Submit full-size drawings and material samples to the City Engineer before application.
4. For new catch basins, stamp design into fresh concrete with tool loaned by the City of Long Beach, Public Service Bureau, Phone (562) 570-2700.

Dimensions are in millimeters, except as noted

REVISIONS NO.	DATE	CITY OF LONG BEACH, CALIFORNIA DEPARTMENT OF PUBLIC WORKS	STANDARD PLAN NO.
1	12/21/02	CATCH BASIN STENCIL	METRIC 636
2	02/28/03		
3		APPROVED BY: CITY ENGINEER R.W.Ph. DATE 4/16/03 EX. DIR. PUBLIC WORKS	SHEET 1 OF 1
4			



VICINITY MAP
NOT TO SCALE

LEGEND

1. LOADING/UNLOADING DOCKS
2. PLANTER BOX (FLOW-THROUGH BIOFILTRATION)
3. LANDSCAPE
4. CATCH BASIN STENCIL "NO DUMPING-THIS DRAINS TO OCEAN"
5. TRASH ENCLOSURE

RD ROOF DRAIN
BOUNDARY
DRAINAGE AREAS
FLOW LINE

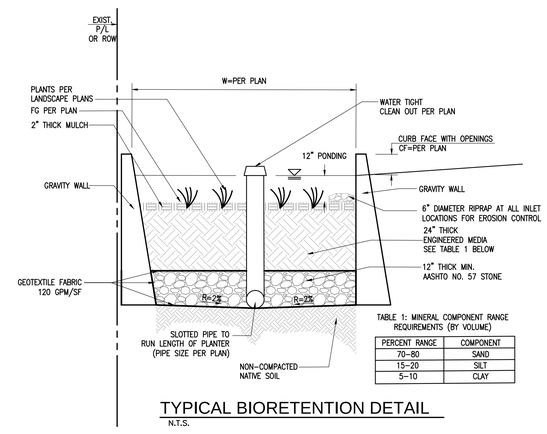


TABLE 1: MINERAL COMPONENT RANGE REQUIREMENTS (BY VOLUME)

PERCENT RANGE	COMPONENT
70-80	SAND
15-20	SILT
5-10	CLAY

TYPICAL BIORETENTION DETAIL
N.T.S.

0 15 30 60 90
SCALE:1"=30'

PREPARED FOR:
PROLOGIS
3546 CONCOURS STREET, SUITE 100
ONTARIO, CA 91764
PHONE: (909) 673-8700
FAX: (909) 673-8703

CIVIL ENGINEER
Thienes Engineering, Inc.
CIVIL ENGINEERING • LAND SURVEYING
14349 FIRESTONE BOULEVARD
LA MIRADA, CALIFORNIA 90638
PH.(714)521-4811 FAX(714)521-4173

CITY OF LONG BEACH
PRELIMINARY
LOW IMPACT DEVELOPMENT PLAN
CSULB TECHNOLOGY PARK III
1924 TECHNOLOGY PLACE

Designed by
Date
Sheet 4 of 4 Sheets

JN 3572

Last Update:7/27/17
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APPENDIX B

HYDROLOGY CALCULATIONS

Peak Flow Hydrologic Analysis

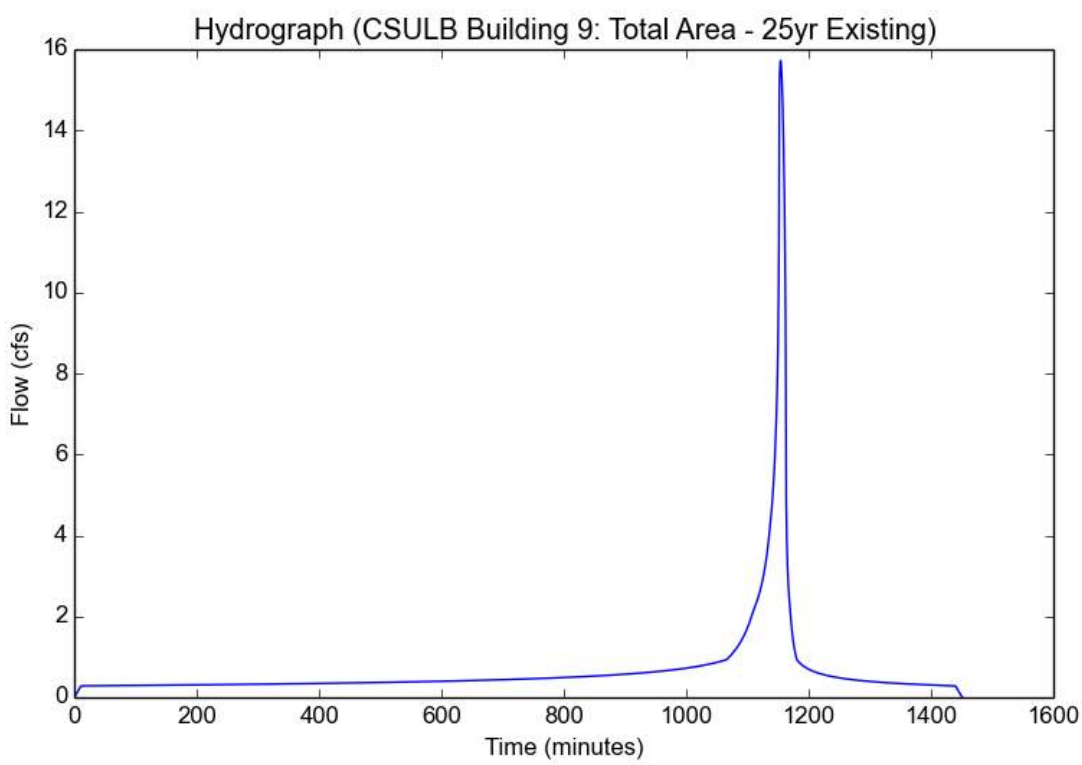
File location: //fuscoe.corp/IRV/Panzura/Projects/309/019/_Support Files/Reports/EIR Tech Reports/MND/Hydrology and WQ Report/HydroCalc/CSULB
Version: HydroCalc 1.0.2

Input Parameters

Project Name	CSULB Building 9
Subarea ID	Total Area - 25yr Existing
Area (ac)	9.68
Flow Path Length (ft)	800.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	5.8
Percent Impervious	0.2
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.0924
Peak Intensity (in/hr)	2.0974
Undeveloped Runoff Coefficient (Cu)	0.743
Developed Runoff Coefficient (Cd)	0.7744
Time of Concentration (min)	11.0
Clear Peak Flow Rate (cfs)	15.7225
Burned Peak Flow Rate (cfs)	15.7225
24-Hr Clear Runoff Volume (ac-ft)	1.3681
24-Hr Clear Runoff Volume (cu-ft)	59595.4586



Peak Flow Hydrologic Analysis

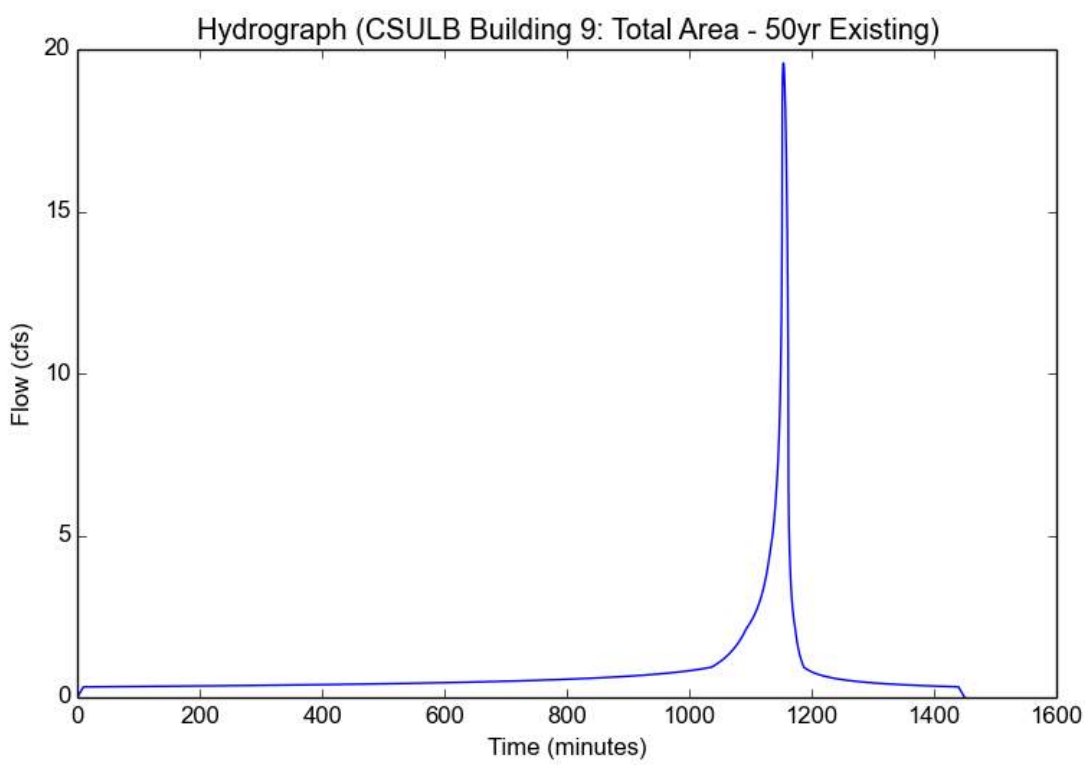
File location: //fuscoe.corp/IRV/Panzura/Projects/309/019/_Support Files/Reports/EIR Tech Reports/MND/Hydrology and WQ Report/HydroCalc/CSULB
Version: HydroCalc 1.0.2

Input Parameters

Project Name	CSULB Building 9
Subarea ID	Total Area - 50yr Existing
Area (ac)	9.68
Flow Path Length (ft)	800.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	5.8
Percent Impervious	0.2
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.8
Peak Intensity (in/hr)	2.4983
Undeveloped Runoff Coefficient (Cu)	0.787
Developed Runoff Coefficient (Cd)	0.8096
Time of Concentration (min)	10.0
Clear Peak Flow Rate (cfs)	19.5789
Burned Peak Flow Rate (cfs)	19.5789
24-Hr Clear Runoff Volume (ac-ft)	1.6178
24-Hr Clear Runoff Volume (cu-ft)	70471.267



Peak Flow Hydrologic Analysis

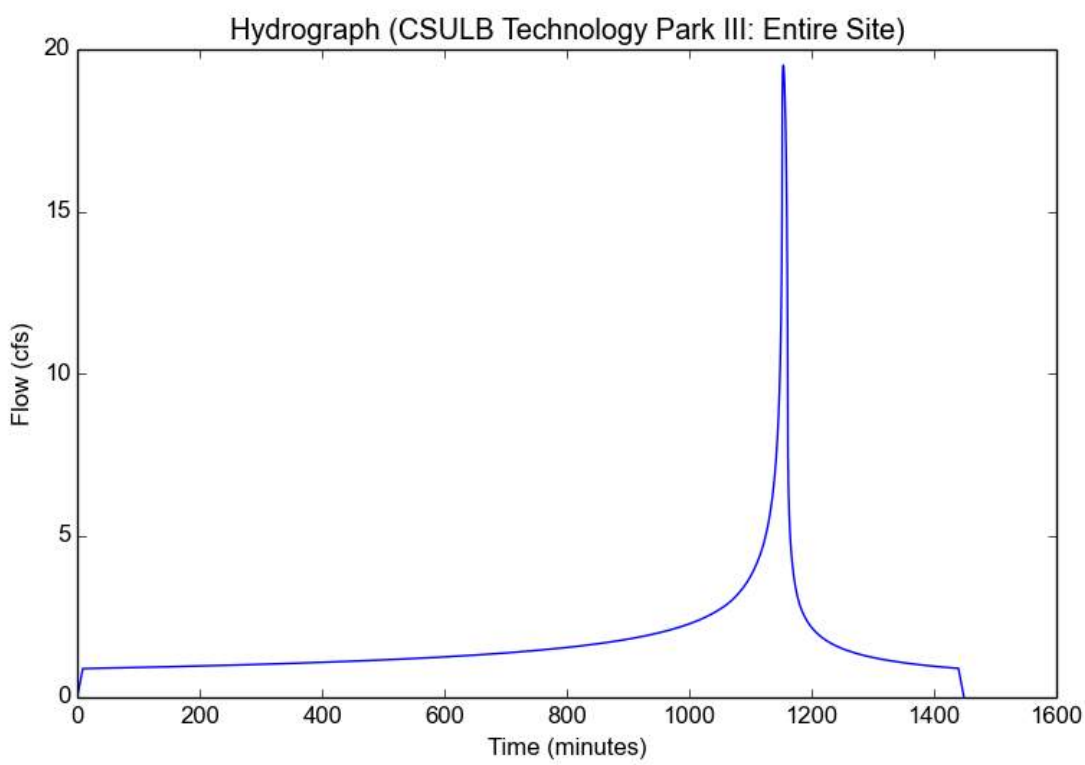
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (25-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	Entire Site
Area (ac)	9.5
Flow Path Length (ft)	650.0
Flow Path Slope (vft/hft)	0.009
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.11
Peak Intensity (in/hr)	2.3128
Undeveloped Runoff Coefficient (Cu)	0.7666
Developed Runoff Coefficient (Cd)	0.888
Time of Concentration (min)	9.0
Clear Peak Flow Rate (cfs)	19.5109
Burned Peak Flow Rate (cfs)	19.5109
24-Hr Clear Runoff Volume (ac-ft)	3.3564
24-Hr Clear Runoff Volume (cu-ft)	146206.1757



Peak Flow Hydrologic Analysis

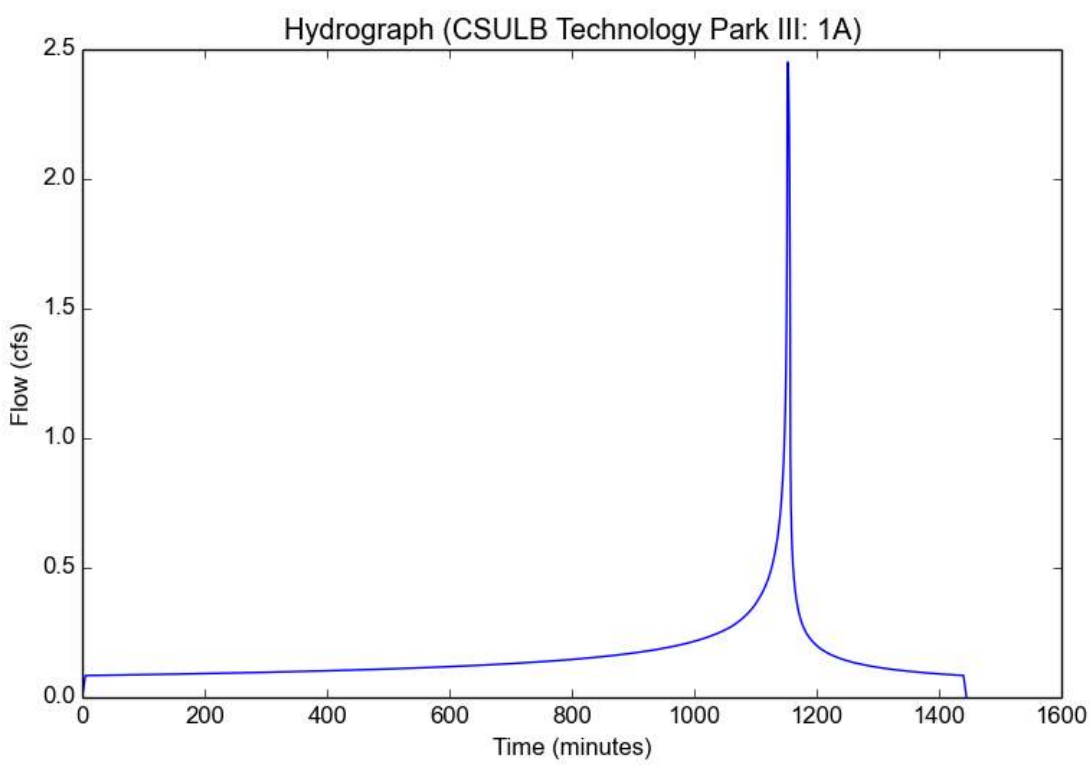
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (25-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	1A
Area (ac)	0.9
Flow Path Length (ft)	215.0
Flow Path Slope (vft/hft)	0.009
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.11
Peak Intensity (in/hr)	3.0487
Undeveloped Runoff Coefficient (Cu)	0.8257
Developed Runoff Coefficient (Cd)	0.8933
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.4511
Burned Peak Flow Rate (cfs)	2.4511
24-Hr Clear Runoff Volume (ac-ft)	0.318
24-Hr Clear Runoff Volume (cu-ft)	13852.8062



Peak Flow Hydrologic Analysis

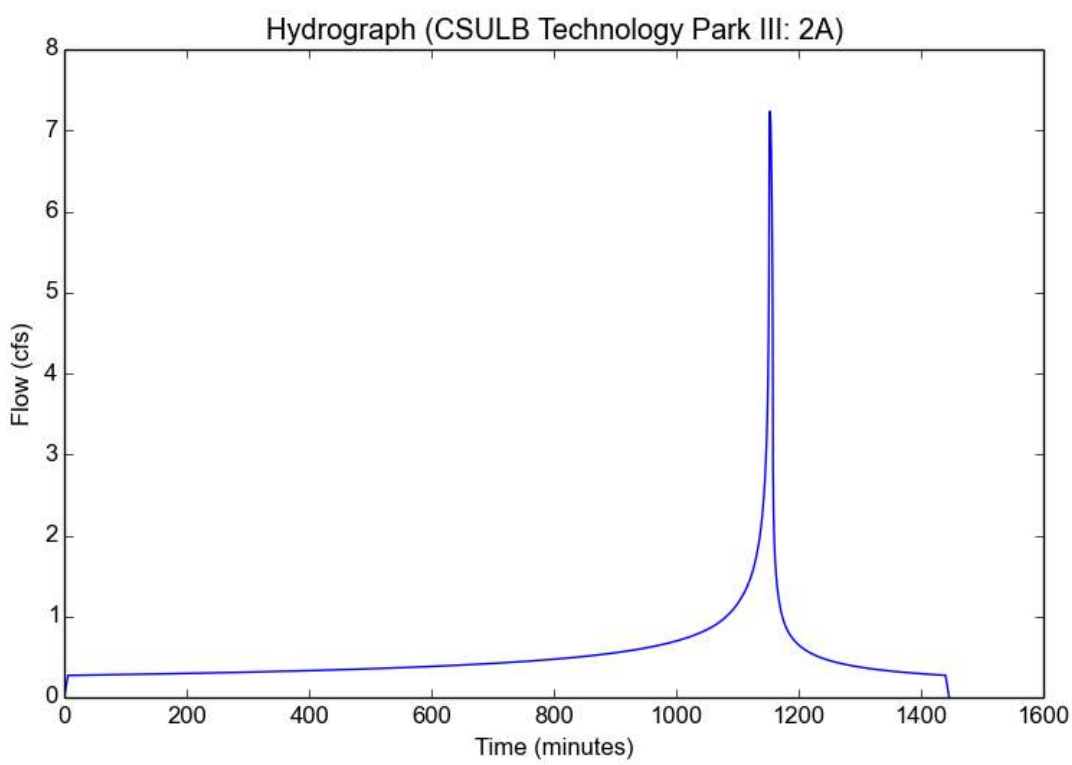
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (25-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	2A
Area (ac)	2.9
Flow Path Length (ft)	320.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.11
Peak Intensity (in/hr)	2.7984
Undeveloped Runoff Coefficient (Cu)	0.8081
Developed Runoff Coefficient (Cd)	0.8917
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	7.2366
Burned Peak Flow Rate (cfs)	7.2366
24-Hr Clear Runoff Volume (ac-ft)	1.0247
24-Hr Clear Runoff Volume (cu-ft)	44635.495



Peak Flow Hydrologic Analysis

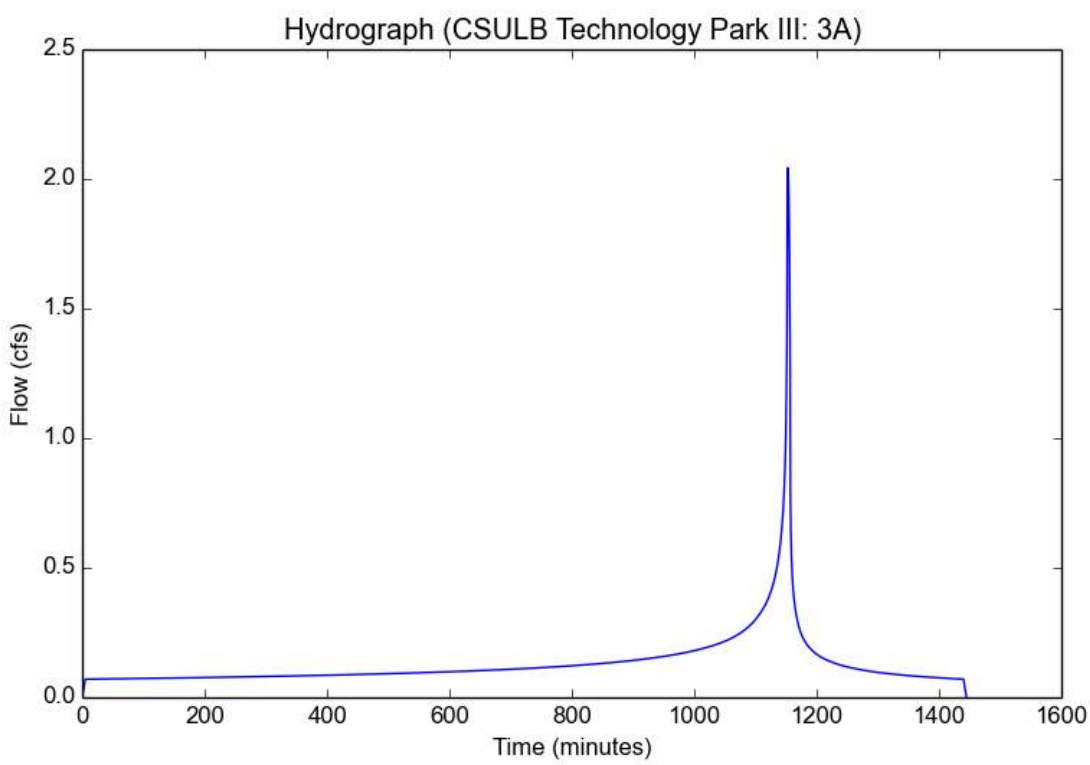
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (25-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	3A
Area (ac)	0.75
Flow Path Length (ft)	150.0
Flow Path Slope (vft/hft)	0.026
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.11
Peak Intensity (in/hr)	3.0487
Undeveloped Runoff Coefficient (Cu)	0.8257
Developed Runoff Coefficient (Cd)	0.8933
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.0426
Burned Peak Flow Rate (cfs)	2.0426
24-Hr Clear Runoff Volume (ac-ft)	0.265
24-Hr Clear Runoff Volume (cu-ft)	11544.0052



Peak Flow Hydrologic Analysis

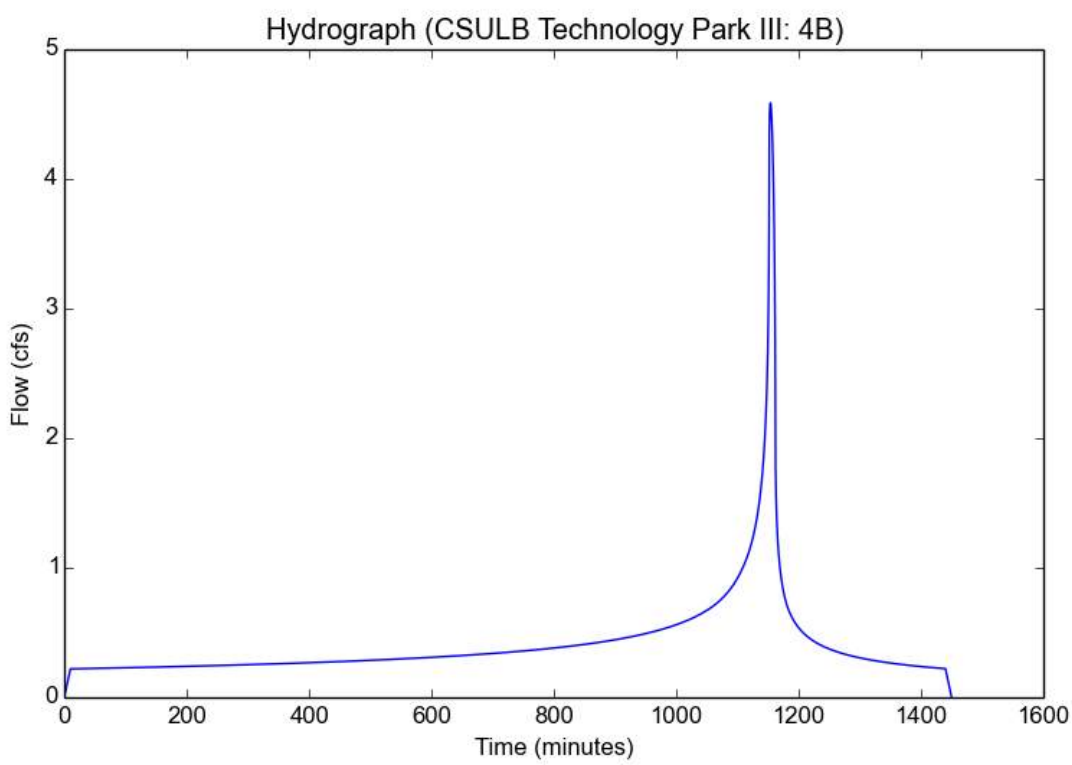
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (25-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	4B
Area (ac)	2.35
Flow Path Length (ft)	640.0
Flow Path Slope (vft/hft)	0.006
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.11
Peak Intensity (in/hr)	2.2011
Undeveloped Runoff Coefficient (Cu)	0.7544
Developed Runoff Coefficient (Cd)	0.8869
Time of Concentration (min)	10.0
Clear Peak Flow Rate (cfs)	4.5875
Burned Peak Flow Rate (cfs)	4.5875
24-Hr Clear Runoff Volume (ac-ft)	0.8303
24-Hr Clear Runoff Volume (cu-ft)	36165.7986



Peak Flow Hydrologic Analysis

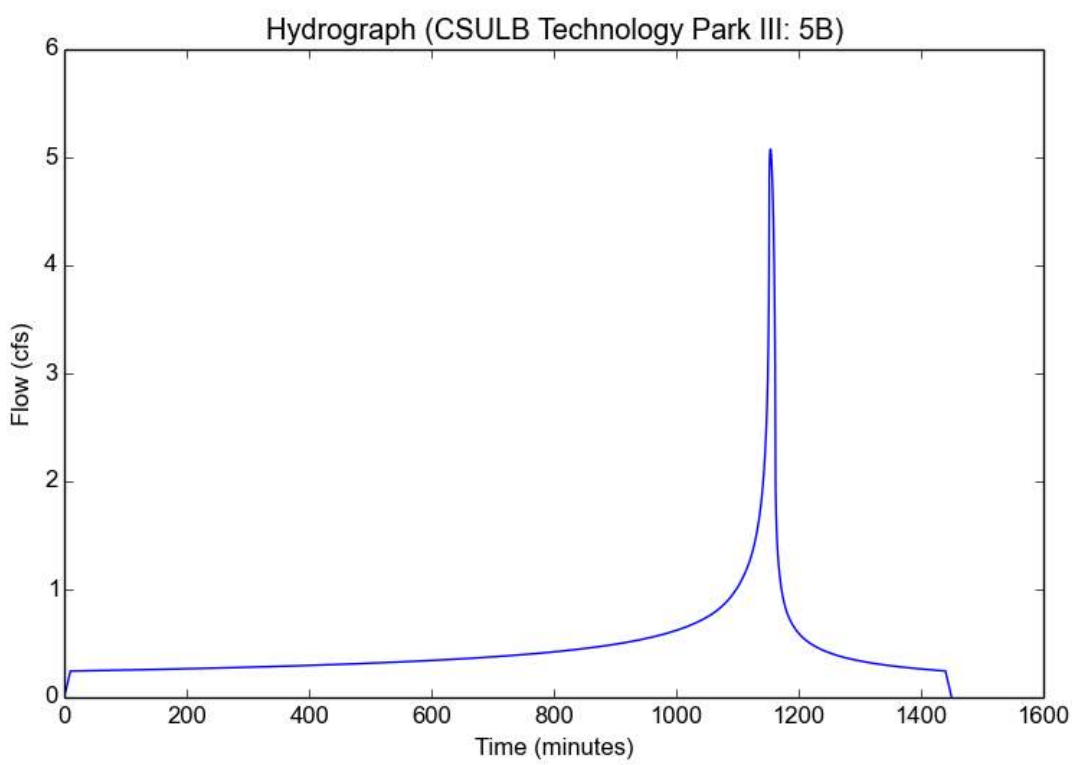
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (25-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	5B
Area (ac)	2.6
Flow Path Length (ft)	650.0
Flow Path Slope (vft/hft)	0.005
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.11
Peak Intensity (in/hr)	2.2011
Undeveloped Runoff Coefficient (Cu)	0.7544
Developed Runoff Coefficient (Cd)	0.8869
Time of Concentration (min)	10.0
Clear Peak Flow Rate (cfs)	5.0755
Burned Peak Flow Rate (cfs)	5.0755
24-Hr Clear Runoff Volume (ac-ft)	0.9186
24-Hr Clear Runoff Volume (cu-ft)	40013.224



Peak Flow Hydrologic Analysis

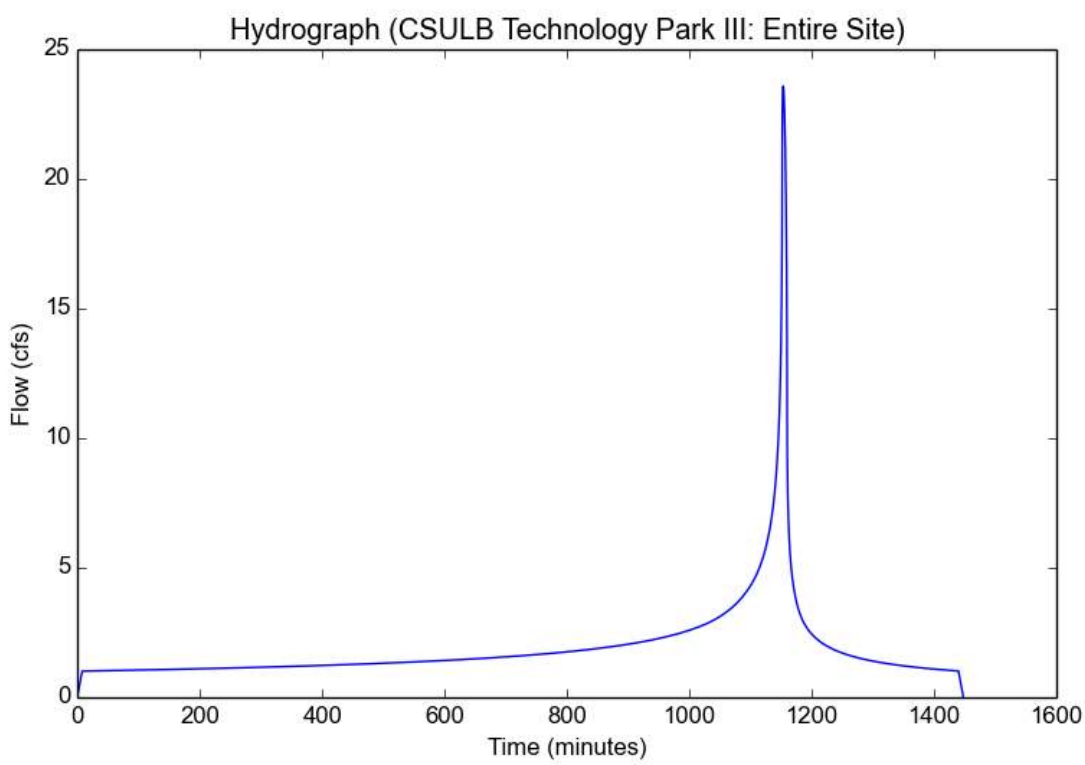
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (50-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	Entire Site
Area (ac)	9.5
Flow Path Length (ft)	650.0
Flow Path Slope (vft/hft)	0.009
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.82
Peak Intensity (in/hr)	2.7841
Undeveloped Runoff Coefficient (Cu)	0.8071
Developed Runoff Coefficient (Cd)	0.8916
Time of Concentration (min)	8.0
Clear Peak Flow Rate (cfs)	23.5832
Burned Peak Flow Rate (cfs)	23.5832
24-Hr Clear Runoff Volume (ac-ft)	3.8294
24-Hr Clear Runoff Volume (cu-ft)	166810.1541



Peak Flow Hydrologic Analysis

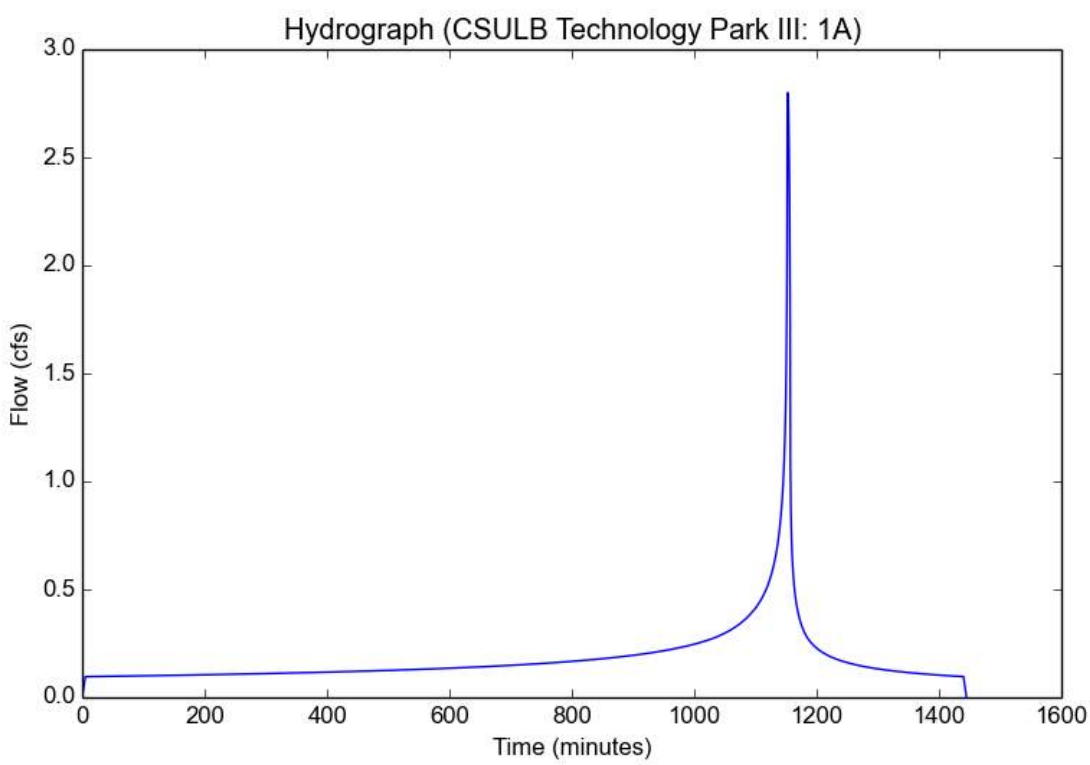
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (50-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	1A
Area (ac)	0.9
Flow Path Length (ft)	215.0
Flow Path Slope (vft/hft)	0.009
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.82
Peak Intensity (in/hr)	3.4724
Undeveloped Runoff Coefficient (Cu)	0.8554
Developed Runoff Coefficient (Cd)	0.896
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.8001
Burned Peak Flow Rate (cfs)	2.8001
24-Hr Clear Runoff Volume (ac-ft)	0.3628
24-Hr Clear Runoff Volume (cu-ft)	15804.5261



Peak Flow Hydrologic Analysis

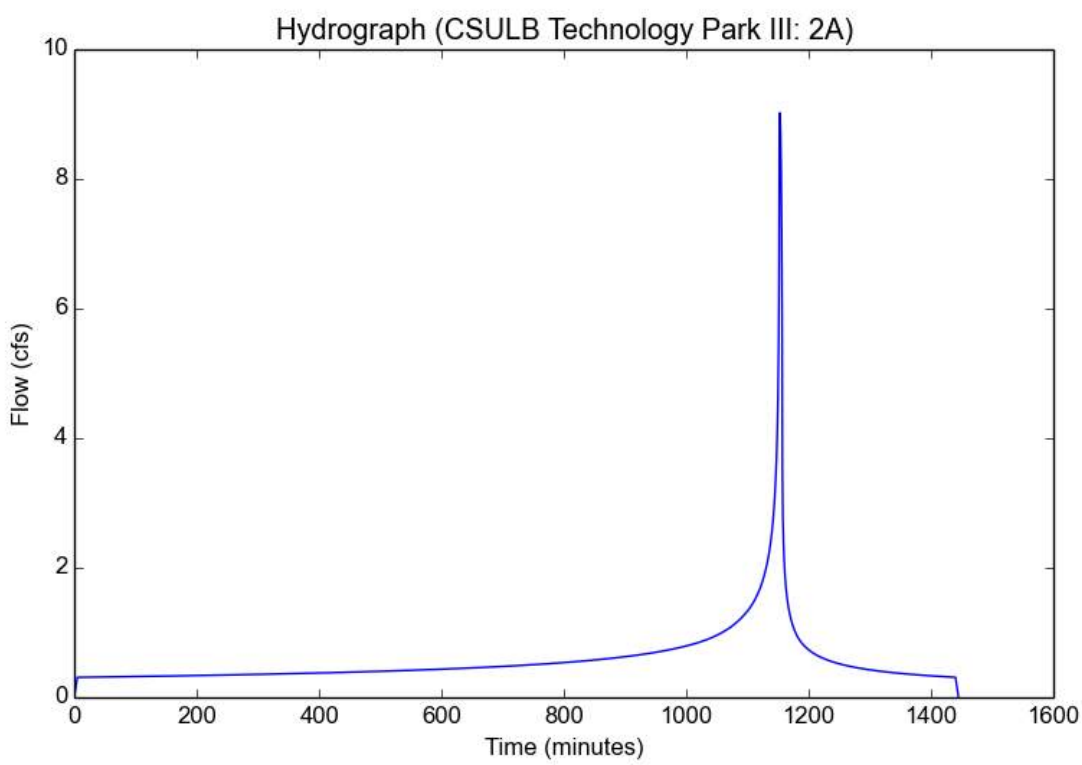
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (50-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	2A
Area (ac)	2.9
Flow Path Length (ft)	320.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.82
Peak Intensity (in/hr)	3.4724
Undeveloped Runoff Coefficient (Cu)	0.8554
Developed Runoff Coefficient (Cd)	0.896
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	9.0225
Burned Peak Flow Rate (cfs)	9.0225
24-Hr Clear Runoff Volume (ac-ft)	1.1691
24-Hr Clear Runoff Volume (cu-ft)	50925.6952



Peak Flow Hydrologic Analysis

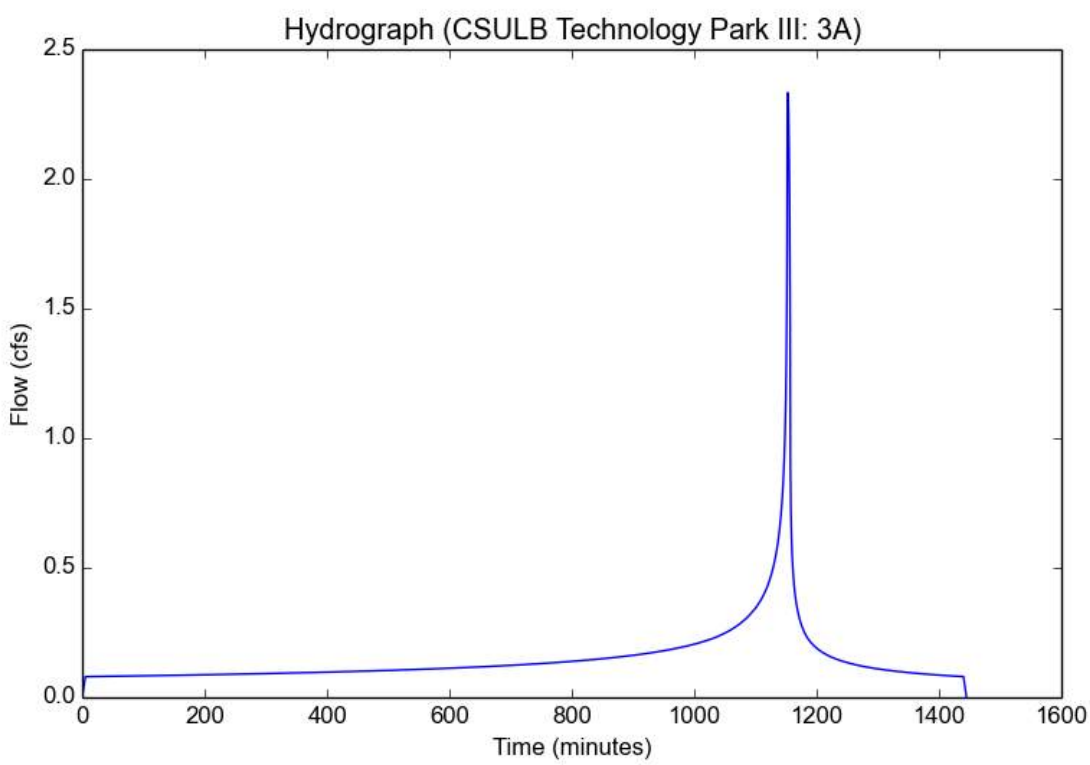
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (50-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	3A
Area (ac)	0.75
Flow Path Length (ft)	150.0
Flow Path Slope (vft/hft)	0.026
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.82
Peak Intensity (in/hr)	3.4724
Undeveloped Runoff Coefficient (Cu)	0.8554
Developed Runoff Coefficient (Cd)	0.896
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	2.3334
Burned Peak Flow Rate (cfs)	2.3334
24-Hr Clear Runoff Volume (ac-ft)	0.3024
24-Hr Clear Runoff Volume (cu-ft)	13170.4384



Peak Flow Hydrologic Analysis

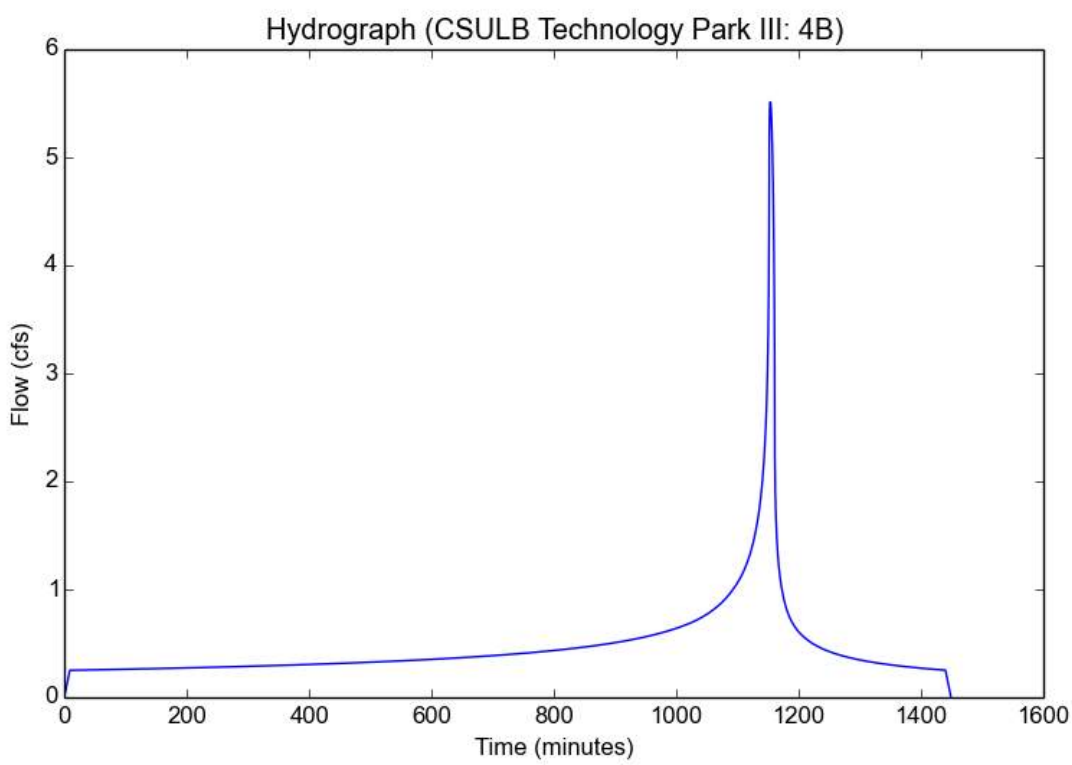
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (50-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	4B
Area (ac)	2.35
Flow Path Length (ft)	640.0
Flow Path Slope (vft/hft)	0.006
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.82
Peak Intensity (in/hr)	2.6342
Undeveloped Runoff Coefficient (Cu)	0.7966
Developed Runoff Coefficient (Cd)	0.8907
Time of Concentration (min)	9.0
Clear Peak Flow Rate (cfs)	5.5137
Burned Peak Flow Rate (cfs)	5.5137
24-Hr Clear Runoff Volume (ac-ft)	0.9473
24-Hr Clear Runoff Volume (cu-ft)	41262.6192



Peak Flow Hydrologic Analysis

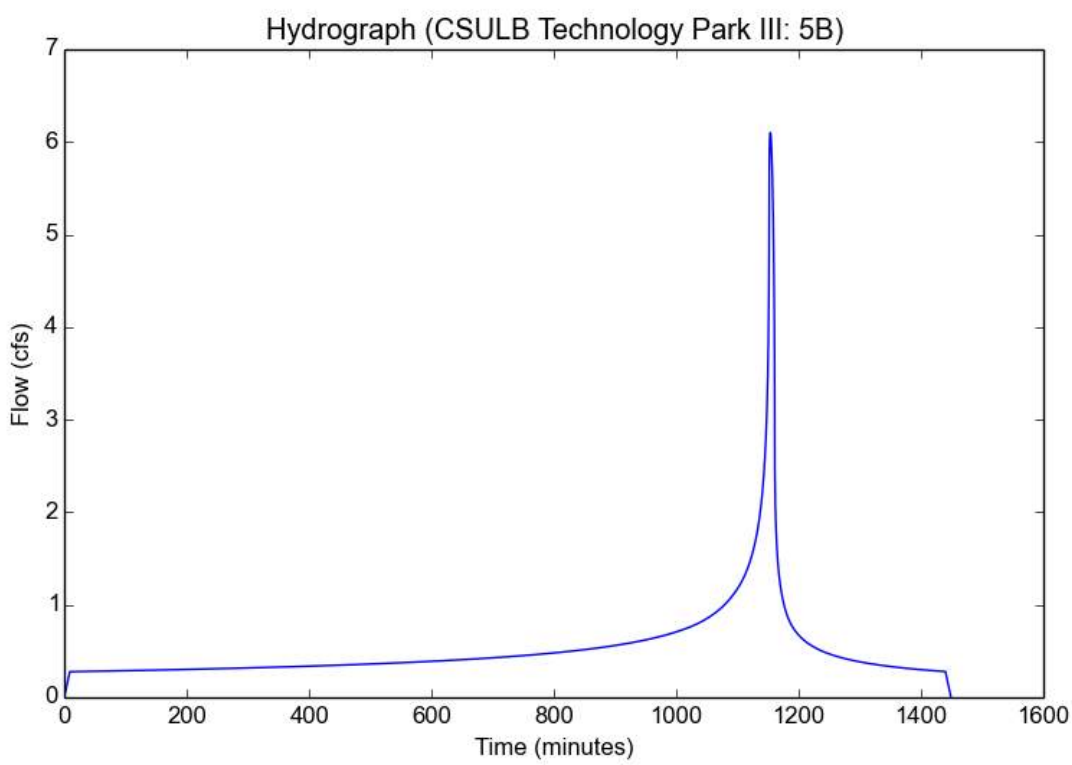
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (50-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	5B
Area (ac)	2.6
Flow Path Length (ft)	650.0
Flow Path Slope (vft/hft)	0.005
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.82
Peak Intensity (in/hr)	2.6342
Undeveloped Runoff Coefficient (Cu)	0.7966
Developed Runoff Coefficient (Cd)	0.8907
Time of Concentration (min)	9.0
Clear Peak Flow Rate (cfs)	6.1003
Burned Peak Flow Rate (cfs)	6.1003
24-Hr Clear Runoff Volume (ac-ft)	1.048
24-Hr Clear Runoff Volume (cu-ft)	45652.2595



Peak Flow Hydrologic Analysis

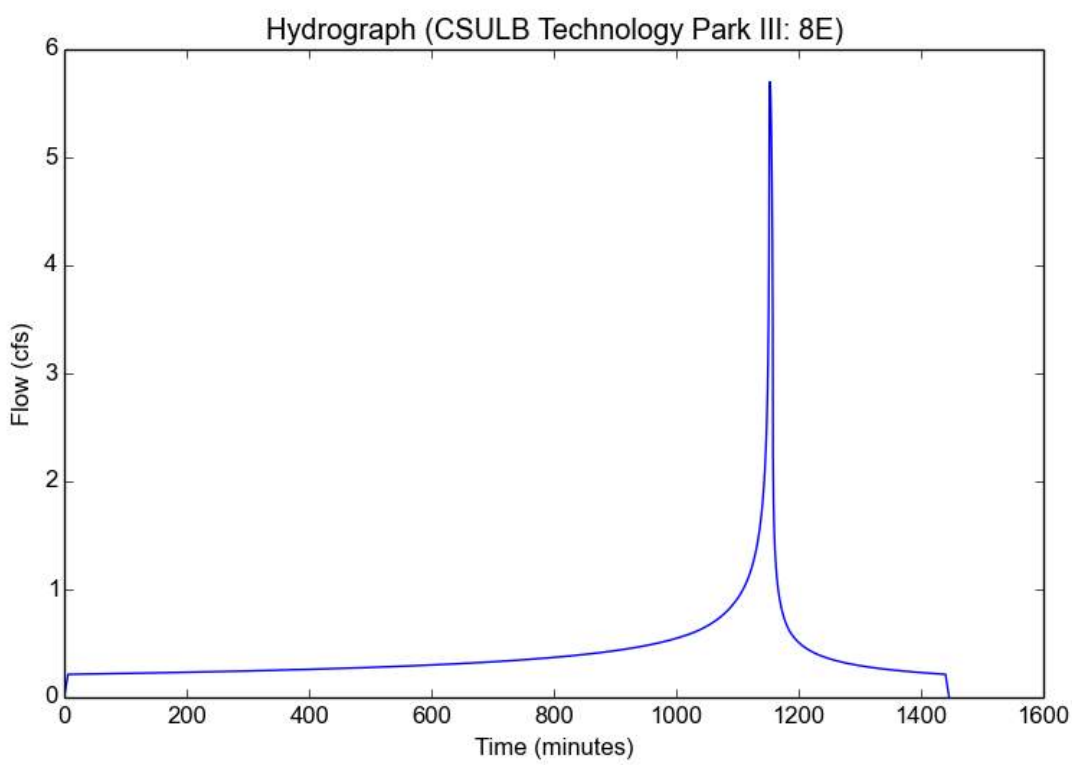
File location: O:/3500-3599/3572/HydroCalc/CSULB Technology Park III Report (Offsite 50-year).pdf
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	8E
Area (ac)	2.0
Flow Path Length (ft)	300.0
Flow Path Slope (vft/hft)	0.005
50-yr Rainfall Depth (in)	5.82
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	50-yr
Fire Factor	0
LID	False

Output Results

Modeled (50-yr) Rainfall Depth (in)	5.82
Peak Intensity (in/hr)	3.1872
Undeveloped Runoff Coefficient (Cu)	0.8354
Developed Runoff Coefficient (Cd)	0.8942
Time of Concentration (min)	6.0
Clear Peak Flow Rate (cfs)	5.6999
Burned Peak Flow Rate (cfs)	5.6999
24-Hr Clear Runoff Volume (ac-ft)	0.8062
24-Hr Clear Runoff Volume (cu-ft)	35119.8603



APPENDIX C

WATER QUALITY CALCULATIONS

Department of Public Works
dpw.lacounty.gov

search our site..

Hydrology Map

A GIS viewer application to view the data for the hydrology manual.

LAYERS

- [ ] 50yr Two Tenths (Rainfall)
- [ ] DPA Zones
- [ ] Soils 2004
- [ ] TG Page
- [x] Final 85th Percentile, 24-hr Rainfall
- [ ] Final 95th Percentile, 24-hr Rainfall
- [ ] 1-year, 1-hour Rainfall Intensity

SEARCH

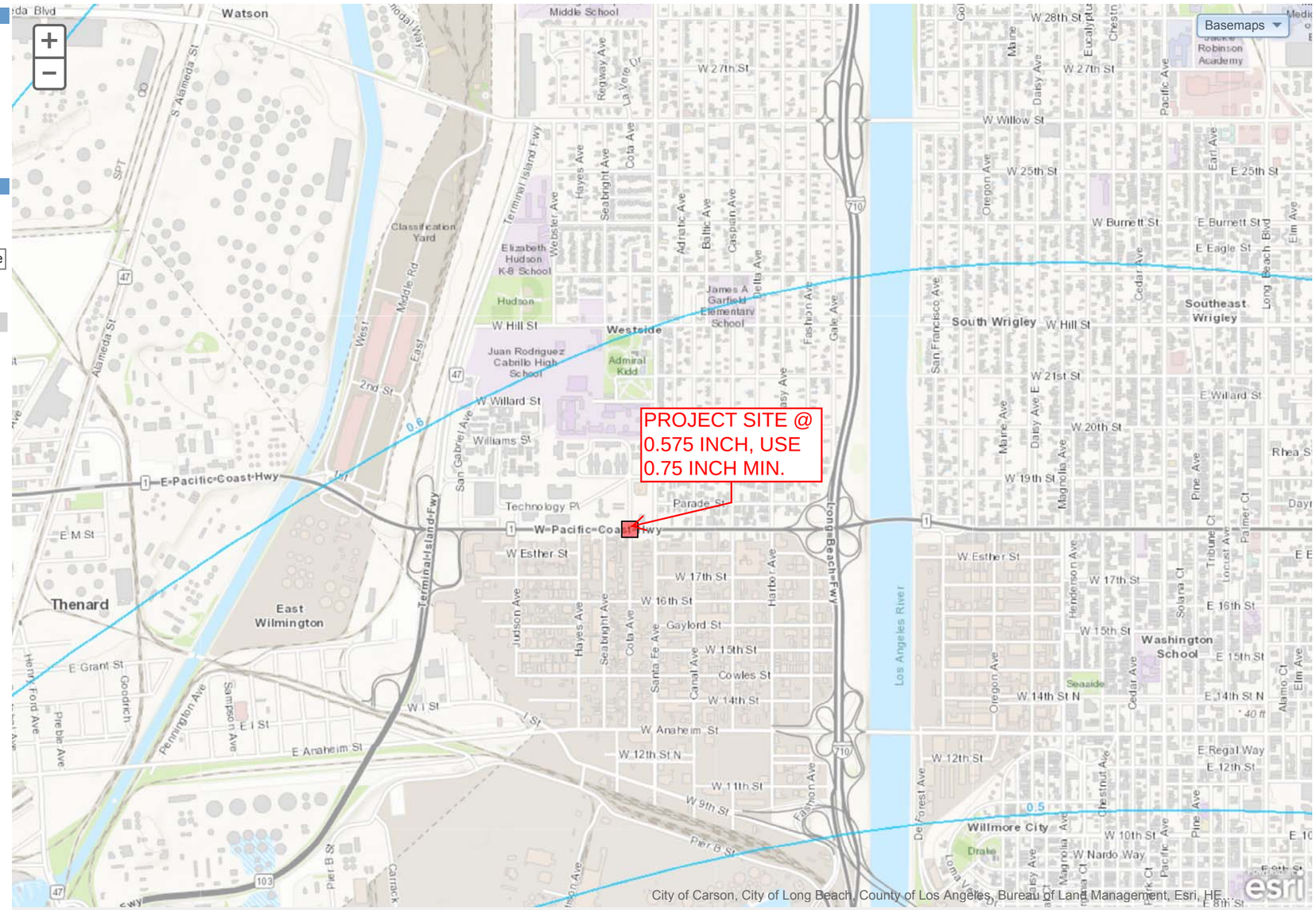
Zoom to TG Page:

Enter Address, Cross Street, or Parcel No.:
(ex: 900 S. Fremont Ave., Fremont@Valley, 5342005904) pacific coast highway and cota ave

Search

Address Search Results:

pacific coast highway and cota ave long beach



Map Tips

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Peak Flow Hydrologic Analysis

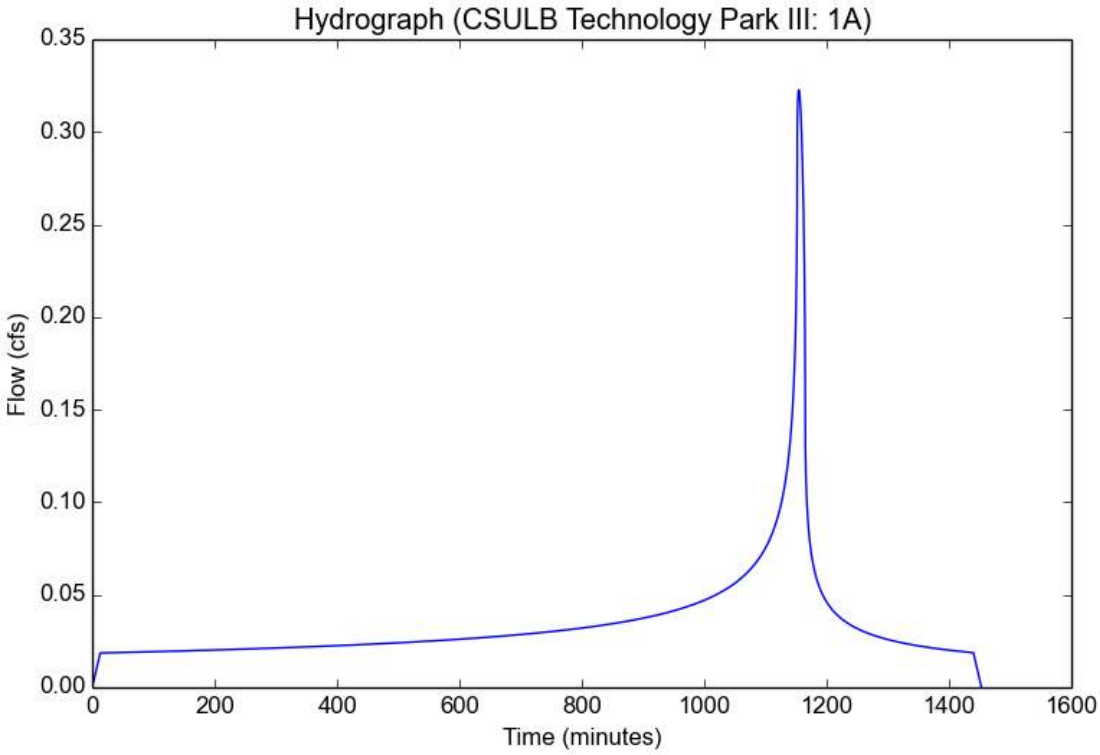
File location: O:/3500-3599/3572/SUSMP/2017-08-04 Preliminary LID to Fuscoe (91% Imp.)/Appendices/Appendix A - SWQDv/HYDROCALC/CSULB Te
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	1A
Area (ac)	0.9
Flow Path Length (ft)	215.0
Flow Path Slope (vft/hft)	0.009
85th Percentile Rainfall Depth (in)	1.125
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.125
Peak Intensity (in/hr)	0.4284
Undeveloped Runoff Coefficient (Cu)	0.2003
Developed Runoff Coefficient (Cd)	0.837
Time of Concentration (min)	13.0
Clear Peak Flow Rate (cfs)	0.3227
Burned Peak Flow Rate (cfs)	0.3227
24-Hr Clear Runoff Volume (ac-ft)	0.0693
24-Hr Clear Runoff Volume (cu-ft)	3019.3983



PLANTER BOX CALCULATIONS

SUBAREA 1A

SWQDv (1.5x)	3,019	
k-sat, media	5	in/hr
FS	2	
k-sat, design	2.5	in/hr
Surface drawdown	48	hours
dp	10.00	ft
dp max	12	in
A-min required	1,858	sq-ft
A-designed	2,230	sq-ft

Peak Flow Hydrologic Analysis

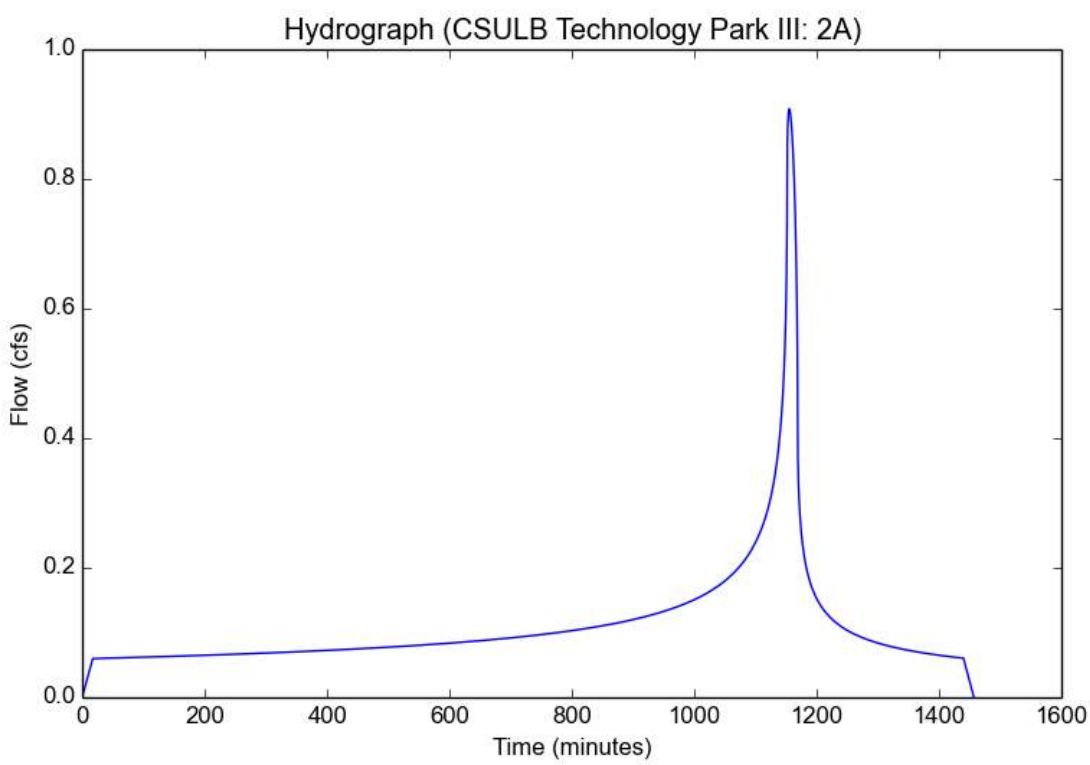
File location: O:/3500-3599/3572/SUSMP/2017-08-04 Preliminary LID to Fuscoe (91% Imp.)/Appendices/Appendix A - SWQDv/HYDROCALC/CSULB Te
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	2A
Area (ac)	2.9
Flow Path Length (ft)	320.0
Flow Path Slope (vft/hft)	0.01
85th Percentile Rainfall Depth (in)	1.125
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.125
Peak Intensity (in/hr)	0.3776
Undeveloped Runoff Coefficient (Cu)	0.1131
Developed Runoff Coefficient (Cd)	0.8292
Time of Concentration (min)	17.0
Clear Peak Flow Rate (cfs)	0.908
Burned Peak Flow Rate (cfs)	0.908
24-Hr Clear Runoff Volume (ac-ft)	0.2233
24-Hr Clear Runoff Volume (cu-ft)	9725.1732



Peak Flow Hydrologic Analysis

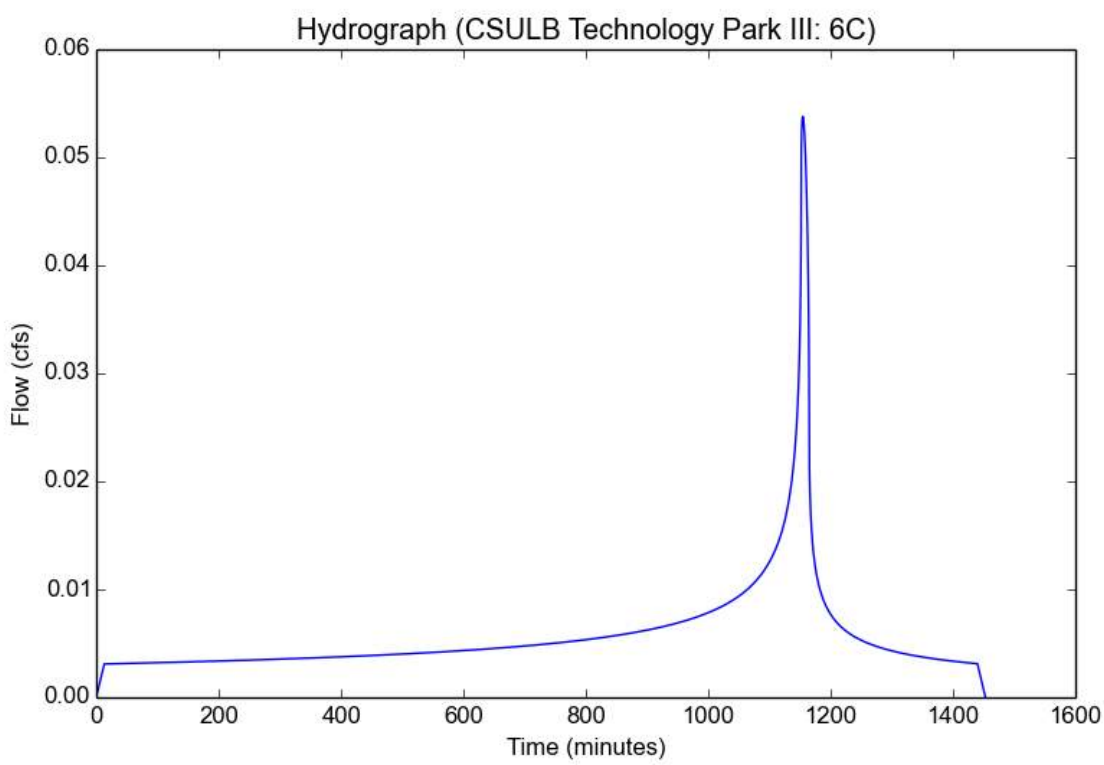
File location: O:/3500-3599/3572/SUSMP/2017-08-04 Preliminary LID to Fuscoe (91% Imp.)/Appendices/Appendix A - SWQDv/HYDROCALC/CSULB Te
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	6C
Area (ac)	0.15
Flow Path Length (ft)	170.0
Flow Path Slope (vft/hft)	0.004
85th Percentile Rainfall Depth (in)	1.125
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.125
Peak Intensity (in/hr)	0.4284
Undeveloped Runoff Coefficient (Cu)	0.2003
Developed Runoff Coefficient (Cd)	0.837
Time of Concentration (min)	13.0
Clear Peak Flow Rate (cfs)	0.0538
Burned Peak Flow Rate (cfs)	0.0538
24-Hr Clear Runoff Volume (ac-ft)	0.0116
24-Hr Clear Runoff Volume (cu-ft)	503.233



Peak Flow Hydrologic Analysis

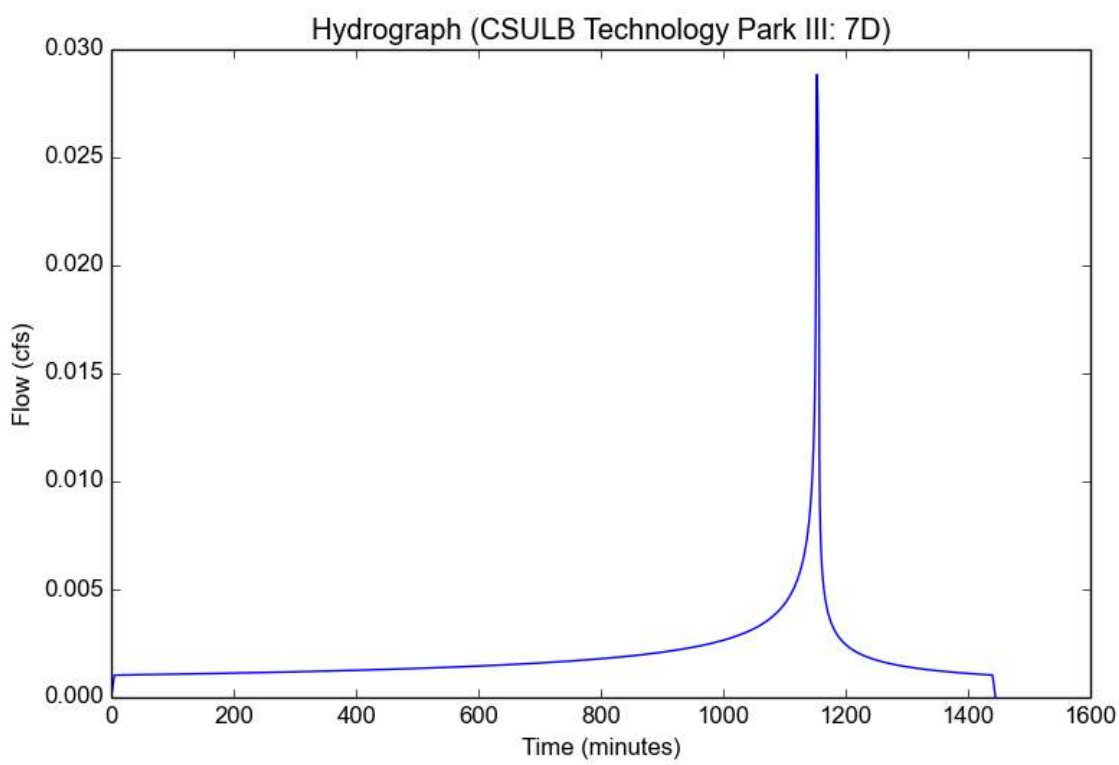
File location: O:/3500-3599/3572/SUSMP/2017-08-04 Preliminary LID to Fuscoe (91% Imp.)/Appendices/Appendix A - SWQDv/HYDROCALC/CSULB Te
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	7D
Area (ac)	0.05
Flow Path Length (ft)	50.0
Flow Path Slope (vft/hft)	0.04
85th Percentile Rainfall Depth (in)	1.125
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.125
Peak Intensity (in/hr)	0.6712
Undeveloped Runoff Coefficient (Cu)	0.4443
Developed Runoff Coefficient (Cd)	0.859
Time of Concentration (min)	5.0
Clear Peak Flow Rate (cfs)	0.0288
Burned Peak Flow Rate (cfs)	0.0288
24-Hr Clear Runoff Volume (ac-ft)	0.0039
24-Hr Clear Runoff Volume (cu-ft)	167.9338



PLANTER BOX CALCULATIONS

SUBAREAS 2A, 6C, AND 7D*

SWQDv (1.5x)	10,396	
k-sat, media	5	in/hr
FS	2	
k-sat, design	2.5	in/hr
Surface drawdown	48	hours
dp	10.00	ft
dp max	12	in
A-min required	6,398	sq-ft
A-designed	6,595	sq-ft

* Subareas 6C and 7D are drainage areas that drain offsite without physically being routed to a planter box. However, Subarea 2A was oversized to accommodate for 6C and 7D runoff as an attempt to mitigate stormwater requirements to the maximum extent practicable.

Peak Flow Hydrologic Analysis

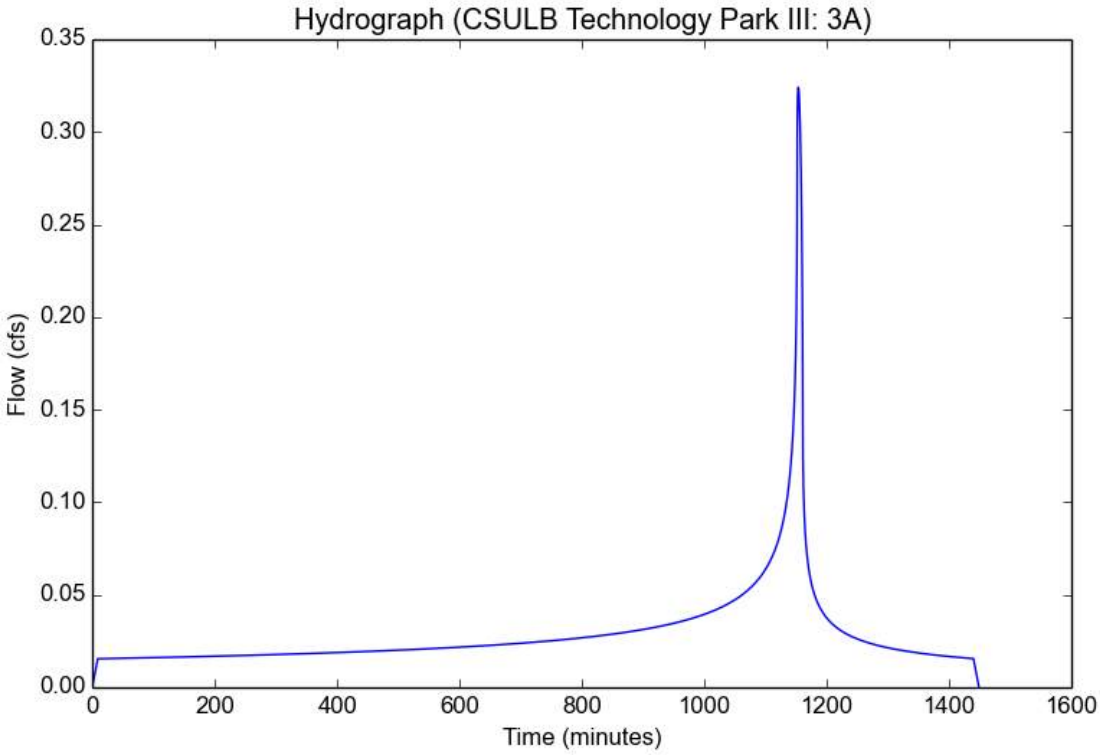
File location: O:/3500-3599/3572/SUSMP/2017-08-04 Preliminary LID to Fuscoe (91% Imp.)/Appendices/Appendix A - SWQDv/HYDROCALC/CSULB Te
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	3A
Area (ac)	0.75
Flow Path Length (ft)	150.0
Flow Path Slope (vft/hft)	0.026
85th Percentile Rainfall Depth (in)	1.125
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.125
Peak Intensity (in/hr)	0.5092
Undeveloped Runoff Coefficient (Cu)	0.3298
Developed Runoff Coefficient (Cd)	0.8487
Time of Concentration (min)	9.0
Clear Peak Flow Rate (cfs)	0.3241
Burned Peak Flow Rate (cfs)	0.3241
24-Hr Clear Runoff Volume (ac-ft)	0.0578
24-Hr Clear Runoff Volume (cu-ft)	2517.8357



PLANTER BOX CALCULATIONS

SUBAREA 3A

SWQDv (1.5x)	2,518	
k-sat, media	5	in/hr
FS	2	
k-sat, design	2.5	in/hr
Surface drawdown	48	hours
dp	10.00	ft
dp max	12	in
A-min required	1,550	sq-ft
A-designed	1,553	sq-ft

Peak Flow Hydrologic Analysis

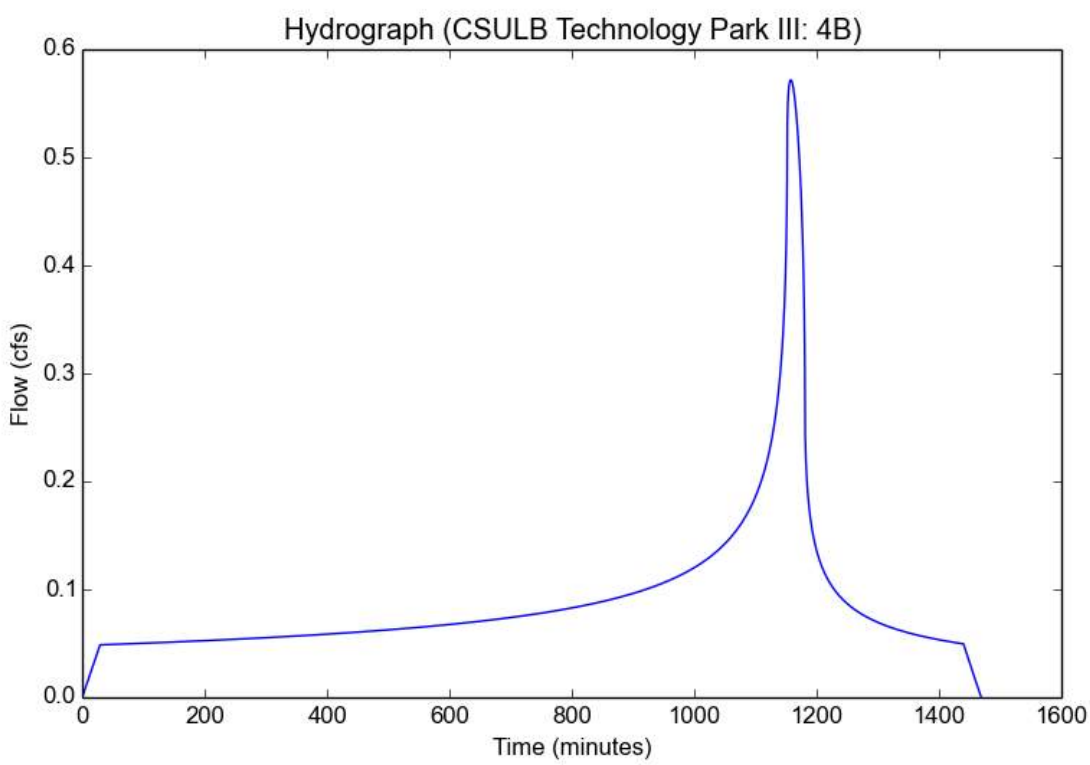
File location: O:/3500-3599/3572/SUSMP/2017-08-04 Preliminary LID to Fuscoe (91% Imp.)/Appendices/Appendix A - SWQDv/HYDROCALC/CSULB Te
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	4B
Area (ac)	2.35
Flow Path Length (ft)	640.0
Flow Path Slope (vft/hft)	0.006
85th Percentile Rainfall Depth (in)	1.125
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.125
Peak Intensity (in/hr)	0.2938
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.828
Time of Concentration (min)	29.0
Clear Peak Flow Rate (cfs)	0.5717
Burned Peak Flow Rate (cfs)	0.5717
24-Hr Clear Runoff Volume (ac-ft)	0.1809
24-Hr Clear Runoff Volume (cu-ft)	7880.5745



PLANTER BOX CALCULATIONS

SUBAREA 4B

SWQDv (1.5x)	7,881	
k-sat, media	5	in/hr
FS	2	
k-sat, design	2.5	in/hr
Surface drawdown	48	hours
dp	10.00	ft
dp max	12	in
A-min required	4,850	sq-ft
A-designed	5,518	sq-ft

Peak Flow Hydrologic Analysis

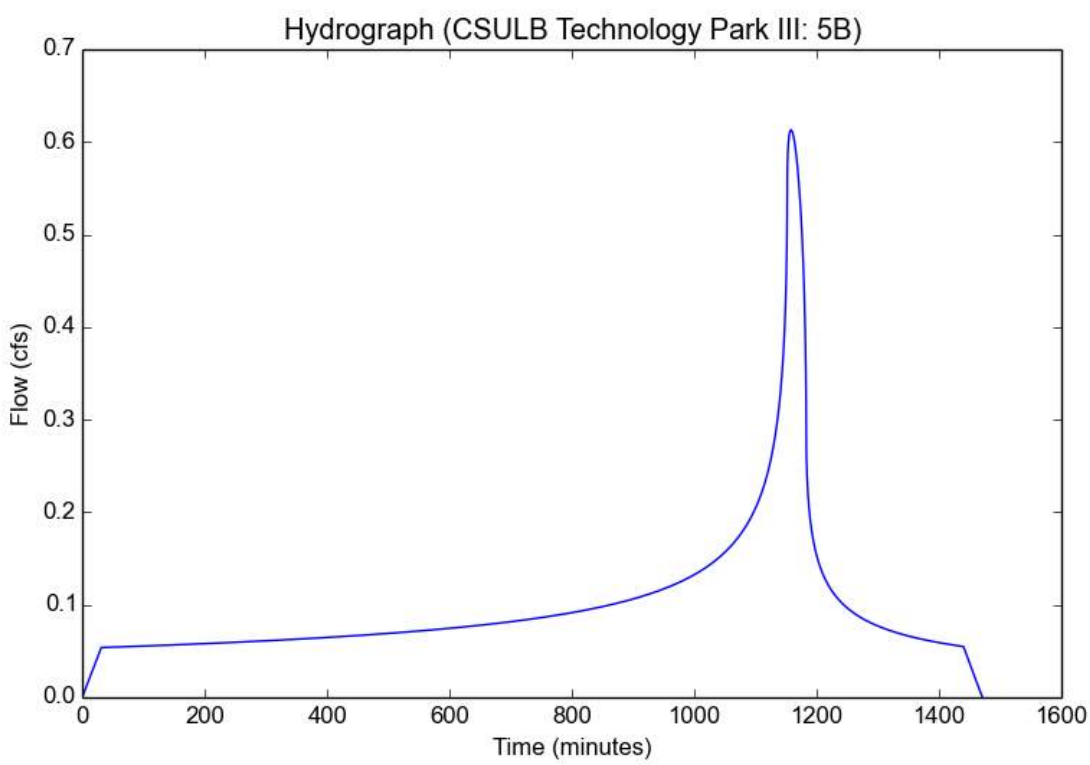
File location: O:/3500-3599/3572/SUSMP/2017-08-04 Preliminary LID to Fuscoe (91% Imp.)/Appendices/Appendix A - SWQDv/HYDROCALC/CSULB Te
Version: HydroCalc 0.3.1-beta

Input Parameters

Project Name	CSULB Technology Park III
Subarea ID	5B
Area (ac)	2.6
Flow Path Length (ft)	650.0
Flow Path Slope (vft/hft)	0.005
85th Percentile Rainfall Depth (in)	1.125
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	85th percentile storm
Fire Factor	0
LID	True

Output Results

Modeled (85th percentile storm) Rainfall Depth (in)	1.125
Peak Intensity (in/hr)	0.2847
Undeveloped Runoff Coefficient (Cu)	0.1
Developed Runoff Coefficient (Cd)	0.828
Time of Concentration (min)	31.0
Clear Peak Flow Rate (cfs)	0.613
Burned Peak Flow Rate (cfs)	0.613
24-Hr Clear Runoff Volume (ac-ft)	0.2002
24-Hr Clear Runoff Volume (cu-ft)	8718.947



PLANTER BOX CALCULATIONS

SUBAREA 5B

SWQDv (1.5x)	8,718	
k-sat, media	5	in/hr
FS	2	
k-sat, design	2.5	in/hr
Surface drawdown	48	hours
dp	10.00	ft
dp max	12	in
A-min required	5,364	sq-ft
A-designed	5,364	sq-ft

APPENDIX D

GEOTECHNICAL FINDINGS

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

NWC Pacific Coast Highway and Cota Avenue
Long Beach, California
for
Prologis

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

May 12, 2016

ProLogis
17777 Center Court Drive N., Suite 100
Cerritos, California, 90703

Attention: Mr. Blake Kelly
Development Manager

Project No.: **16G144-1**

Subject: **Geotechnical Investigation**
Proposed Commercial/Industrial Building
NWC Pacific Coast Highway and Cota Avenue
Long Beach, California

Gentlemen:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

John A. Seminara, CEG 2125
Principal Geologist

Robert G. Trazo, GE 2655
Principal Engineer

Distribution: (1) Addressee

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B Boring Logs
C Laboratory Test Results
D Grading Guide Specifications
E Seismic Design Parameters
F Liquefaction Evaluation Spreadsheets

1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Geotechnical Design Considerations

- The results of the liquefaction evaluation indicate that total dynamic settlements on the order of 7± inches could occur at the site during the design seismic event at the boring locations. The liquefaction induced differential settlements are expected to be on the order of 3 to 4± inches.
- Groundwater was encountered at depths of 9 to 11± feet below existing site grades. Research indicates historic high groundwater is 8 feet below existing grade.
- Several of the borings encountered a surficial layer of fill soils. All of the borings encountered loose/soft native alluvial soils with a moderate to severe potential for consolidation settlement. Low strength, compressible soils are present at the boring locations within the foundation influence zone of the proposed structure.
- While remedial grading could control static settlements to tolerable levels, it would have no effect on the potential liquefaction induced settlements.
- The most feasible method of mitigating potential static and dynamic settlements at this site is considered to be remedial grading of the near surface soils in conjunction with ground improvement of the deeper liquefiable soils. Based on the groundwater levels and the soil conditions, techniques such as deep soil mixing or grout injection are considered the most applicable for ground improvement. A specialty contractor should be contacted for specifics of design build ground improvement methods.
- Ground improvement should be designed to reduce total settlements (static and seismic) to less than 4 inches.
- Based on the relatively shallow depth to groundwater, remedial grading will be limited in depth and the soils exposed at the base of the overexcavation will likely require stabilization due to high moisture contents.
- Most of the overexcavated soils will also require drying prior to reuse in compacted fills, due to high moisture contents.

Site Preparation Recommendations

- Initial site preparation should include stripping of the existing grass and weed growth present at the site. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.
- Demolition of the existing CSULB buildings and the current alignment of Technology Place will be required. Any remnants of the previous improvements, including foundations, slabs, and the debris resulting from demolition activities should be properly disposed of off-site. Concrete and asphalt debris may be re-used within compacted fills, provided they are pulverized and the maximum particle size is less than 2 inches. Such materials should be thoroughly blended with on-site soils prior to use as fill.

- Remedial grading is recommended to be performed within the proposed building pad area in order to provide uniform support conditions, and control static settlements to acceptable levels.
- The proposed building area should be overexcavated to a depth of at least 5 feet below existing grade and to a depth of 5 feet below the proposed building pad subgrade elevation. Within the foundation influence zones, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation should extend horizontally at least 5 feet beyond the building perimeter.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be removed. The resulting subgrade should then be scarified to a depth of 12 inches and moisture conditioned to 2 to 4 percent above optimum.
- Stabilization of the excessively moist soils at the base of the overexcavated areas will likely be required. The previously excavated soils may then be replaced as compacted structural fill provided that they can be dried back to a suitable moisture content. All structural fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom) due to the presence of liquefiable soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 100 psi/in
- Reinforcement consisting of No. 3 rebars at 18 inches on center in both directions due to the presence of liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.

Pavements

ASPHALT PAVEMENTS (R = 50)					
Materials	**Thickness (inches)**				
	Automobile Parking (TI = 4.0)	Automobile Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	3	4	5	5
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS			
Materials	**Thickness (inches)**		
	Automobile and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic	
		(TI = 7.0)	(TI = 8.0)
PCC	5	6	7
Compacted Subgrade (95% minimum compaction)	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 16P171, dated March 8, 2016. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the northwest corner of Pacific Coast Highway and Cota Avenue in Long Beach, California. The site is divided by Technology Place which extends from the west-central boundary to the northeast corner of the site. The site is bordered to the west by CSULB Technology Park, to the north by Long Beach Job Corps Center, to the east by Cota Avenue, and to the south by Pacific Coast Highway.

The site consists of a rectangular shaped parcel, 9.82± acres in size. We understand that the overall site was once developed with U.S. Naval residential barracks. However, the site is currently developed with vacated administrative buildings for Cal State Long Beach. The buildings are located to the south of Technology Place and are single-story structures of wood-frame construction, presumably supported on conventional shallow foundation systems with concrete slab-on-grade floors. Several carport canopies are located in the central region of the site, east of the existing buildings. A sewer line and associated 15 foot wide easement traverse the west central portion of the site in a north-south direction. Ground surface cover surrounding the buildings generally consists of heavy vegetation, with the exception of the asphaltic concrete parking areas. Based on historical photographs obtained from Google Earth, the northern region of the site was previously developed with a parking lot. However, it appears that the parking area was removed by 2009. Ground surface cover in this portion of the site consists of moderate to heavy native grass and weed growth. Several soil stockpiles, measuring 5 to 10± feet in height, encompass the northern region of the site. The stockpiles are covered with heavy vegetation, including native grass and weeds. Several large trees are also located throughout the overall site.

Topographical information for the subject site was obtained from an ALTA survey prepared by Cornerstone Engineering. With the exception of the aforementioned stockpiles, the plan indicates that site grades range from an elevation of 10± feet mean sea level (msl) in the northeast corner of the site to an elevation of 8± feet msl in the southwest portion of the site. Site grades at the top of the stockpiles range from elevations of 15± feet msl to 20± feet msl. With the exception of the soil stockpiles and minor localized variations in topography, site topography generally slopes downward to the southwest at a gradient of less than 1± percent.

3.2 Proposed Development

A conceptual site plan for the proposed development was provided to our office by the client. This plan indicates that the site will be developed with one (1) new commercial/industrial building. The building will be located in the central region of the site and will have a footprint of 195,008± ft^2. Loading docks will be constructed on the north side of the building. The building will be surrounded by asphaltic concrete pavements in the parking and drive lane areas and Portland cement concrete pavements in the loading dock areas.

Detailed structural information has not been provided. It is assumed that the building will be a single-story structure of tilt-up concrete construction, supported on a conventional shallow foundation system with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 80 kips and 3 to 4 kips per linear foot, respectively.

The proposed development is not expected to include any significant amounts of below grade construction such as basements or crawl spaces. Based on the existing site topography and assuming a relatively level site, cuts and fills of up to 3± feet are expected to be necessary to achieve the proposed building pad grades.

3.3 Previous Studies

As part of the preliminary project information, the client provided a geotechnical investigation report prepared by Kleinfelder for the subject site. This report was dated February 28, 2012. The proposed site development at the time of the report included the construction of a 116,800 ft^2 retail store.

The subsurface exploration for the Kleinfelder investigation consisted of thirty-three (33) rotary wash hollow stem borings, seven (7) cone penetration tests (CPTs), and five (5) hand auger borings. The borings were advanced to depths ranging between 4 and 51½± feet and the CPTs were advanced to depths of 60± feet.

The geotechnical conditions identified in the Kleinfelder report are generally similar to the geotechnical conditions encountered for the present investigation. Kleinfelder encountered fill soils consisting of silty sands and sandy silts in the upper 2 to 3 feet, with the exception of the stockpile areas, underlain by low strength alluvial sands, clays, silts, within the upper 40± feet. At greater depths, the alluvium consisted of dense to very dense silty sands and sands.

Laboratory testing performed by Kleinfelder included Moisture Content and Dry Density, Expansion Index testing, Grain Size Distribution, Atterberg Limits (ASTM D-4318), Maximum Dry Density and Optimum Moisture content, and Consolidation (ASTM D-2435).

Kleinfelder also submitted a sample to be analyzed by an analytical laboratory. Kleinfelder utilized the results of these tests to determine the potential corrosivity of the on-site soils. The results of the testing indicated that the soils are classified as corrosive to ferrous metals.

Expansion index testing was performed on samples of near surface soils. The results of this test indicates that the near surface soils possess a very low expansion index (EI = 3).

Kleinfelder calculated potential liquefaction induced settlements of 4 to 8± inches, with differential settlements as much as 3 to 5± inches over a distance of 50 feet.

Kleinfelder recommends ground improvement to allow the use of a shallow foundation system. The suggested methods of ground improvement are Deep Soil Mixing and Stone Columns. The

criteria for ground improvement are based on limiting static and seismic settlement (total and differential) to within 1 inch total and ½ inch differential over 40 feet in the building pad area.

Site preparation recommendations for the building pad subsequent to the ground improvement procedures consisted of remedial grading to depths on the order of 3 feet.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of six (6) borings advanced to depths of 5± to 50± feet below presently existing site grades. All of the borings were logged during drilling by a member of our staff.

The borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. In-situ samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface and beneath the pavements at all of the boring locations, except for Boring No. B-2, extending to depths of 2½ to 4½± feet below the existing site grades. The fill soils generally consist of loose to medium dense sands and silts. The fill soils possess variable strengths and a disturbed appearance, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered at the ground surface and beneath the fill soils at all of the boring locations, extending to at least the maximum depth explored of 50± feet below existing site grades. The alluvium generally consists of very loose to loose silty fine sands, fine sandy silts, and soft to medium stiff clayey silts and silty clays, extending to depths of 42 to 44½± feet below existing site grades. At greater depths, the alluvium consists of medium dense fine sands

with varying silt content, extending to the maximum depth explored of 50± feet below the existing site grades.

Groundwater

Free water was encountered during drilling at depths of 9 to 11± feet. Based on the water level measurements and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at depths of 9 to 11± feet below existing site grades at the time of the subsurface investigation.

As part of our research, we reviewed historic groundwater levels obtained from the CA DMG Open-File Report 98-19 for the Long Beach Quadrangle. Plate 1.2 of OFR 98-19 is a map which displays the historically highest ground water levels using contour lines. No contour lines are mapped within the project site. Interpolation between water level contour lines in the areas surrounding the site indicates a historic ground water level of 8± feet below ground surface. More recent water level data was obtained from the California State Water Resources Control Board Geotracker website, http://geotracker.waterboards.ca.gov/. Several monitoring wells are located in the vicinity of the subject site. Water level readings indicate high groundwater levels ranging from 8 to 13± feet below the ground surface. Therefore, a historic groundwater depth of 8± feet is considered to be conservative with respect to the recent site conditions.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. Field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

Dry Density and Moisture Content

The dry density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-8 in Appendix C of this report.

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these laboratory tests are shown on the attached boring logs.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples of various soil strata encountered at the site. This test is used to determine the Liquid Limit and Plastic Limit of the

soil. The Plasticity Index is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high plasticity, and a high expansion potential. Nonsensitive soils with a PI greater than 18 are not considered to be susceptible to liquefaction when the moisture content of the soil is less than 80 percent of the liquid limit. The results of the Atterberg Limits testing are presented on the boring logs.

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	**Soluble Sulfates (%)**	**ACI Classification**
B-2 @ 0 to 5 feet	0.153	Moderate

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample was tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of the testing are plotted on Plate C-9 in Appendix C of this report.

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	**Expansion Index**	**Expansive Potential**
B-1 @ 0 to 5 feet	0	Very Low
B-5 @ 0 to 5 feet	2	Very Low

R-value

The R-(resistance) value was determined for a representative soil sample, taken from the proposed parking and drive lane areas, in accordance with CA Test Method 301. This test provides a measure of the pavement support characteristics of the soils, and is used in the pavement thickness design procedure. The results of the R-value testing are as follows:

Sample Identification	**R-Value**
B-4 @ 0 to 5 feet	53

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations. The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

The 2013 California Building Code (CBC) was adopted by all municipalities within Southern California on January 1, 2014. The CBC provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2013 CBC Seismic Design Parameters have been generated using U.S. Seismic Design Maps, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site, calculates seismic design parameters in accordance with the 2013 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. The table below is a compilation of the data provided by the USGS application. A copy of the output generated from this program is included as Plate E-1 in Appendix E of this report. A copy of the Design Response Spectrum, as generated by the USGS application is also

included in Appendix E. Based on this output, the following parameters may be utilized for the subject site:

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.644
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.620
Site Class	---	F*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.644
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.930
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.096
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.620

*The 2013 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site coefficients are to be determined in accordance with Section 11.4.7 of ASCE 7-10. However, Section 20.3.1 of ASCE 7-10 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of both of the structures is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structures have fundamental periods greater than 0.5 seconds, a site specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.

Ground Motion Parameters

For the liquefaction evaluation, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2013 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-10. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-10. The web-based software application U.S. Seismic Design Maps (described in the previous section) was used to determine PGA_M, which is 0.642g. A portion of the program output is included as Plate E-2 of this report. An associated earthquake magnitude was obtained from the 2008 USGS Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 7.21, based on the peak ground acceleration and NEHRP soil classification D.

Liquefaction

Research of the Long Beach Quadrangle, California 7.5 Minute Seismic Hazard Zone Map, published by the California Geological Survey, indicates that the site is located in a designated liquefaction hazard zone. Therefore, the scope of this investigation included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which

the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85% of the liquid limit, are considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

As part of the liquefaction evaluation, Boring Nos. B-1 and B-5 were extended to depths of 50± feet. The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.642g for a magnitude 7.21 seismic event.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the corrected N-value to determine the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included in Appendix F.

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at the site. Several potentially liquefiable strata are located at various depths between 10 and 45± feet at the boring locations. Soils which are located above the historic groundwater table, or possess factors of safety of at least 1.3 are considered non-liquefiable. Several silty clay strata and clayey are also considered to be non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the criteria of Bray and Sancio (2006). Settlement analyses were conducted for each of the potentially liquefiable strata.

Based on the settlement analysis (also tabulated on the spreadsheets in Appendix F) total dynamic (liquefaction induced) settlements of 7.38, and 6.92± inches are expected at Boring Nos. B-1, and B-5, respectively, during the design level earthquake. Based on these total settlements, differential settlements of up to 3 to 4± inches should be expected to occur during a liquefaction inducing seismic event. The estimated differential settlement could be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion on the order o 0.003 inches per inch.

Based on the presence of liquefiable soils at the boring locations, and the relatively large associated dynamic settlements, ground improvement is recommended in order to reduce these settlements to allow the use of a conventional shallow foundation system. Designing the proposed building to remain completely undamaged during a major seismic event is not considered to be economically feasible. The ground improvement program should be designed to reduce the total settlement (static and seismic) to less than 4 inches.

Any utility connections to the structures should be designed to withstand the estimated dynamic settlements. It should also be noted that minor to moderate repairs, including releveling, restoration of utility connections, repair of damaged drywall and stucco, etc., would likely be required after occurrence of a major earthquake.

The use of shallow foundation systems in conjunction with the recommend ground improvement, as described in this report, is typical for buildings of these types, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the buildings at this site will also be typical of similar buildings in the vicinity of this project. Other geotechnical and structural options are available, including the use of deep foundations such as driven piles, and drilled piers, but are considered to be less economically feasible.

6.2 Geotechnical Design Considerations

General

The results of the liquefaction evaluation indicate that total dynamic settlements on the order of 7± inches could occur at the site during the design seismic event at the boring locations. The liquefaction induced differential settlements are expected to be on the order of 3 to 4± inches. While remedial grading could control static settlements to tolerable levels, it would have no effect on the potential liquefaction induced settlements. Several of the borings encountered a surficial layer of fill soils. All of the borings encountered loose/soft native alluvial soils with a moderate to severe potential for consolidation settlement. Low strength, compressible soils are present at the boring locations within the foundation influence zone of the proposed structure.

The most feasible method of mitigating potential static and dynamic settlements at this site is considered to be remedial grading of the near surface soils in conjunction with ground improvement of the deeper liquefiable soils. Based on the groundwater levels and the soil conditions, techniques such as deep soil mixing or grout injection are considered the most applicable for ground improvement. A specialty contractor should be contacted for specifics of

design build ground improvement methods. Ground improvement should be designed to reduce total settlements (static and seismic) to less than 4 inches.

Based on the relatively shallow depth to groundwater, remedial grading will be limited in depth and the soils exposed at the base of the overexcavation will likely require stabilization due to high moisture contents. Most of the overexcavated soils will also require drying prior to reuse in compacted fills, due to high moisture contents.

Settlement

The recommended ground improvement in conjunction with limited remedial grading will control total settlements to acceptable levels and allow the use of a conventional shallow foundation system. These mitigative measures will limit the post-construction settlements of the proposed structures to within tolerable limits.

Expansion

Laboratory testing performed on representative samples of the near surface soils indicates that these soils possess very low expansion potentials (EI = 0 & 2). The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils contain levels of soluble sulfates that are classified as having a moderate potential to attack concrete, in accordance with the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, it is recommended that a sulfate-resistant concrete mix design be utilized for the foundations and floor slabs at this site. In accordance with the ACI requirements, it is recommended that this concrete incorporate the following characteristics:

- Cement Type: II (Two)
- Minimum Compressive Strength (f'_c) = 4,000 lbs/in^2
- Maximum Water/Cement Ratio: 0.50

It is recommended that additional sulfate testing be performed at the completion of rough grading to verify the concentrations which are present in the actual building pad subgrade soils.

Shrinkage/Subsidence

Removal and recompaction of the near surface soils is estimated to result in an average shrinkage of 10 to 15 percent. Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.15± feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependant on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

Grading and foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

Several existing structure at the site will require demolition. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. The existing portion of Technology Drive which traverses the site will also be abandoned and demolished. Concrete and asphalt debris may be crushed to a maximum 2-inch particle size, mixed with the on-site soils, and reused as compacted structural fill. It may also be feasible to crush these materials for use as crushed miscellaneous base (CMB).

Initial site preparation should also include removal of any surficial vegetation. Based on conditions encountered at the time of the subsurface exploration, the site possesses moderate grass and weed growth, and several trees. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Ground Improvement

Ground improvement consisting of deep soil mixing or grout injection is recommended to mitigate the potential dynamic settlements of the liquefiable soils. Based on the groundwater levels and the soil conditions, techniques such as deep soil mixing or grout injection are considered the most applicable for ground improvement. A specialty contractor should be contacted for specifics of design-build ground improvement methods. Ground improvement should be designed to reduce total settlements (static and seismic) to less than 4 inches. The actual design of the ground improvement method should be performed by the design-build contractor who is specialized and experienced with these methods. Ground improvement

methods are performed by specialty contractors on a design–build basis where the contractors are ultimately responsible for their designs.

Treatment of Existing Soils: Building Pad

Subsequent to ground improvement procedures, remedial grading should be performed within the proposed building pad area in order to remove the near surface fill soils and existing potentially compressible/collapsible native alluvium. It is recommended that the overexcavation extend to a depth of at least 5 feet below existing grade, and to a depth of at least 5 feet below proposed grade, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation should also extend to a depth sufficient to remove all undocumented fill soils.

The overexcavation areas should extend at least 5 feet beyond the building perimeter, and to an extent equal to the depth of fill below the new foundations. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the overexcavation areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, overly moist, or low density native soils are encountered at the base of the overexcavation.

Based on conditions encountered at the exploratory boring locations, moist to very moist soils will be encountered at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils is expected to be necessary. Scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization, usually consisting of coarse crushed stone or geotextile, could be necessary. In this event, the geotechnical engineer should be contacted for supplementary recommendations. Typically, an unstable subgrade can be stabilized using a suitable geotextile fabric, such as Mirafi 580I, HP 570 or HP 270, and/or a 12 to 18-inch thick layer of coarse (2 to 4 inch particle size) crushed stone. Crushed asphalt and concrete debris resultant from demolition could also be used as a subgrade stabilization material. Other options, including lime treatment, are also available.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, and moisture conditioned to 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill, provided that they are dried to within 2 to 4 percent above the optimum moisture content. The use of an imported select fill material may be desirable if the construction schedule does not allow for drying of the on-site soils.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill. Any undocumented fill soils should also be removed from the retaining wall areas. In both cases, the overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Parking Areas

Based on economic considerations, overexcavation of the existing fill soils and/or the surficial alluvial soils in the new parking areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading.

Subgrade preparation in the new parking areas should initially consist of removal of all soils disturbed during stripping operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not completely mitigate the extent of existing collapsible and compressible alluvium in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 3 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork Areas

Subgrade preparation in the new flatwork areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned (or air dried) to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer. However, the some of the near surface soils possess moisture contents well over the optimum moisture content. Therefore, some drying of the on-site soils will be necessary prior to compaction as structural fill.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2013 CBC and the grading code of the city of Long Beach.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of low expansive (EI < 50), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by city of Long Beach. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

6.4 Construction Considerations

Moisture Sensitive Subgrade Soils

The near surface fill soils generally consist of silty sands and sandy silts and some clayey silts. Based on their silt and clay content some of these soils will become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations. **It should be noted that the silts and clays present at depths greater than 5± feet possess relatively high moisture contents. Subgrade stabilization is expected to be necessary where excavations extend into these soils.**

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad areas as well as the need for subgrade stabilization.

Excavation Considerations

The near surface soils generally consist of sands and silty sands. These materials are expected to be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes should be no steeper than 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Groundwater

The static groundwater table at this site is considered to be present at depths of 9 to 11± feet. In general, groundwater is not expected to impact the grading or foundation construction activities. However, if any excavations extend to depths of 8 or more feet below the existing site grades, some dewatering may be required.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by newly placed structural fill soils extending to depths of at least 3 feet below foundation bearing grade. Based on this subsurface profile, the proposed structure may be supported on shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom) due to the presence of liquefiable soils.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill compacted at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than

0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report. However, the likelihood of these two settlements combining is considered remote. The static settlements are expected to occur in a relatively short period of time after the building loads being applied to the foundations, during and immediately subsequent to construction. It should be noted that the projected potential dynamic settlement is related to a major seismic event and a conservative historic high groundwater level.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill soils. The maximum allowable passive pressure is 2,500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill (or densified existing soils), extending to a depth of at least 5 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 5 inches.

- Modulus of Subgrade Reaction: 100 lbs/in^3.

- Minimum slab reinforcement: No. 3 rebars at 18 inches on center in both directions due to the presence of liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire slab area where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications.

Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

6.7 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for sidewalks, patios, and other concrete flatwork, should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. Based on geotechnical considerations, exterior slabs on grade may be designed as follows:

- Minimum slab thickness: 4½ inches.

- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.

- Moisture condition the slab subgrade soils to at least 2 to 4 percent of optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

6.8 Retaining Wall Design and Construction

New retaining walls are expected to be necessary in the truck court areas. Additionally, although not indicated on the site plan, the proposed development may require some small retaining walls (less than 5± feet in height) to facilitate the new site grades.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of silty sands and sandy silts. Based on their classifications, the silty sand, and fine sandy silt materials are expected to possess a friction angle of at least 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		**Soil Type** On-Site Silty Sands and Sandy Silts
Internal Friction Angle (ϕ)		30°
Unit Weight		120 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	40 lbs/ft^3
	Active Condition (2h:1v backfill)	65 lbs/ft^3
	At-Rest Condition (level backfill)	60 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the 2013 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls be used. If the drainage composite material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The drainage composite should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should

include a one cubic foot gravel pocket surrounded by a suitable geotextile at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.9 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be supported on the existing fill and/or native soils that have been scarified, moisture conditioned, and recompacted. These materials generally consist of silty sands, and sandy. Based on R-value testing, these soils exhibit good pavement support characteristics, with an R-values of 53. The subsequent pavement designs are based upon a conservative R-value of 50. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It may be desirable to perform R-value testing after the completion of rough grading to verify the R-value of the as-graded parking subgrade.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20 year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 50)					
Materials	**Thickness (inches)**				
	Automobile Parking (TI = 4.0)	Automobile Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	3	4	5	5
Compacted Subgrade	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS			
Materials	**Thickness (inches)**		
	Automobile and Light Truck Traffic (TI = 5.0 & 6.0)	Truck Traffic	
		(TI = 7.0)	(TI = 8.0)
PCC	5	6	7
Compacted Subgrade (95% minimum compaction)	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

7.0 GENERAL COMMENTS

This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

8.0 REFERENCES

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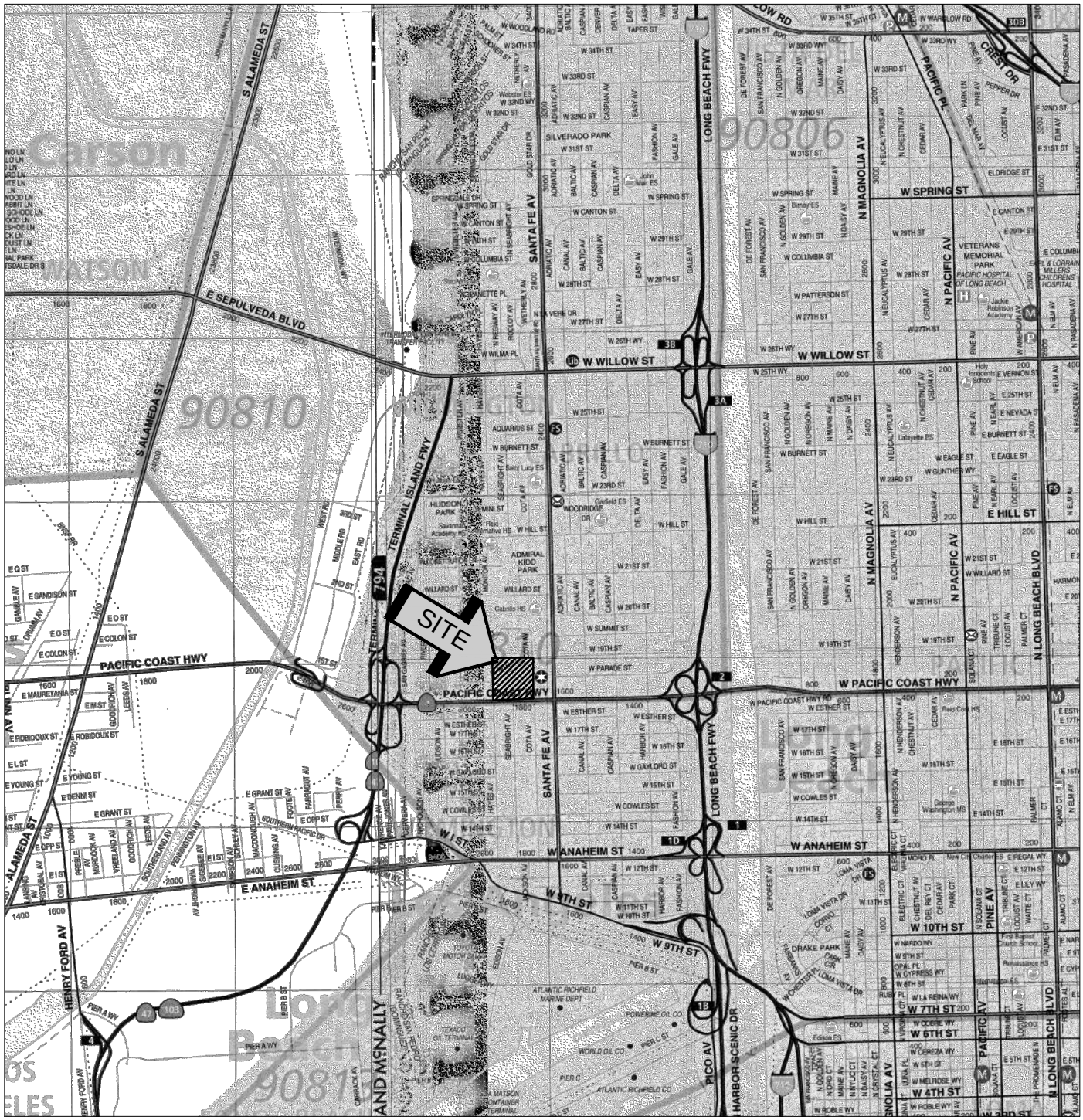
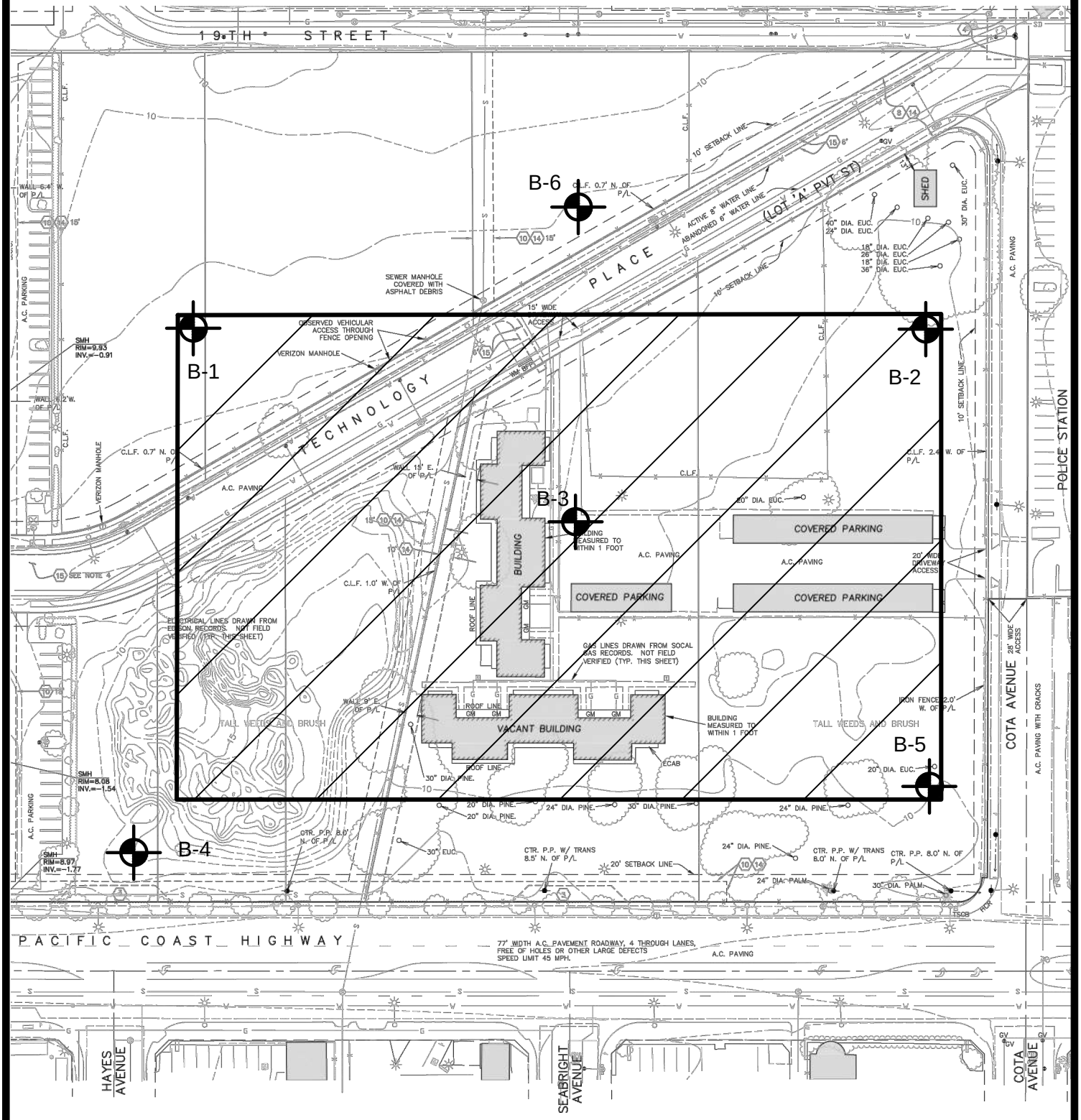
APPENDIX A



SOURCE: LOS ANGELES COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP
PROLOGIS COMMERCIAL/INDUSTRIAL BUILDING
LONG BEACH, CALIFORNIA
SCALE: 1" = 2400'
DRAWN: JLH CHKD: JAS
SCG PROJECT 16G144-1
PLATE 1

SOUTHERN CALIFORNIA GEOTECHNICAL



NOTE: BASE MAP PREPARED BY CORNERSTONE ENGINEERING, INC.

GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

AREA OF PROPOSED BUILDING
(APPROXIMATELY 195,008 S.F.)

NORTH
SoCalGeo

BORING LOCATION PLAN

PROPOSED COMMERCIAL/INDUSTRIAL BUILDING

LONG BEACH, CALIFORNIA

SCALE: 1" = 100'	
DRAWN: PM CHKD: JAS	
SCG PROJECT 16G144-1	
PLATE 2	

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.

MOISTURE CONTENT: Moisture content of a soil sample, expressed as a percentage of the dry weight.

LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

PASSING #200 SIEVE: The percentage of the sample finer than the #200 standard sieve.

UNCONFINED SHEAR: The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 16G144
PROJECT: Proposed C/I Bldg
LOCATION: Long Beach, California

DRILLING DATE: 3/31/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 10 feet
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	10			FILL: Light Gray Brown fine Sand, trace Silt, loose to medium dense-damp		5					EI = 0 @ 0 to 5'
5	X	4			ALLUVIUM: Gray fine Sandy Silt, little Clay nodules, trace roots, very loose to loose-very moist to wet		32					
	X	2			Gray Brown Clayey Silt, little fine Sand, very soft-wet		37	39	28	87		
10	X	3	0.5		@ 10 feet, Water encountered during drilling		43					
					Gray Silty fine Sand to fine Sandy Silt, trace Shell fragments, very loose-wet		37			46		
	X	3			Dark Gray Silty fine Sand, occasional Shell fragments, abundant Iron oxide staining, 2" Silt lense, very loose-wet		37			61		
15	X	3					35					
	X	5	0.5		Gray Clayey Silt, trace fine Sand, 1" lense Sandy Silt, soft to medium stiff-wet		50					
20	X	4	0.5				48	61	20	95		
	X	4	0.5				52					
					Gray fine Sandy Silt, medium dense-wet							
25	X	11					34					
	X	5					23	27	23	53		
					Dark Gray Silty fine Sand to fine Sandy Silt, little Clay nodules, loose-very moist to wet							
30												
					Gray Silty fine Sand, little Clay nodules, loose-wet							
	X	16					30					

TBL 16G144.GPJ SOCALGEO.GDT 5/12/16

TEST BORING LOG

PLATE B-1a

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 16G144
PROJECT: Proposed C/I Bldg
LOCATION: Long Beach, California

DRILLING DATE: 3/31/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 10 feet
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Gray Silty fine Sand, little Clay nodules, loose-wet							
		10			Gray Silt, little fine Sand, loose to medium dense-wet		35			92		
40												
					Gray Silty fine Sand to fine Sandy Silt, medium dense-wet							
		19					29			54		
45												
					Gray fine Sand, trace Clay, medium dense-wet							
		23					22			7		
50												
					Boring Terminated at 50'							

TBL 16G144.GPJ SOCALGEO.GDT 5/12/16

TEST BORING LOG

PLATE B-1b

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 16G144
PROJECT: Proposed C/I Bldg
LOCATION: Long Beach, California

DRILLING DATE: 3/31/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 11 feet
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	14			ALLUVIUM: Light Brown fine Sandy Silt, some fine root fibers, some Iron oxide staining, loose-damp	100	9					
	X	8			@ 3 feet, very moist to wet	72	26					
5	X	8			Gray to Gray Brown fine Sand, little Silt, trace Iron oxide staining, very loose to loose-moist	88	8					
	X	5				78	10					
10	X	6	2.0		Dark Gray Brown Clayey Silt, little fine Sand, occasional Organic fragments, soft to medium stiff-very moist	86	33					
					@ 11 feet, Water encountered during drilling							
					Gray Brown fine Sand, little Silt, trace fine root fibers, very loose to loose-wet							
15	X	5				91	28					
20	X	9										No Sample Recovered

Boring Terminated at 20'

TBL 16G144.GPJ SOCALGEO.GDT 5/12/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 16G144
PROJECT: Proposed C/I Bldg
LOCATION: Long Beach, California

DRILLING DATE: 3/31/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 9 feet
CAVE DEPTH: 14 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					2± inches Asphaltic concrete, 5± inches Aggregate base							
	X	10			FILL: Brown fine Sandy Silt, trace fine root fibers, trace Iron oxide staining, loose-very moist	89	25					
	X	6			ALLUVIUM: Gray Brown Silty fine Sand, trace Iron oxide staining, very loose to loose-very moist	94	9					
5	X	5			Brown fine Sandy Silt, trace Iron oxide staining, very loose-very moist to wet	79	13					
	X	6			Brown fine Sand, little Silt, trace Iron oxide staining, very loose to loose-damp	95	20					
10	X	10			Dark Gray Brown Silty fine Sand to fine Sandy Silt, loose-very moist to wet @ 9 feet, Water encountered during drilling	96	26					
					Brown Silty fine Sand, very loose to loose-wet							
15	X	5 7					38					No Sample Recovered
					Boring Terminated at 16½'							

TBL 16G144.GPJ SOCALGEO.GDT 5/12/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 16G144
PROJECT: Proposed C/I Bldg
LOCATION: Long Beach, California

DRILLING DATE: 3/31/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 9 feet
CAVE DEPTH: 11 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	23			FILL: Brown fine Sandy Silt, trace fine Gravel, mottled, medium dense-damp to moist	118	9					
	X	9			ALLUVIUM: Gray Silt, little fine Sand, loose-damp	88	9					
5	X	13			Gray fine Sand, little Silt, loose-damp	106	7					
	X	3			Dark Brown Silty fine Sand to fine Sandy Silt, very loose-very moist to wet	89	20					
10	X	4	0.5		Dark Gray Brown Silt, trace Clay, some Iron oxide staining, soft-wet @ 9 feet, Water encountered during drilling	75	44					
					Dark Gray Brown fine Sandy Silt, very loose-wet							
15	X	5			Dark Gray fine Sand, little Silt, very loose-wet	87	36					
					Dark Gray Clayey Silt, some Shell fragments, soft-wet							
20	X	5				77	42					

Boring Terminated at 20'

TBL 16G144.GPJ SOCALGEO.GDT 5/12/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 16G144
PROJECT: Proposed C/I Bldg
LOCATION: Long Beach, California

DRILLING DATE: 3/31/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 10 feet
CAVE DEPTH: 14 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	10			FILL: Light Gray Brown Silty fine Sand, losoe to medium dense-dry to damp		3					EI = 2 @ 0 to 5'
	X	5			ALLUVIUM: Light Gray Brown Silty fine Sand with 1" Silt lense, loose-damp		7					
5	X	4			Dark Gray Brown Silty fine Sand, very loose-very moist		25			38		
					Dark Gray Brown Silt, trace fine Sand, very loose-very moist		31			94		
	X	3			Brown Clayey Silt, little fine Sand, soft-very moist to wet		28			85		
10					@ 10 feet, Water encountered during drilling							
	X	2			Brown Silty fine Sand, very loose-wet		31			40		
15												
	X	14			Brown fine Sand, trace to little Silt, medium dense-wet		28			8		
20												
	X	2	0.5		Gray Silty Clay, trace fine Sand, very soft-wet		51					
25												
	X	2					56	62	36	96		
30												
	X	2					50					
					@ 33½ feet, 1" lense of fine Sandy Clay, occasional Shell fragments							

TBL 16G144.GPJ SOCALGEO.GDT 5/12/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 16G144
PROJECT: Proposed C/I Bldg
LOCATION: Long Beach, California

DRILLING DATE: 3/31/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

WATER DEPTH: 10 feet
CAVE DEPTH: 14 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	8			Gray fine Sand, little Silt, 1" Gray Clay lense, loose to medium dense-wet		30			11		
	X	11					29			17		
					Gray fine Sandy Clay, loose-wet		39			88		
40	X	9			Gray Clayey fine Sand, stiff-very moist		20			42		
	X	11	0.5		Gray Silty Clay, trace fine Sand, 1" Clayey fine Sand lense, stiff-wet		24	47	25	37		
45	X	28			Gray fine Sand, little Silt, medium dense-wet		23					
	X	27					27			9		
50	X	28					25					
					Boring Terminated at 51½'							

TBL 16G144.GPJ SOCALGEO.GDT 5/12/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 16G144
PROJECT: Proposed C/I Bldg
LOCATION: Long Beach, California

DRILLING DATE: 3/31/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jason Hiskey

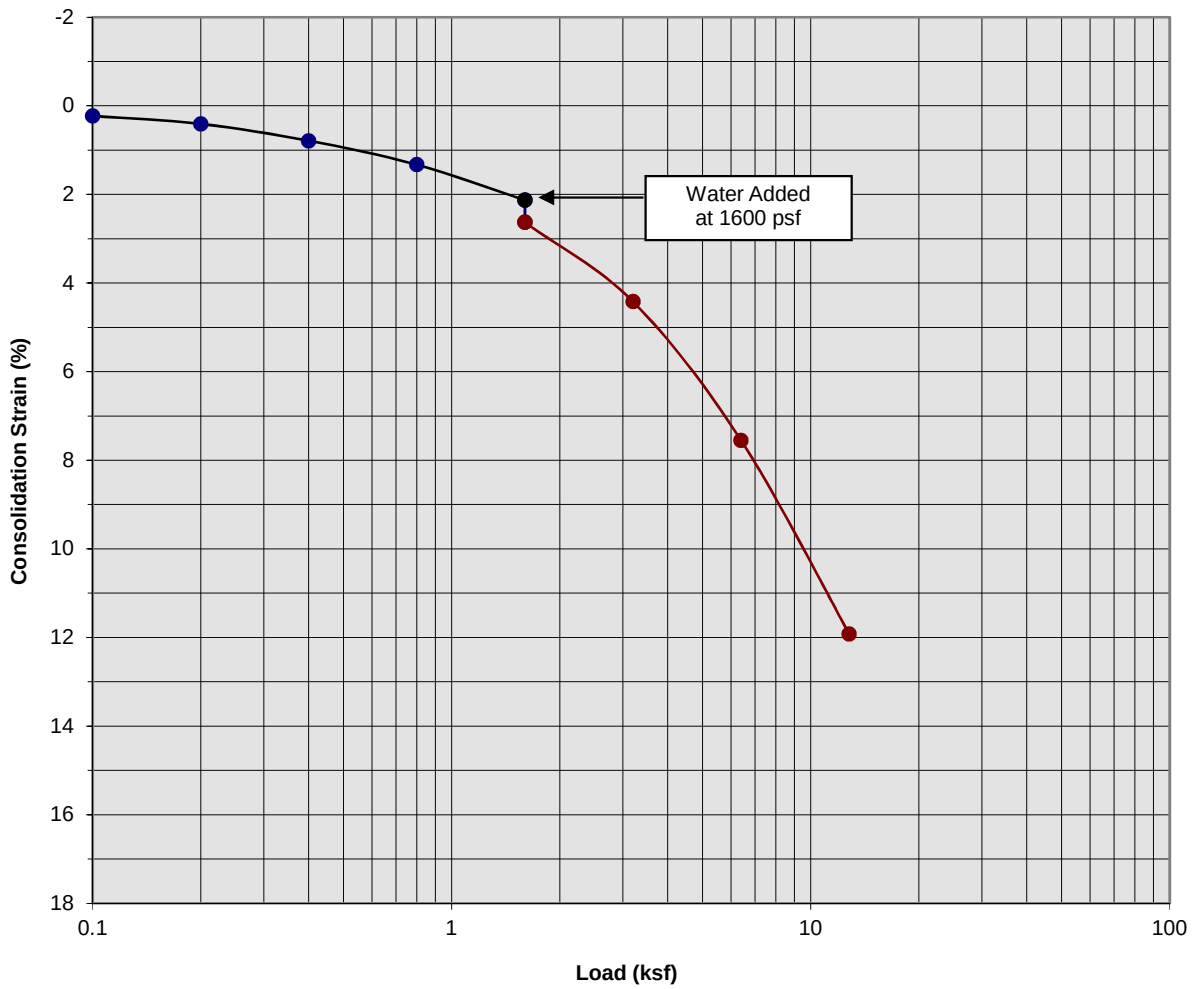
WATER DEPTH: Dry
CAVE DEPTH: 3.5 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	13			FILL: LIght Gray Brown Silt, trace fine Sand, trace calcareous nodules, medium dense-moist		21					
	X	6			ALLUVIUM: Light Gray Brown fine Sandy Silt, little Clay nodules, some Iron oxide staining, loose to medium dense-damp		6					
5					Boring Terminated at 5'							

TBL 16G144.GPJ SOCALGEO.GDT 5/12/16

APPENDIX C

Consolidation/Collapse Test Results



Classification: Light Brown fine Sandy Silt

Boring Number:	B-2	Initial Moisture Content (%)	26
Sample Number:	---	Final Moisture Content (%)	34
Depth (ft)	3 to 4	Initial Dry Density (pcf)	72.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	86.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.50

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C- 1

SOUTHERN CALIFORNIA GEOTECHNICAL
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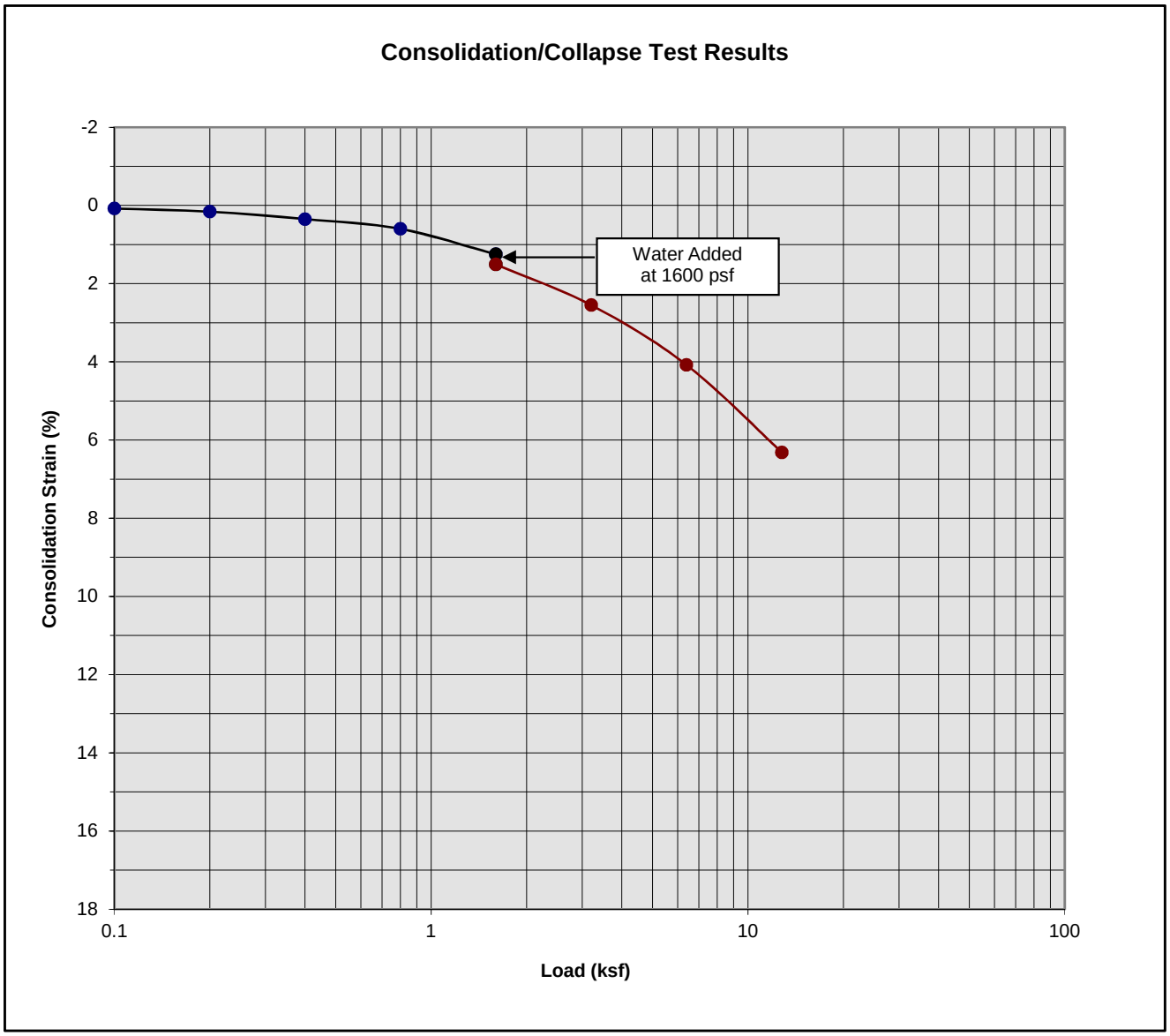


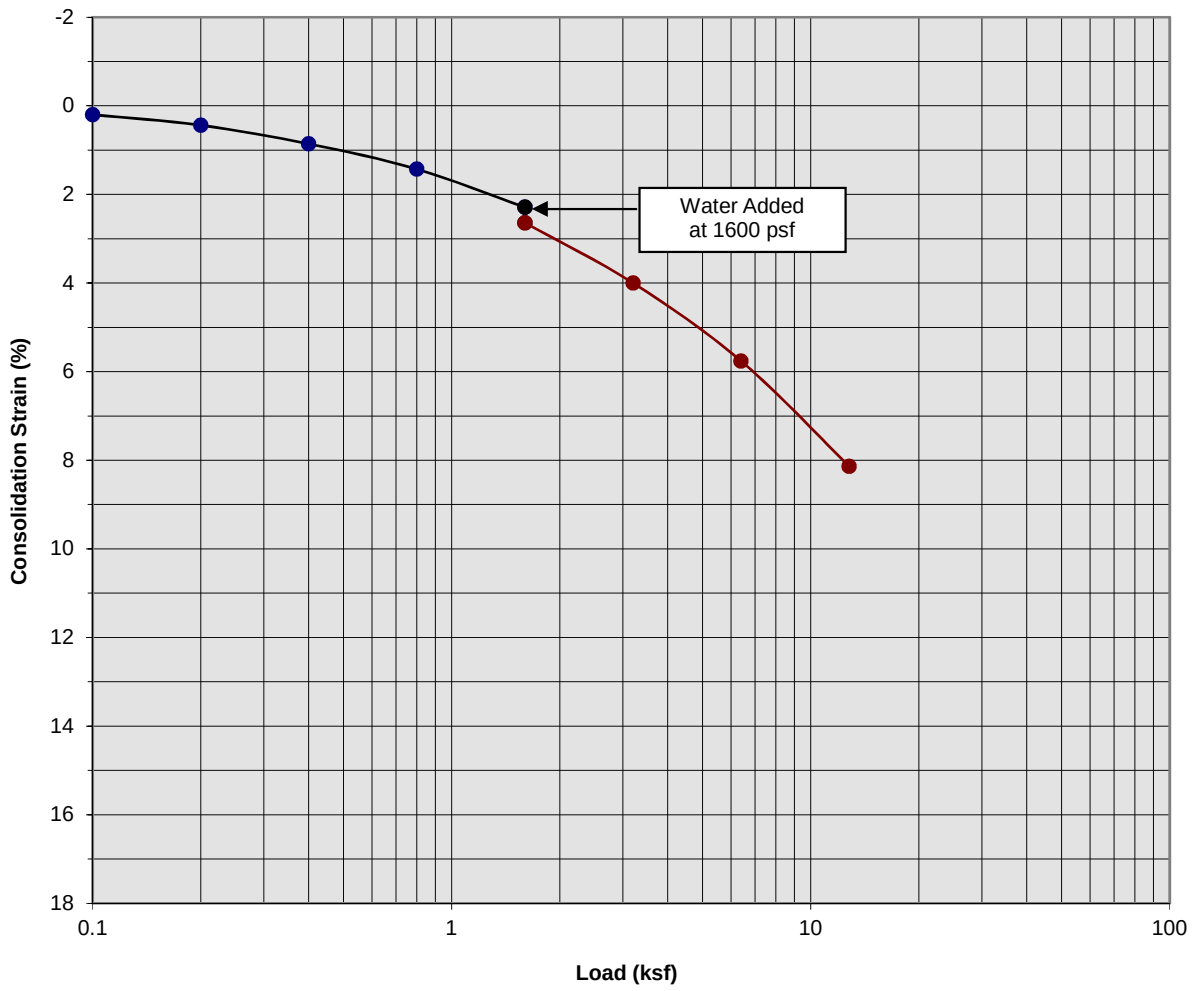
Classification: Gray to Gray Brown fine Sand, little Silt

Boring Number:	B-2	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	37
Depth (ft)	5 to 6	Initial Dry Density (pcf)	88.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	93.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.26

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C- 2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Gray to Gray Brown fine Sand, little Silt

Boring Number:	B-2	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	38
Depth (ft)	7 to 8	Initial Dry Density (pcf)	78.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	84.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.35

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C- 3

SOUTHERN CALIFORNIA GEOTECHNICAL
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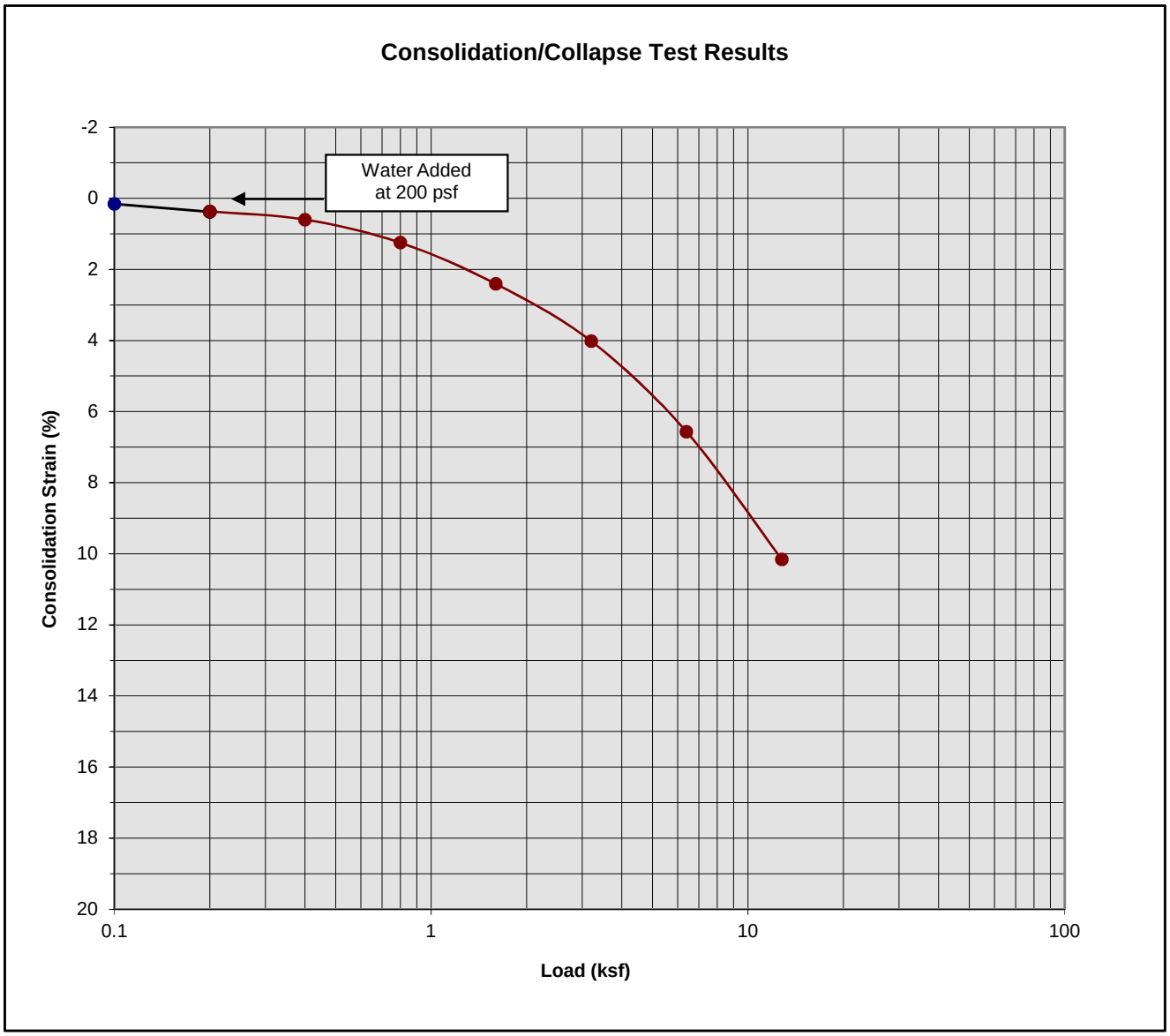
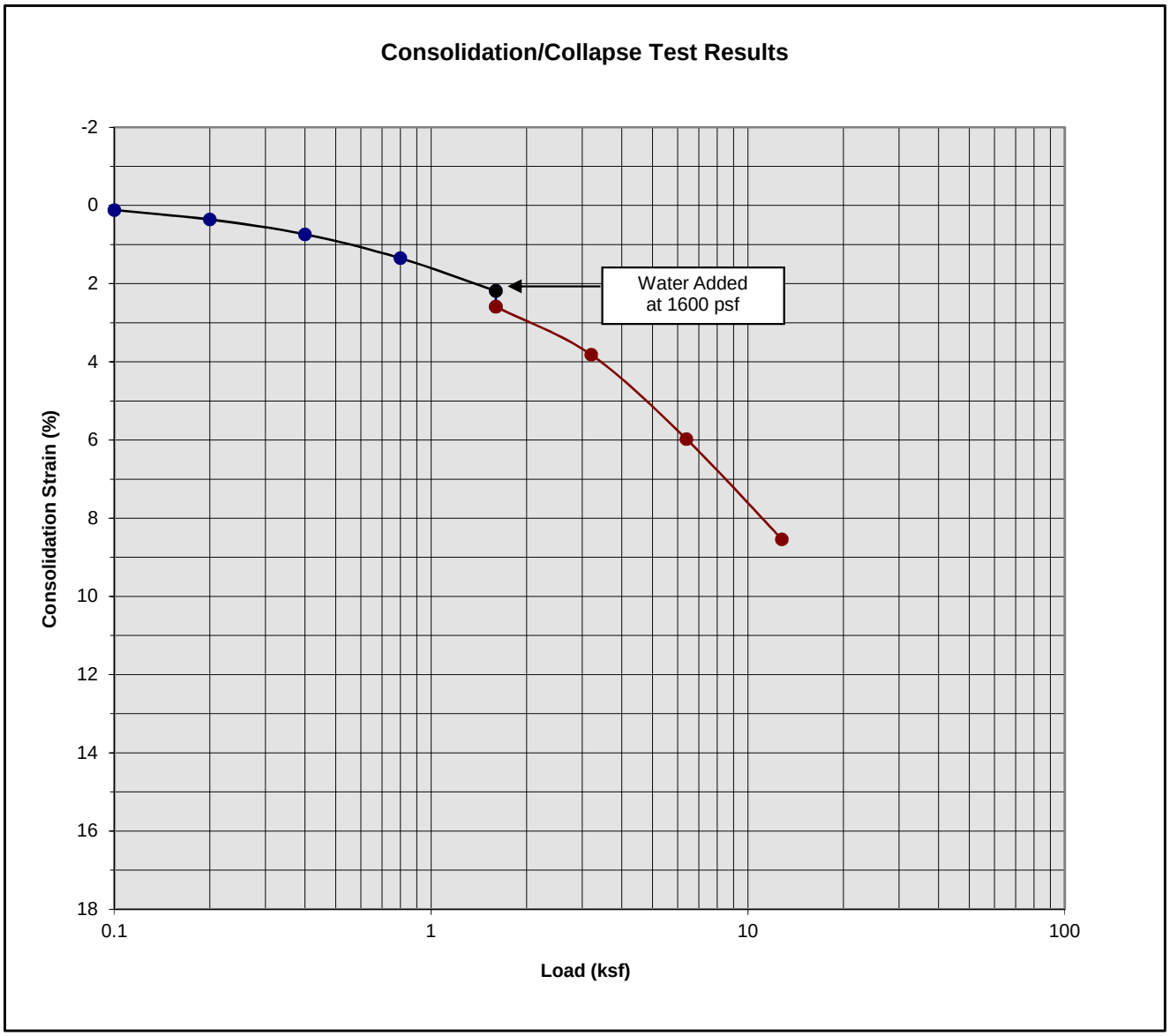


Classification: Dark Gray Brown Clayey Silt, little fine Sand

Boring Number:	B-2	Initial Moisture Content (%)	33
Sample Number:	---	Final Moisture Content (%)	28
Depth (ft)	9 to 10	Initial Dry Density (pcf)	86.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	95.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.01

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Gray Silt, little fine Sand

Boring Number:	B-4	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	33
Depth (ft)	3 to 4	Initial Dry Density (pcf)	88.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.40

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C- 5

SOUTHERN CALIFORNIA GEOTECHNICAL
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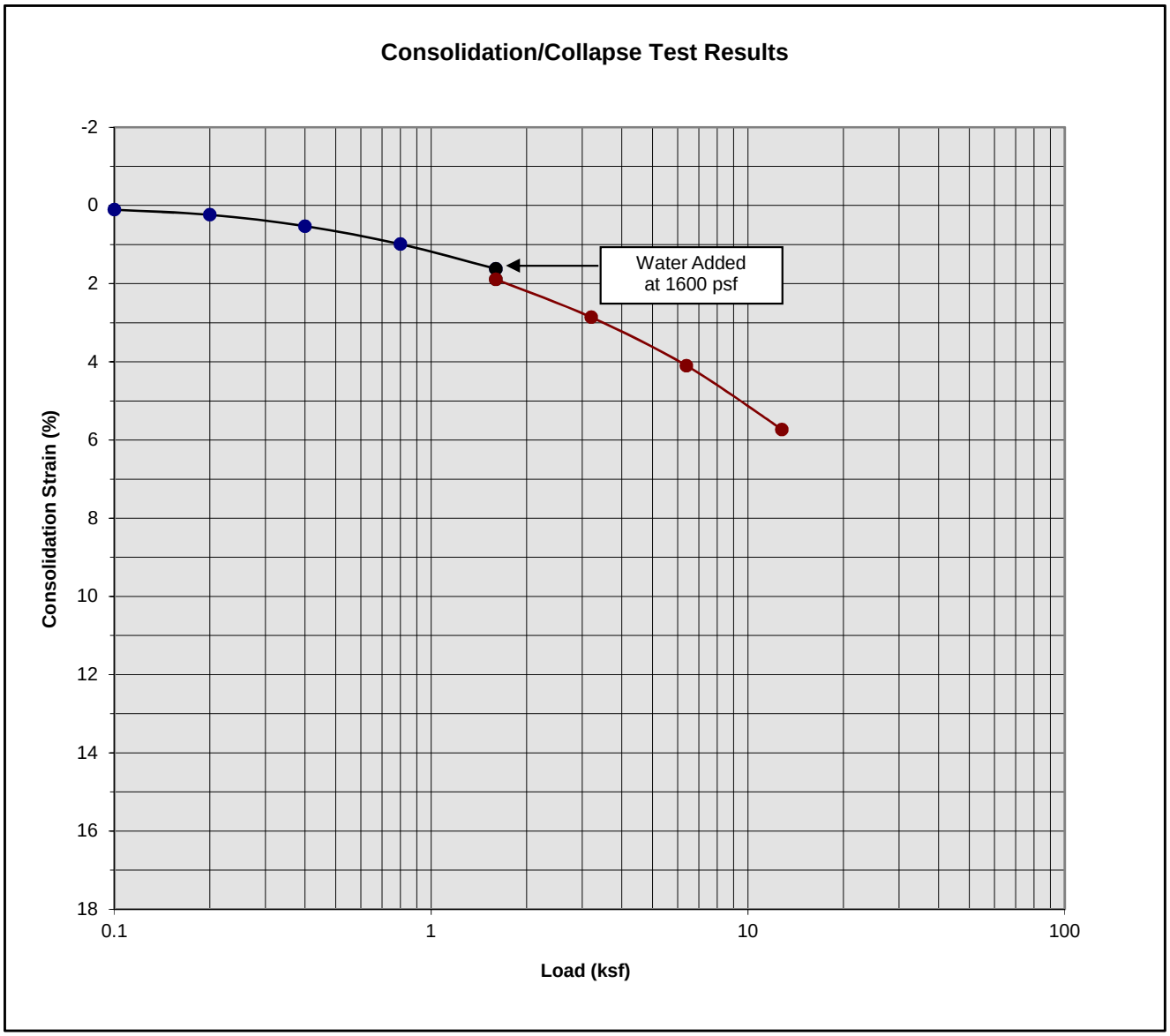


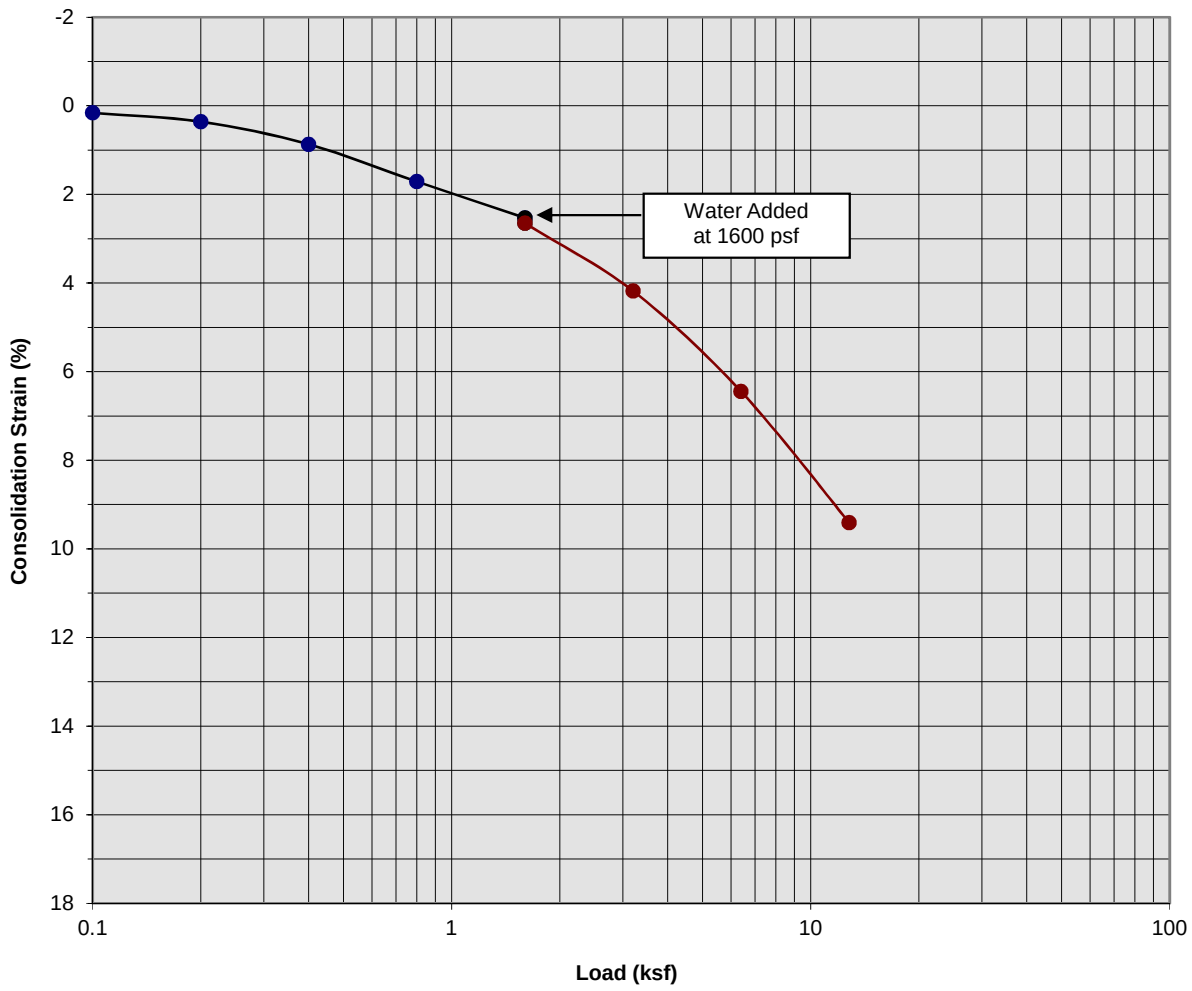
Classification: Gray fine Sand, little Silt

Boring Number:	B-4	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	30
Depth (ft)	5 to 6	Initial Dry Density (pcf)	106.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.27

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C- 6

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



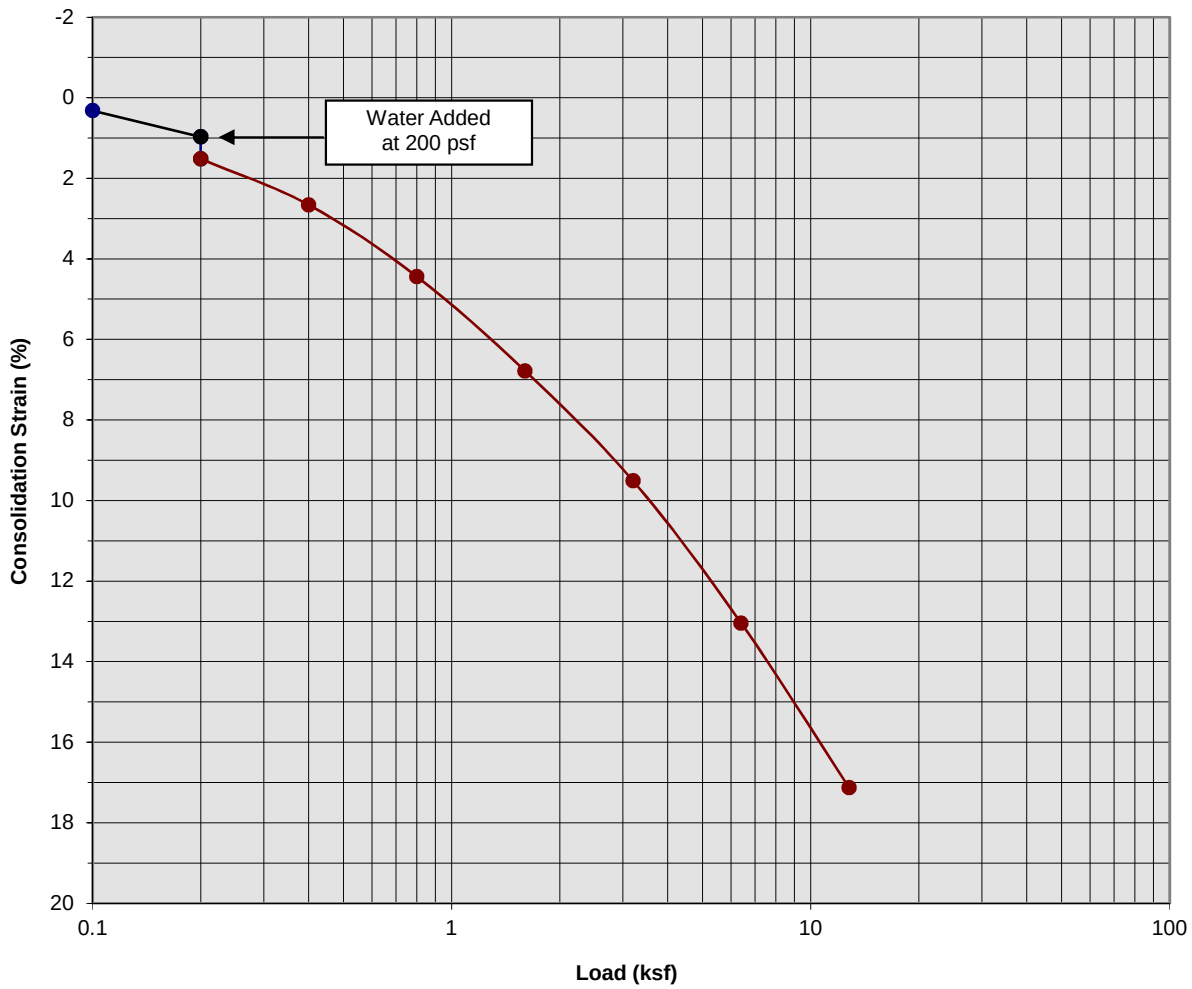
Classification: Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-4	Initial Moisture Content (%)	21
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	7 to 8	Initial Dry Density (pcf)	90.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	100.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.12

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C- 7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

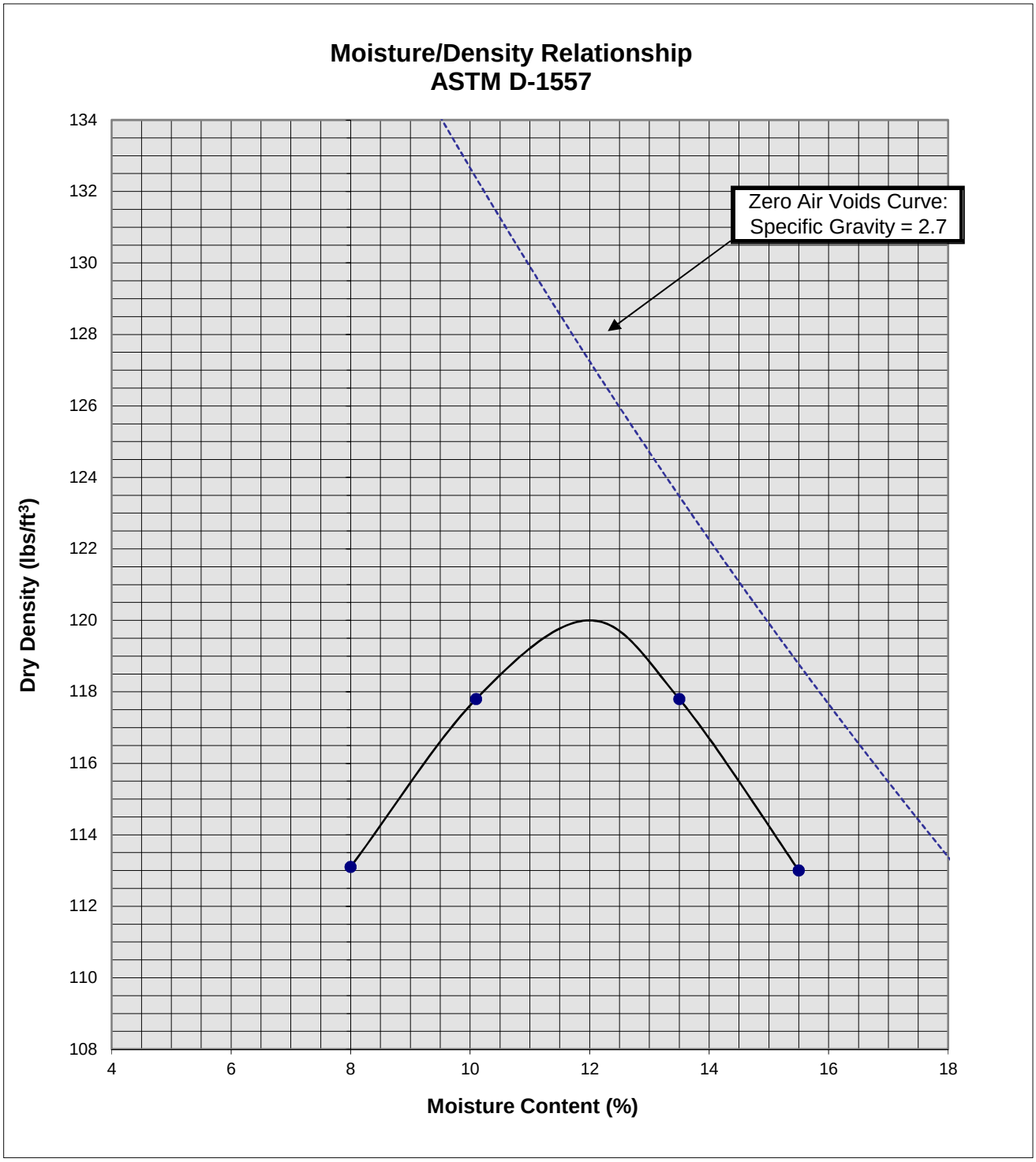


Classification: Dark Gray Brown Silt, trace Clay

Boring Number:	B-4	Initial Moisture Content (%)	44
Sample Number:	---	Final Moisture Content (%)	40
Depth (ft)	9 to 10	Initial Dry Density (pcf)	74.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	90.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.55

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C- 8

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-2 @ 0 to 5'
Optimum Moisture (%)	12
Maximum Dry Density (pcf)	120
Soil Classification	Gray Brown fine Sandy Silt, trace fine Gravel, trace medium Sand

Proposed C/I Bldg
Long Beach, California
Project No. 16G144
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated,” and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

    - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

    - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or

concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

- Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

- To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

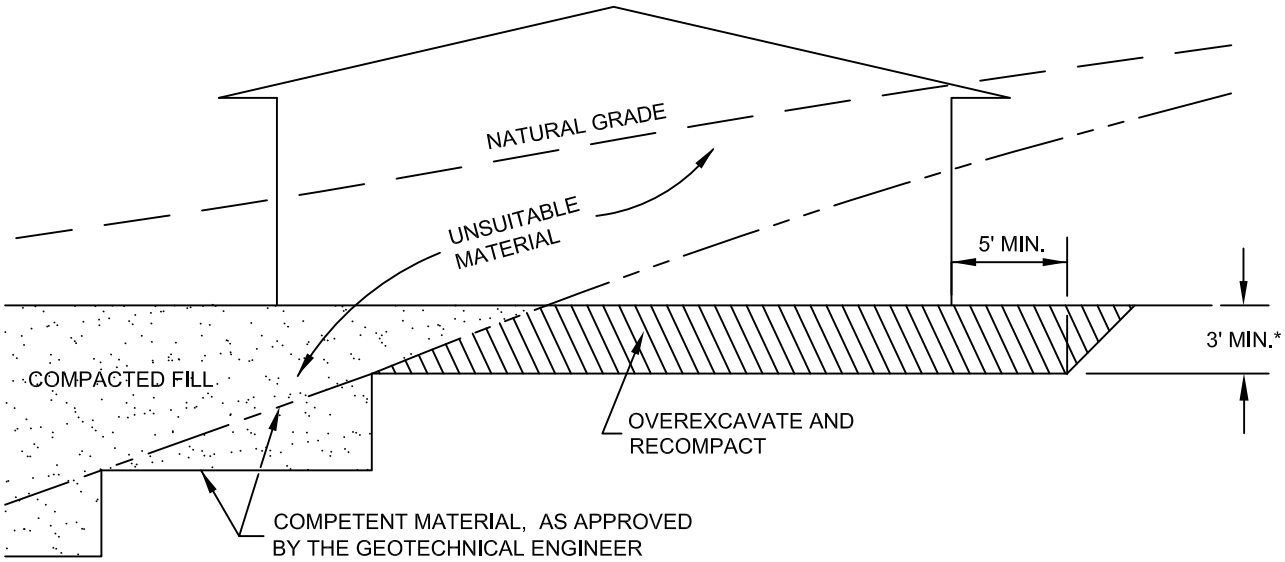
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

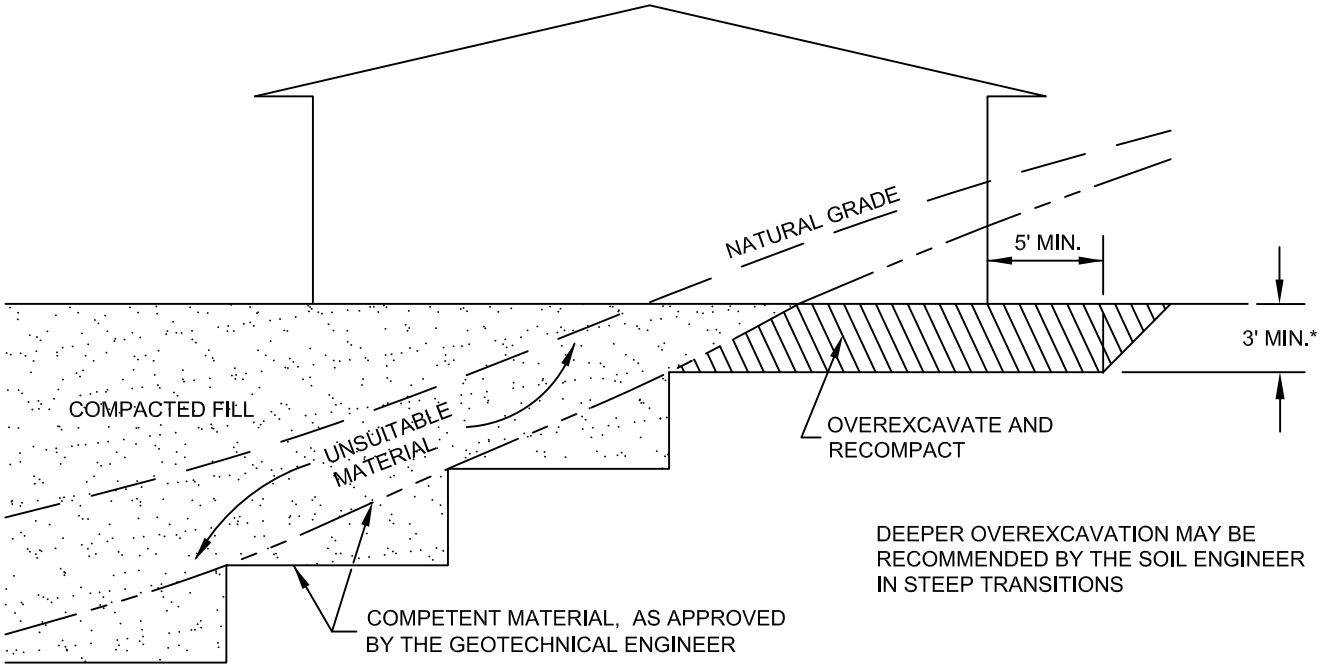
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

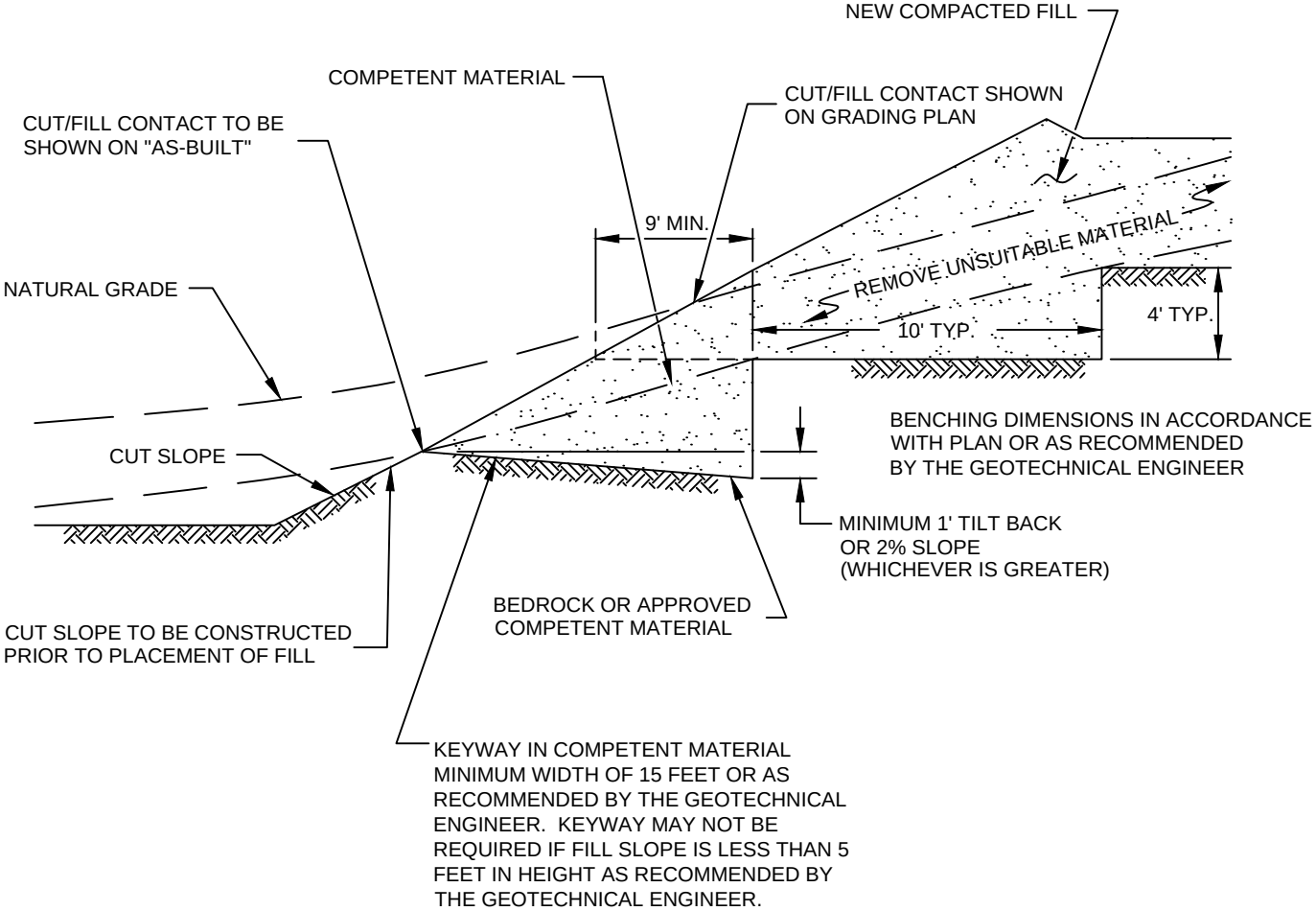
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

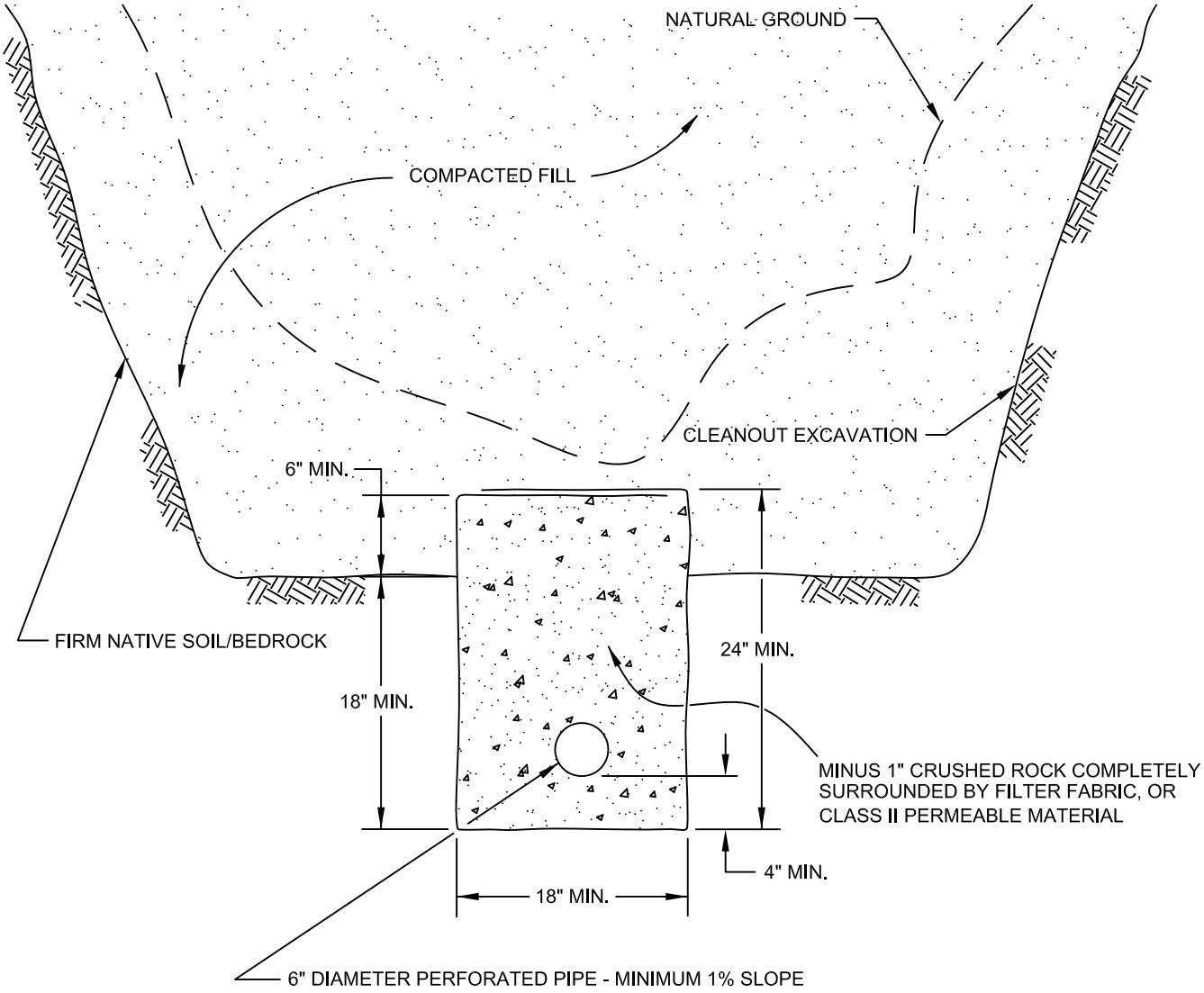
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SOUTHERN CALIFORNIA GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

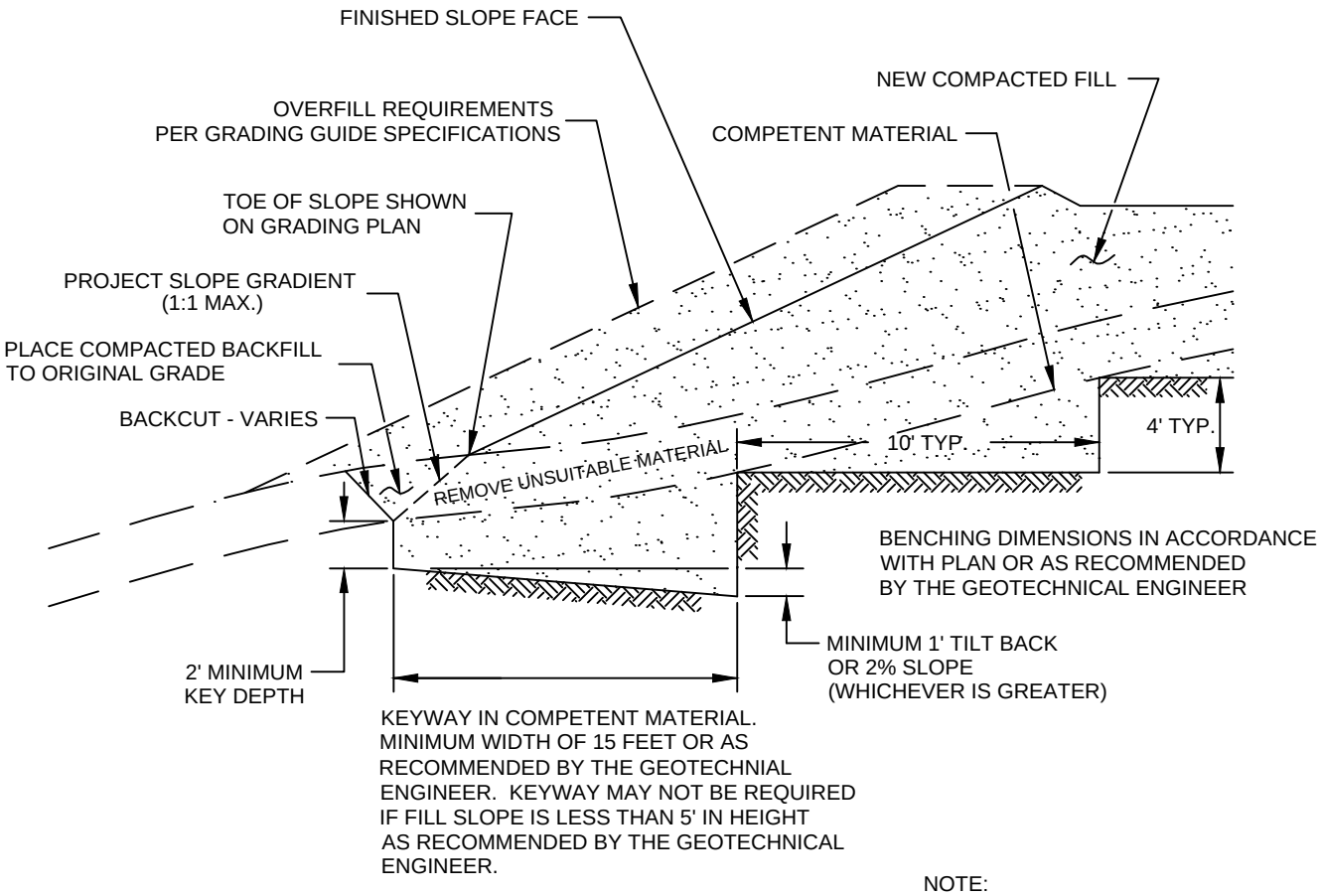
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SOUTHERN CALIFORNIA GEOTECHNICAL



NOTE:
BENCHING SHALL BE REQUIRED
WHEN NATURAL SLOPES ARE
EQUAL TO OR STEEPER THAN 5:1
OR WHEN RECOMMENDED BY
THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

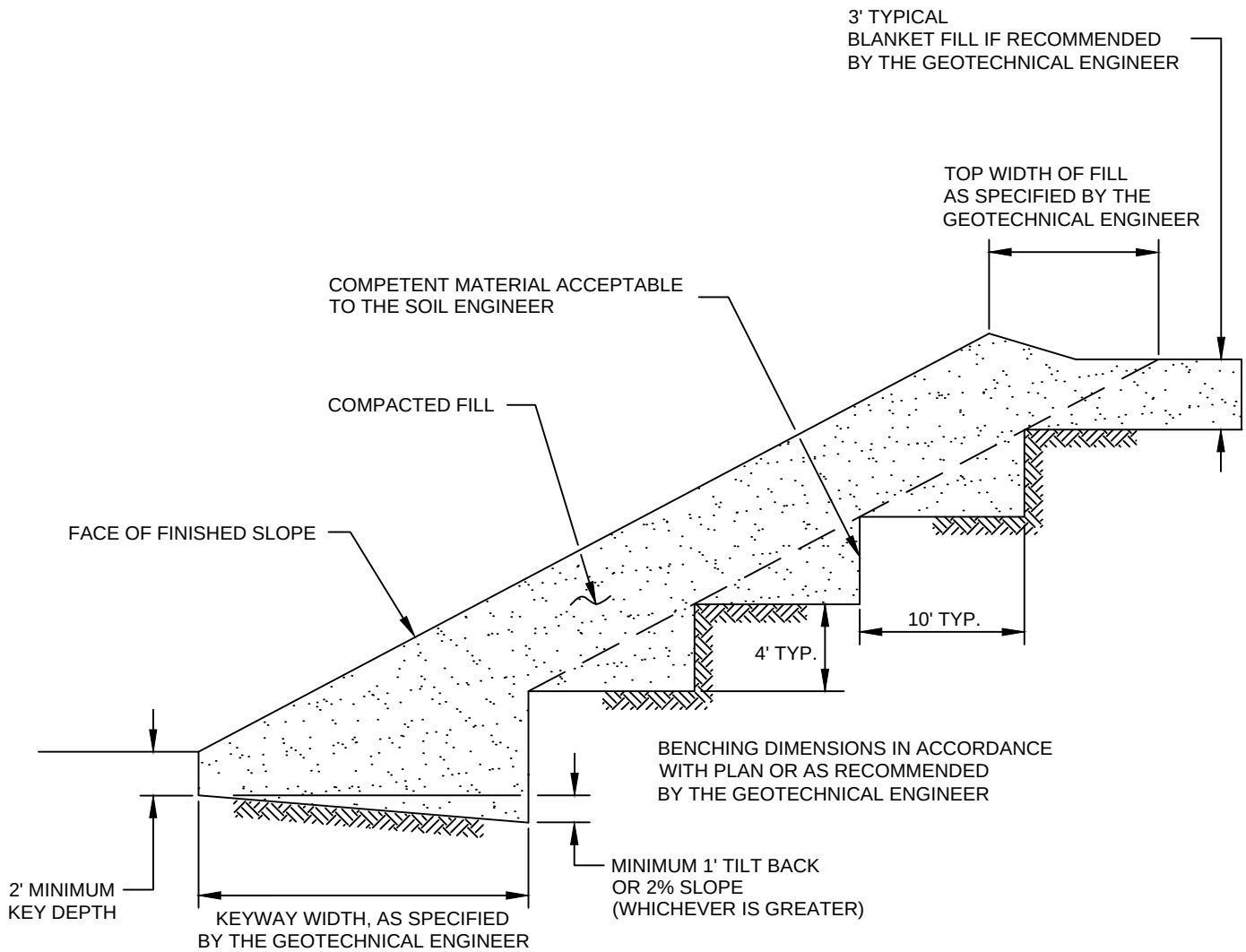
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

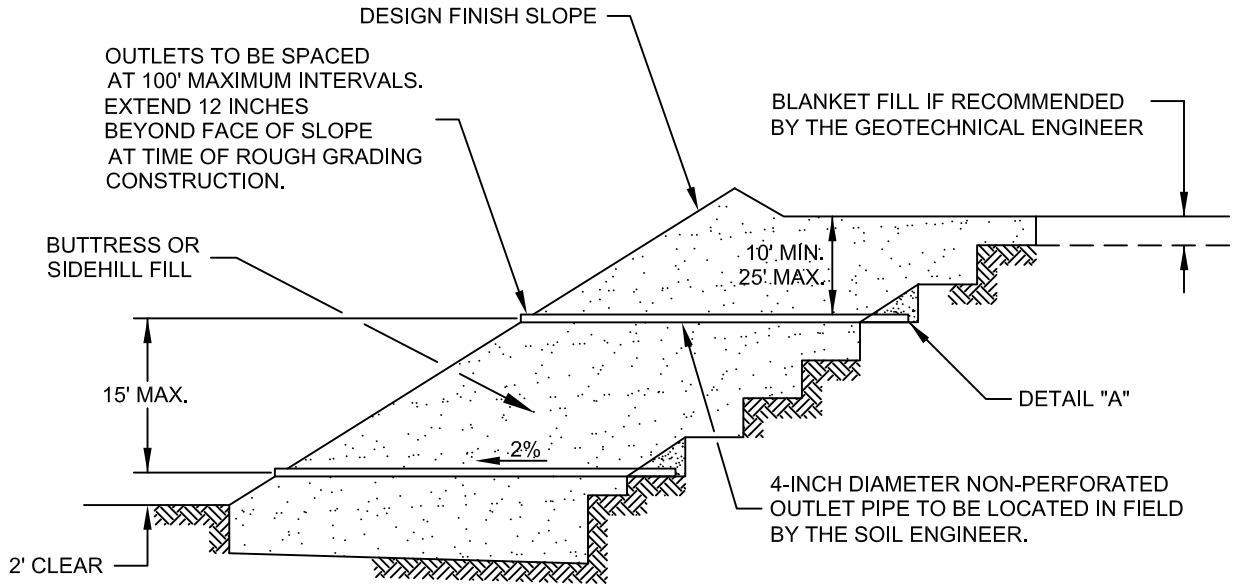
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SOUTHERN CALIFORNIA GEOTECHNICAL



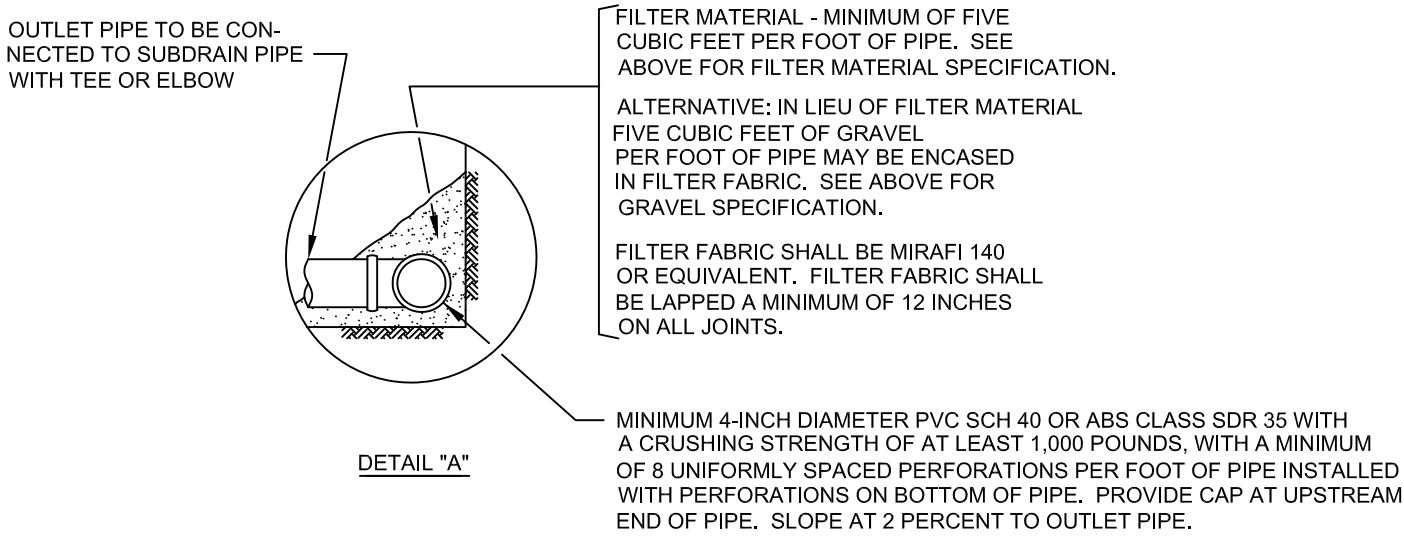
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE 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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

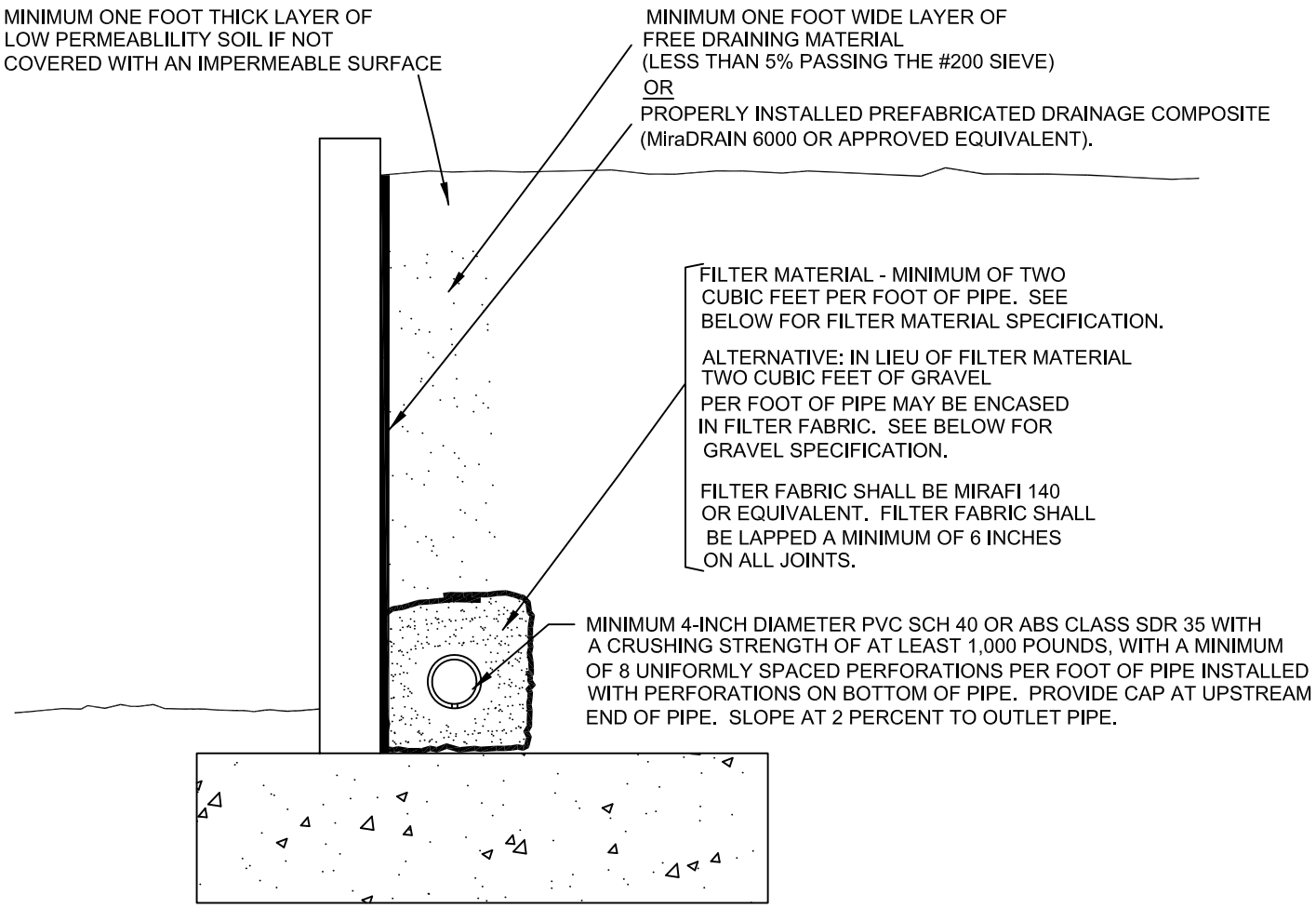
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE 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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

GRADING GUIDE SPECIFICATIONS

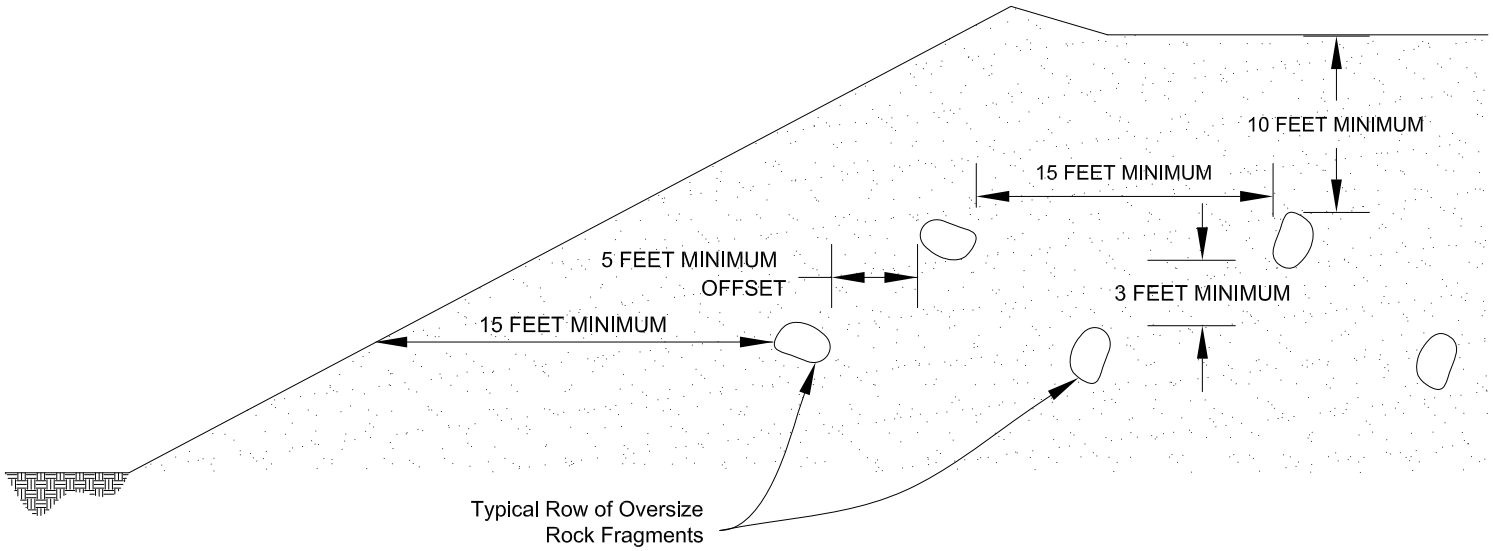
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

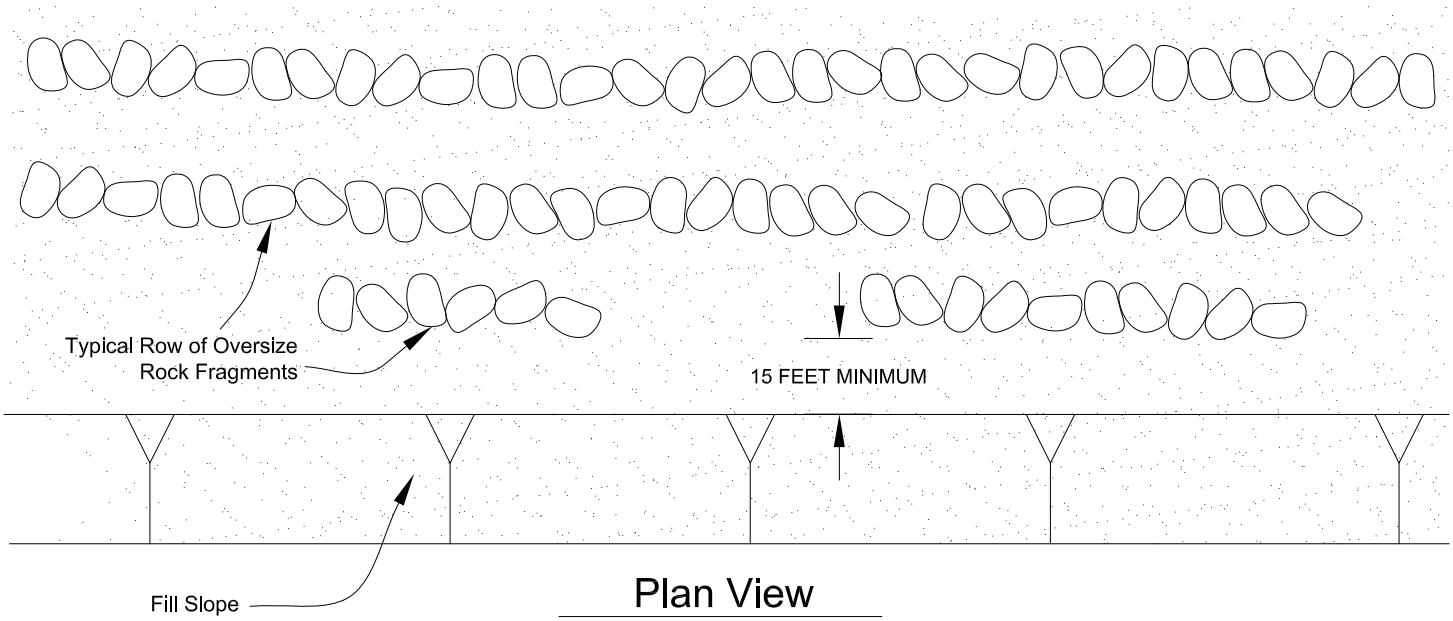
PLATE D-7

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.79089°N, 118.21772°W

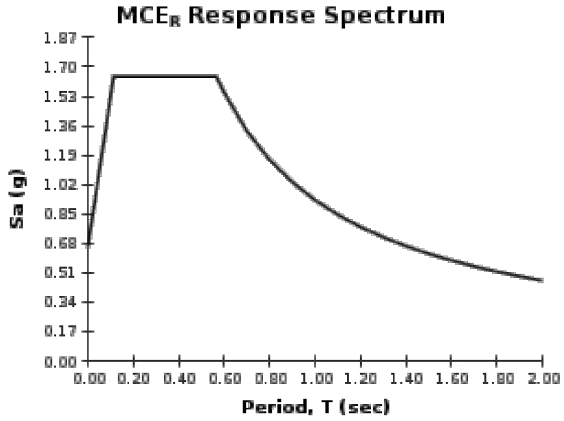
Site Soil Classification Site Class D – "Stiff Soil"

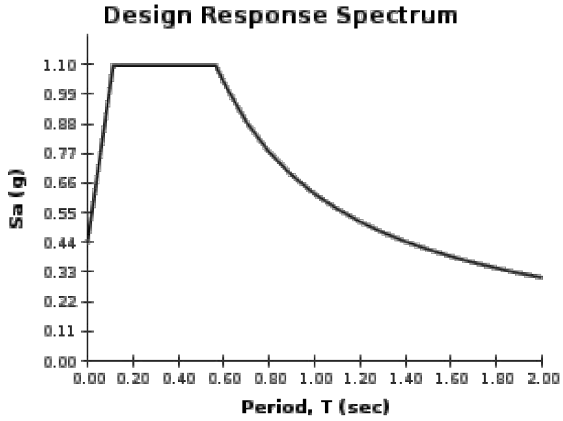
Risk Category I/II/III

USGS–Provided Output

S_S =	1.644 g	S_{MS} =	1.644 g	S_{DS} =	1.096 g
S_1 =	0.620 g	S_{M1} =	0.930 g	S_{D1} =	0.620 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

SEISMIC DESIGN PARAMETERS
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING
LONG BEACH, CALIFORNIA
DRAWN: PM CHKD: JAS
SCG PROJECT 16G144-1
PLATE E-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.642

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.642 = 0.642$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.642 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.941

From Figure 22-18 [6] C_{R1} = 0.947

SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

MCE PEAK GROUND ACCELERATION
PROPOSED COMMERCIAL/INDUSTRIAL BUILDING
LONG BEACH, CALIFORNIA

DRAWN: PM
CHKD: JAS
SCG PROJECT 16G144-1
PLATE E-2

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Building	MCE_G Design Acceleration	0.642	(g)
Project Location	Long Beach, CA	Design Magnitude	7.21	
Project Number	16G144	Historic High Depth to Groundwater	8.0	(ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	10	(ft)
		Borehole Diameter	6	(in)
Boring No.	B-1			

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.21)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
5	0	8	4	2	120		1.3	1.05	1.1	1.70	0.75	3.8	3.8	480	480	480	1.00	1.01	1.1	0.08	0.09	N/A	N/A	Above Ground Water
9	8	9.5	8.75	2	120	87	1.3	1.05	1.1	1.48	0.75	3.3	8.8	1050	1003	1050	0.98	1.02	1.07	0.11	0.12	N/A	N/A	Non-Liq: PI>18
11	9.5	10	9.75	3	120	87	1.3	1.05	1.1	1.38	0.75	4.6	10.2	1170	1061	1170	0.98	1.02	1.06	0.12	0.13	N/A	N/A	Non-Liq: PI>18
11	10	10.5	10.3	3	120	87	1.3	1.05	1.1	1.35	0.75	4.6	10.1	1230	1090	1214	0.98	1.02	1.06	0.12	0.13	N/A	N/A	Non-Liq: PI>18
11	10.5	12	11.3	3	120	46	1.3	1.05	1.1	1.32	0.75	4.4	10.1	1350	1147	1272	0.97	1.02	1.06	0.12	0.13	0.48	0.27	**Liquefiable**
13.5	12	14.5	13.3	3	120	61	1.3	1.05	1.1	1.25	0.85	4.8	10.4	1590	1262	1387	0.97	1.02	1.05	0.12	0.13	0.51	0.25	**Liquefiable**
16	14.5	17	15.8	3	120	61	1.3	1.05	1.1	1.19	0.85	4.6	10.2	1890	1406	1531	0.96	1.02	1.04	0.12	0.13	0.54	0.24	**Liquefiable**
18.5	17	19.5	18.3	5	120	95	1.3	1.05	1.1	1.12	0.95	8.0	13.5	2190	1550	1675	0.94	1.03	1.03	0.14	0.15	N/A	N/A	Non-Liq: PI>18
21	19.5	22	20.8	4	120	95	1.3	1.05	1.1	1.08	0.95	6.2	11.7	2490	1694	1819	0.93	1.02	1.02	0.13	0.14	N/A	N/A	Non-Liq: PI>18
23.5	22	24.5	23.3	4	120	95	1.3	1.05	1.1	1.04	0.95	5.9	11.4	2790	1838	1963	0.92	1.02	1.01	0.13	0.13	N/A	N/A	Non-Liq: PI>18
26	24.5	27	25.8	11	120	53	1.3	1.05	1.17	1.00	0.95	16.7	22.3	3090	1982	2107	0.91	1.06	1.01	0.24	0.25	0.59	0.43	**Liquefiable**
28.5	27	32	29.5	5	120	53	1.3	1.05	1.1	0.95	0.95	6.8	12.4	3540	2198	2323	0.89	1.02	1	0.14	0.14	0.60	0.23	**Liquefiable**
34.5	32	37	34.5	16	120	55	1.3	1.05	1.25	0.93	1	25.4	31.0	4140	2486	2611	0.87	1.11	0.96	0.56	0.59	0.60	0.98	**Liquefiable**
39.5	37	42	39.5	10	120	92	1.3	1.05	1.13	0.87	1	13.4	18.9	4740	2774	2899	0.84	1.05	0.96	0.19	0.20	0.60	0.32	**Liquefiable**
44.5	42	47	44.5	19	120	54	1.3	1.05	1.29	0.87	1	29.3	34.9	5340	3062	3187	0.82	1.12	0.9	1.09	1.10	0.59	1.84	Non-Liquefiable
49.5	47	50	48.5	23	120	7	1.3	1.05	1.3	0.85	1	34.8	35.0	5820	3293	3418	0.80	1.12	0.88	1.10	1.09	0.59	1.86	Non-Liquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Building
Project Location	Long Beach, CA
Project Number	16G144
Engineer	PM

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
5	0	8	4	3.8	0.0	3.8	N/A	0.50	0.95	0.00	8.00		0.000		0.00	Above Ground Water
9	8	9.5	8.75	3.3	5.5	8.8	N/A	0.50	0.93	0.00	1.50		0.000		0.00	Non-Liq: PI>18
11	9.5	10	9.75	4.6	5.5	10.2	N/A	0.46	0.91	0.00	0.50		0.000		0.00	Non-Liq: PI>18
11	10	10.5	10.3	4.6	5.5	10.1	N/A	0.47	0.91	0.00	0.50		0.000		0.00	Non-Liq: PI>18
11	10.5	12	11.3	4.4	5.6	10.1	0.27	0.47	0.91	0.47	1.50		0.037		0.67	**Liquefiable**
13.5	12	14.5	13.3	4.8	5.6	10.4	0.25	0.45	0.91	0.45	2.50		0.037		1.10	**Liquefiable**
16	14.5	17	15.8	4.6	5.6	10.2	0.24	0.46	0.91	0.46	2.50		0.037		1.11	**Liquefiable**
18.5	17	19.5	18.3	8.0	5.5	13.5	N/A	0.32	0.81	0.00	2.50		0.000		0.00	Non-Liq: PI>18
21	19.5	22	20.8	6.2	5.5	11.7	N/A	0.39	0.87	0.00	2.50		0.000		0.00	Non-Liq: PI>18
23.5	22	24.5	23.3	5.9	5.5	11.4	N/A	0.40	0.88	0.00	2.50		0.000		0.00	Non-Liq: PI>18
26	24.5	27	25.8	16.7	5.6	22.3	0.43	0.12	0.39	0.12	2.50		0.021		0.63	**Liquefiable**
28.5	27	32	29.5	6.8	5.6	12.4	0.23	0.36	0.85	0.36	5.00		0.033		1.96	**Liquefiable**
34.5	32	37	34.5	25.4	5.6	31.0	0.98	0.04	-0.16	0.04	5.00		0.007		0.42	**Liquefiable**
39.5	37	42	39.5	13.4	5.5	18.9	0.32	0.18	0.57	0.18	5.00		0.024		1.44	**Liquefiable**
44.5	42	47	44.5	29.3	5.6	34.9	1.84	0.02	-0.43	0.00	5.00		0.001		0.04	Non-Liquefiable
49.5	47	50	48.5	34.8	0.1	35.0	1.86	0.02	-0.43	0.00	3.00		0.001		0.02	Non-Liquefiable
											Total Deformation (in)				7.38	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Building	MCE_G Design Acceleration	0.642	(g)
Project Location	Long Beach, CA	Design Magnitude	7.21	
Project Number	16G144	Historic High Depth to Groundwater	8.0	(ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	10	(ft)
		Borehole Diameter	6	(in)
Boring No.	B-5			

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.21)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
6.5	0	8	4	4	120		1.3	1.05	1.1	1.70	0.75	7.7	7.7	480	480	480	1.00	1.01	1.1	0.10	0.11	N/A	N/A	Above Ground Water
9.5	8	10	9	3	120	85	1.3	1.05	1.1	1.43	0.75	4.8	10.4	1080	1018	1080	0.98	1.02	1.07	0.12	0.13	N/A	N/A	Non-Liq: PI>18
9.5	10	12	11	3	120	85	1.3	1.05	1.1	1.33	0.75	4.5	10.0	1320	1133	1258	0.97	1.02	1.06	0.12	0.13	N/A	N/A	Non-Liq: PI>18
14.5	12	17	14.5	2	120	40	1.3	1.05	1.1	1.23	0.85	3.1	8.7	1740	1334	1459	0.96	1.02	1.04	0.11	0.12	0.52	0.22	**Liquefiable**
19.5	17	22	19.5	14	120	8	1.3	1.05	1.24	1.08	0.95	24.4	24.8	2340	1622	1747	0.94	1.07	1.04	0.28	0.32	0.57	0.56	**Liquefiable**
24.5	22	27	24.5	2	120	96	1.3	1.05	1.1	1.02	0.95	2.9	8.4	2940	1910	2035	0.92	1.02	1.01	0.11	0.11	N/A	N/A	Non-Liq: PI>18
29.5	27	31.5	29.3	2	120	96	1.3	1.05	1.1	0.95	0.95	2.7	8.2	3510	2184	2309	0.89	1.02	1	0.11	0.11	N/A	N/A	Non-Liq: PI>18
33.5	31.5	34	32.8	2	120	96	1.3	1.05	1.1	0.91	1	2.7	8.2	3930	2386	2510	0.88	1.02	0.99	0.11	0.11	N/A	N/A	Non-Liq: PI>18
35.5	34	37	35.5	8	120	11	1.3	1.05	1.11	0.89	1	10.7	12.3	4260	2544	2669	0.86	1.02	0.98	0.14	0.14	0.60	0.22	**Liquefiable**
38.5	37	38.5	37.8	11	120	17	1.3	1.05	1.15	0.88	1	15.3	19.1	4530	2674	2798	0.85	1.05	0.97	0.20	0.20	0.60	0.33	**Liquefiable**
38.5	38.5	39.5	39	11	120	88	1.3	1.05	1.15	0.88	1	15.1	20.7	4680	2746	2870	0.85	1.05	0.96	0.21	0.22	0.60	0.36	**Liquefiable**
41	39.5	42	40.8	9	120	42	1.3	1.05	1.12	0.85	1	11.7	17.3	4890	2846	2971	0.84	1.04	0.96	0.18	0.18	0.60	0.30	**Liquefiable**
43.5	42	44.5	43.3	11	120	37	1.3	1.05	1.15	0.84	1	14.5	20.0	5190	2990	3115	0.82	1.05	0.95	0.21	0.21	0.60	0.35	**Liquefiable**
46	44.5	47	45.8	28	120	9	1.3	1.05	1.3	0.89	1	44.2	45.0	5490	3134	3259	0.81	1.12	0.88	2.00	1.97	0.59	3.33	Non-Liquefiable
48.5	47	49.5	48.3	27	120	9	1.3	1.05	1.3	0.87	1	41.9	42.6	5790	3278	3403	0.80	1.12	0.87	2.00	1.94	0.59	3.31	Non-Liquefiable
51	49.5	51.5	50.5	28	120	9	1.3	1.05	1.3	0.87	1	43.1	43.9	6060	3408	3533	0.79	1.12	0.86	2.00	1.92	0.58	3.29	Non-Liquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Building
Project Location	Long Beach, CA
Project Number	16G144
Engineer	PM

Boring No. B-5

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
6.5	0	8	4	7.7	0.0	7.7	N/A	0.50	0.95	0.00	8.00		0.000		0.00	Above Ground Water
9.5	8	10	9	4.8	5.5	10.4	N/A	0.45	0.91	0.00	2.00		0.000		0.00	Non-Liq: PI>18
9.5	10	12	11	4.5	5.5	10.0	N/A	0.47	0.91	0.00	2.00		0.000		0.00	Non-Liq: PI>18
14.5	12	17	14.5	3.1	5.6	8.7	0.22	0.50	0.94	0.50	5.00		0.040		2.42	**Liquefiable**
19.5	17	22	19.5	24.4	0.4	24.8	0.56	0.09	0.25	0.09	5.00		0.019		1.15	**Liquefiable**
24.5	22	27	24.5	2.9	5.5	8.4	N/A	0.50	0.94	0.00	5.00		0.000		0.00	Non-Liq: PI>18
29.5	27	31.5	29.3	2.7	5.5	8.2	N/A	0.50	0.94	0.00	4.50		0.000		0.00	Non-Liq: PI>18
33.5	31.5	34	32.8	2.7	5.5	8.2	N/A	0.50	0.94	0.00	2.50		0.000		0.00	Non-Liq: PI>18
35.5	34	37	35.5	10.7	1.6	12.3	0.22	0.37	0.85	0.37	3.00		0.033		1.18	Liquefiable
38.5	37	38.5	37.8	15.3	3.9	19.1	0.33	0.18	0.56	0.18	1.50		0.024		0.43	Liquefiable
38.5	38.5	39.5	39	15.1	5.5	20.7	0.36	0.15	0.48	0.15	1.00		0.022		0.27	**Liquefiable**
41	39.5	42	40.8	11.7	5.6	17.3	0.30	0.21	0.65	0.21	2.50		0.026		0.78	**Liquefiable**
43.5	42	44.5	43.3	14.5	5.5	20.0	0.35	0.16	0.52	0.16	2.50		0.023		0.69	**Liquefiable**
46	44.5	47	45.8	44.2	0.7	45.0	3.33	0.00	-1.19	0.00	2.50		0.000		0.00	Non-Liquefiable
48.5	47	49.5	48.3	41.9	0.7	42.6	3.31	0.00	-1.00	0.00	2.50		0.000		0.00	Non-Liquefiable
51	49.5	51.5	50.5	43.1	0.7	43.9	3.29	0.00	-1.10	0.00	2.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				6.92	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

APPENDIX E

RBF "HYDROLOGY & HYDRAULICS REPORT FOR TRACT 52467" EXCERPTS

HYDROLOGY & HYDRAULICS REPORT

FOR

TENTATIVE TRACT NO. 52467

LONG BEACH, CALIFORNIA

Prepared for:

C.S.U.L.B. Foundation
6300 State University Drive, Suite 260
Long Beach, California 90815

Prepared by:

Robert Bein, William Frost & Associates

Contact Persons:
Sean Davis
Jeffrey Langdon, RCE 42429

JN 33716
September 30, 1999

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I INTRODUCTION

Robert Bein, William Frost and Associates (RBF) has been retained by the owner and developer of Tentative Tract No. 52467, C.S.U.L.B. Foundation, to prepare improvement plans for T.T. No. 52467. The storm drain improvements included in this submittal provide the appropriate levels of storm water conveyance and protection for development of T.T. No. 52467. The Hydrology and Hydraulics Analysis contained in this report supports the design used in the storm drain improvements.

II HYDROLOGY

The existing conditions of the site were analyzed to determine impact from offsite runoff and if existing facilities could be used in the design of the new storm drain system. This site was formerly used for naval housing and now will be redeveloped into an industrial/commercial development. The site will be rough graded and developed for companies to lease. Bordering the site on the northeast corner of the site is a Federal Job Corps Center with runoff self contained and released into the existing 48" storm drain system. The northwest section of the project currently borders existing naval housing which drains onto Technology Place and drains into an existing catch basin which drains into the 48" storm drain system. This does not impact the storm drain system under design. To the west is San Gabriel Avenue which has no impact on the site. South of the project is West 19th Street and Pacific Coast Highway. West 19th Street receives runoff from the site and drains into an existing catch basin that connects to the 54" storm drain. On the east side of the project is the Long Beach City Police Department which drains onto Cota Avenue which in turn drains towards Pacific Coast Highway (PCH) shich drains westerly. There is a cross-gutter at the intersection of Technology Place and PCH that caries the drainage to an existing a catch basin at the intersection of River Avenue and PCH. . This connects to the storm drain system down stream of the project.

The rainfall zones and soil classification areas within the project site were identified from the Los Angeles County Department of Public Works Hydrology Manual(December 1991). All hydrologic computations were performed using the Los Angeles County Department of Public Works Hydrology Manual guidelines. Hydrologic computations to determine the 25-year and 50-year capital storm were performed using the Los Angeles County Department of Public Works Modified Rational Method hydrology methodology. The Los Angeles County Rational Method was used to compute the time of concentrations for subarea inputs of the Modified Rational Method. The Los Angeles County Rational Method and F0601 Modified Rational Hydrology Computer Programs were used in generating results of this report.

In determining the "Time of Concentration" the project was divided into subareas based on changes of slopes, changes of conveyance, and lateral entry. The hydrologic maps from the LACDPW Hydrology Manual indicate the soil type in the study area is soil type 6. The percent impervious was estimated to be 0.91 based on the average for industrial land use. The storm rainfall pattern for Zone K was used for a 25-year and 50-year frequency.

III DETENTION BASIN

The new storm drain system requires a design which includes a detention basin located at the intersection of Technology Place and Pacific Coast Highway. A detention basin analysis was completed by Psomas and Associates in August, 1997 to determine the a storage volume required to meet the existing storm drain system capacity. The analysis assumed the future land use would be single family housing and the detention basin would have a 10-year storm inflow rate. The proposed detention basin has been designed for a 25-year storm event and commercial/industrial land use with the outlet from the basin not to exceed the existing discharge from the site. The detention basin analysis in the Psomas report determined a factor for the allowable outflow from the detention basin. This factor is used to determine the allowable runoff from each subarea. The condition established by Psomas unreasonably requires a detention basin to mitigate the effects the watershed has on the storm drain system. The proposed detention basin successfully detains the increased flow caused by the proposed development.

The outflow from the basin was calculated using the LACDPW Basin Routing program. It used the hydrograph from the Modified Rational Method program and input stage storage and stage outflow values to perform the flood routing analysis. These calculations along with the output are found in the Appendix.

The storage volume of the detention basin is 24,400 ft^3 and the approved tentative tract map has a 24,000 ft^3 detention basin.

During a 25-year storm the detention basin will detain the peak flow and release 40 cfs through the new 36" pipe. The control for the detention basin is the junction of the existing 54", 48", 36", and 18" pipe at the intersection of W. 19th Street and River Avenue. The tailwater elevation was set for the 25-year storm by determining the existing runoff into the system and finding the depth of a full flow diameter pipe at the outlet into the junction structure. The flowrate was 39 cfs with 18 acres contributing to the storm drain system. The assumed slope of the pipe was 0.1% and the elevation of the invert is 0.65 feet. The depth of water is 3.55 feet. The tailwater elevation is 4.2 feet. The Appendix contains the data used to calculated the size and capacity of the detention basin.

IV HYDRAULICS

A portion of the existing storm drain system on the project site will still be used. The four catch basins located on Technology Place will be used to some extent. The two sump catch basins will be replaced and the pipe connecting the catch basins will be replaced with 35" by 24" Trishield arch pipe. The old alignment of the storm drain will not be used and a new alignment within Technology Place is a reinforced concrete box storm drain. The existing 43" CSP storm drain that connects to a junction structure at the intersection of West 19th Street and old River Avenue will be replaced by a 36" RCP. It will join the new 7' by 3' reinforced concrete box at the entrance to the detention basin. Existing catch basins on West 19th Street and Technology Place at the north end will be used. An analysis was done to determine the impact on the existing structures due to the change in runoff from the site.

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
MODIFIED RATIONAL METHOD HYDROLOGY

TT 52467 INDUSTRIAL SITE 10-YEAR STORM FN:TECH10-DAT STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
33716	1A	4.	8.	4.	8.	4	40.	0.00100	1.50	0.00	0.	6	16	K10	0.91
33716	2A	8.	15.	12.	23.	4	110.	0.00100	1.50	0.00	0.	6	16	K10	0.91
33716	3B	3.	7.	3.	7.	4	40.	0.00100	1.50	0.00	0.	6	12	K10	0.91
33716	4B	3.	8.	6.	15.	4	1.	0.00100	1.50	0.00	0.	6	9	K10	0.91

****
* CONFLUENCE Q'S *
* 33716 5A TA 1155 QA 23. QAB 38. QB 15. 33716 5B TB 1154 QB 15. QBA 38. QA 23. *
* 33716 5AB TAB 1154 QAB 38. QA 23. QB 15. *
****

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
33716	5AB	6.	15.	18.	38.	4	530.	0.00100	3.00	0.00	0.	6	0	K10	0.00
33716	6A	4.	10.	22.	47.	4	50.	0.00100	3.00	0.00	0.	6	10	K10	0.91
33716	7A	2.	5.	24.	52.	4	100.	0.00100	3.00	0.00	0.	6	9	K10	0.91

MODIFIED RATIONAL METHOD HYDROLOGY

TT 52467 INDUSTRIAL SITE 25-YEAR STORM FN:TECH25-DAT STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
33716	1A	4.	9.	4.	9.	4	40.	0.00500	1.50	0.00	0.	6	16	K25	0.91
33716	2A	8.	17.	12.	26.	4	110.	0.00500	1.50	0.00	0.	6	16	K25	0.91
33716	3B	3.	7.	3.	7.	4	40.	0.00500	1.50	0.00	0.	6	12	K25	0.91
33716	4B	3.	9.	6.	16.	4	1.	0.00500	1.50	0.00	0.	6	9	K25	0.91

****

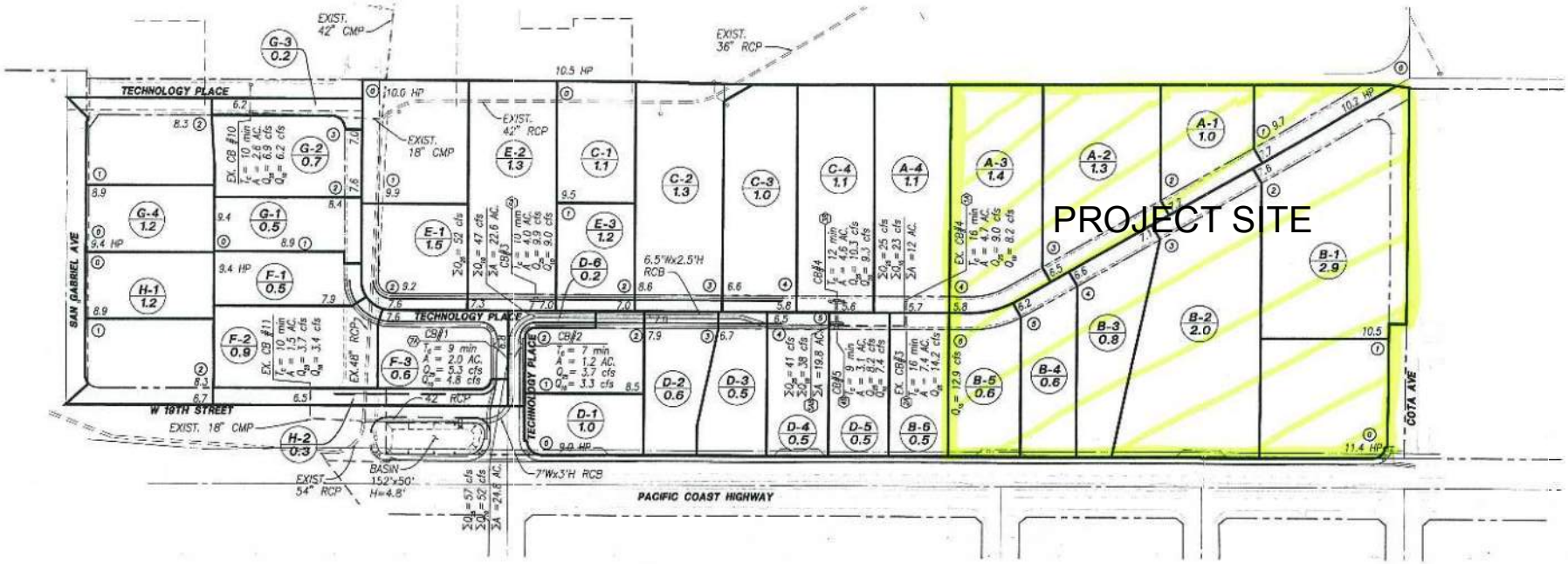
* CONFLUENCE Q'S *

* 33716 5A TA 1155 QA 26. QAB 42. QB 16. 33716 5B TB 1154 QB 16. QBA 42. QA 26. *

* 33716 5AB TAB 1154 QAB 42. QA 26. QB 16. *

****

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
33716	5AB	6.	16.	18.	42.	4	530.	0.00500	3.00	0.00	0.	6	0	K25	0.00
33716	6A	4.	11.	22.	52.	4	50.	0.00500	3.00	0.00	0.	6	10	K25	0.91
33716	7A	2.	6.	24.	58.	4	100.	0.00500	3.00	0.00	0.	6	9	K25	0.91



LEGEND

A / 3.2	SUBAREA NO. / SUBAREA AREA – Acre
①	TIME OF CONCENTRATION NODE
⑯	TIME OF CONCENTRATION NODE
9.4 HP	HIGH POINT ELEVATION
Q_{25}	25 YEAR STORM CONFLUENCE
——	AREA TRIBUTARY BOUNDARY

GRAPHIC SCALE: 1"=80'

PLAN PREPARED BY: Robert Bein, William Frost & Associates
PROFESSIONAL ENGINEERS, PLANNERS & SURVEYORS

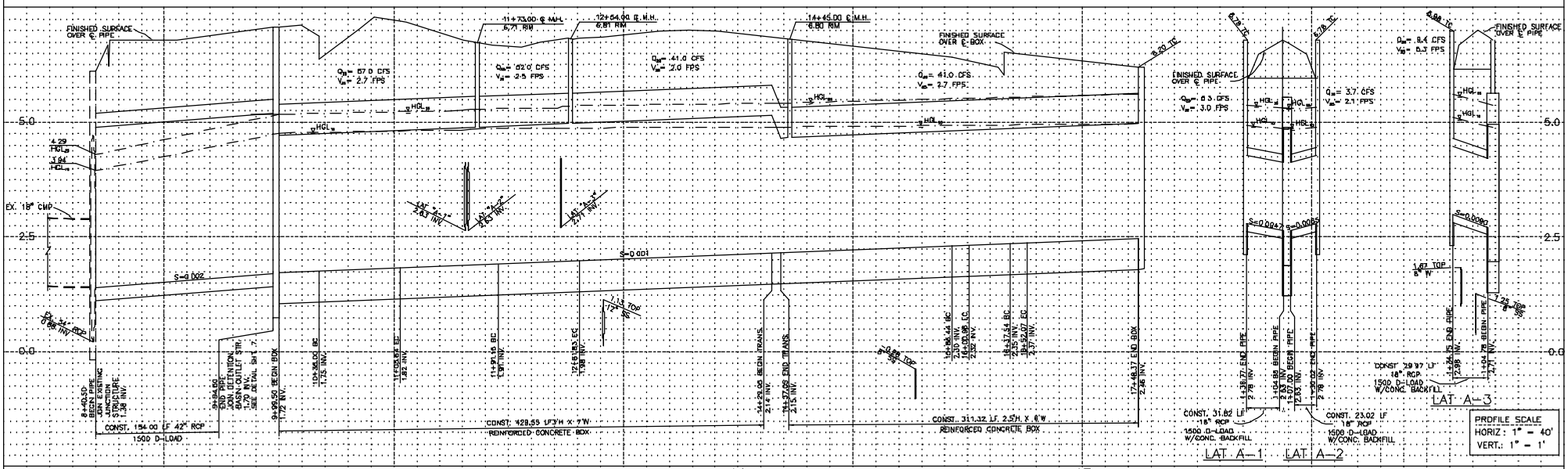
REVIEWED BY: JEFFREY C. LANGDON R.C.E. # 42429 DATE:

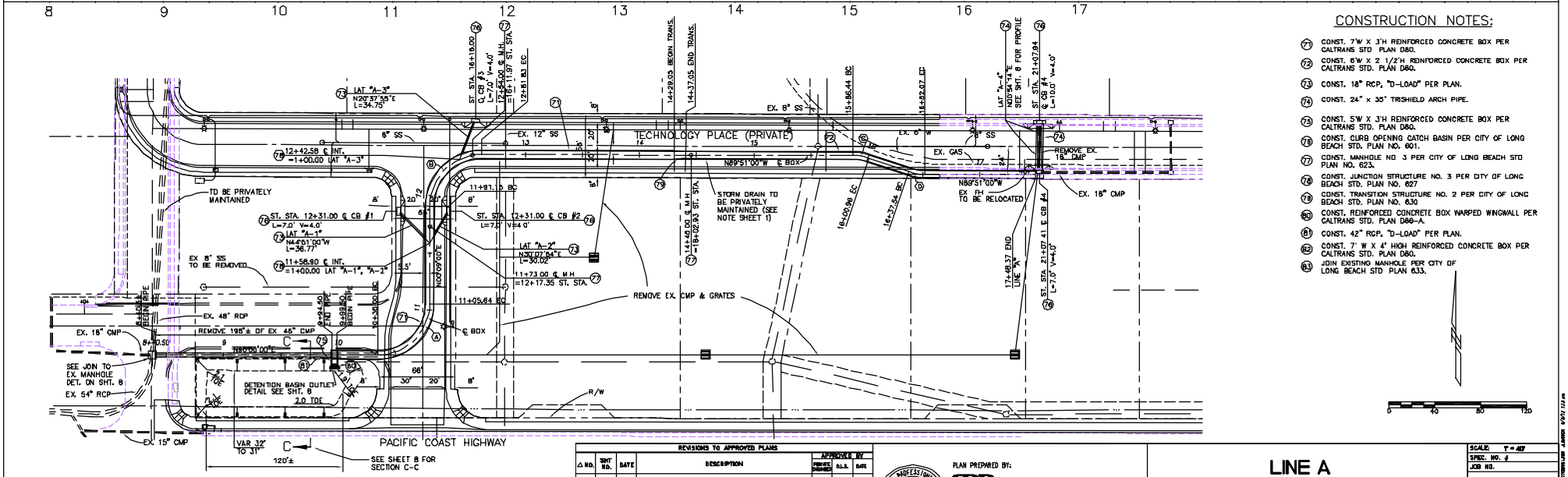
REGISTERED PROFESSIONAL ENGINEER JEFFREY C. LANGDON NO. 42429 Exp. 3-31-00 CIVIL STATE OF CALIFORNIA

HYDROLOGY MAP
PROPOSED
TENTATIVE TRACT 52467

DEPARTMENT OF PUBLIC WORKS, CITY OF LONG BEACH, BUREAU OF ENGINEERING CALIFORNIA

SCALE: 1"=80'
SPEC. NO. #
JOB NO.
SHEET 1 OF 1
DWG NO.





CONSTRUCTION NOTES:

(71) CONST. 7'W X 3'H REINFORCED CONCRETE BOX PER CALTRANS STD PLAN D80.

(72) CONST. 6'W X 2 1/2'H REINFORCED CONCRETE BOX PER CALTRANS STD. PLAN D80.

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(78) CONST. JUNCTION STRUCTURE NO. 3 PER CITY OF LONG BEACH STD. PLAN NO. 627

(79) CONST. TRANSITION STRUCTURE NO. 2 PER CITY OF LONG BEACH STD. PLAN NO. 630

(80) CONST. REINFORCED CONCRETE BOX WARPED WINGWALL PER CALTRANS STD. PLAN D86-A.

(81) CONST. 42" RCP, "D-LOAD" PER PLAN.

(82) CONST. 7' W X 4' HIGH REINFORCED CONCRETE BOX PER CALTRANS STD. PLAN D80.

(83) JOIN EXISTING MANHOLE PER CITY OF LONG BEACH STD PLAN 633.

0 40 80 120

CURVE DATA TABLE

○	DELTA	RADIUS	LENGTH	TANGENT
A	90°56'31"	45.00'	70.64'	44.99'
B	90°00'00"	45.00'	70.69'	45.00'
C	18°29'38"	45.00'	14.52'	7.33'
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REVISIONS TO APPROVED PLANS

△ NO.	SHT NO.	DATE	DESCRIPTION	APPROVED BY PRIVATE ENGINEER	D.L.B.	DATE

REGISTERED PROFESSIONAL ENGINEER JEFFREY C. LANSDOWN No. 42429 Exp. 3-31-04 CIVIL STATE OF CALIFORNIA

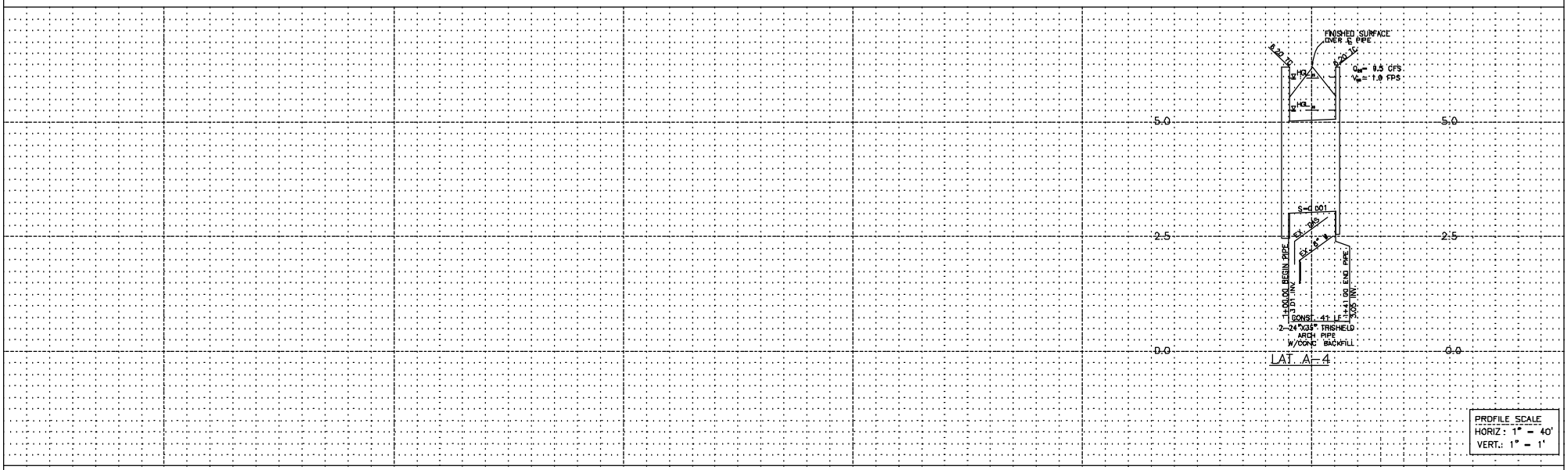
PLAN PREPARED BY:
Robert Bein, William Frost & Associates
PROFESSIONAL ENGINEERS, PLANNERS & SURVEYORS
P.O. BOX 57057 • 14725 ALTON PARKWAY, IRVINE, CALIFORNIA 92618-7057
(949) 472-3505 FAX (949) 472-8122

REVIEWED BY: DATE:
JEFFREY C. LANSDOWN R.C.E. # 42429

LINE A
STA. 10+00.00 TO 17+48.37
STORM DRAIN PLAN

DEPARTMENT OF PUBLIC WORKS, CITY OF LONG BEACH. BUREAU OF ENGINEERING CALIFORNIA

SCALE: 1" = 40'
SPEC. NO. #
JOB NO.
SHEET 7 OF 10
DWG NO.



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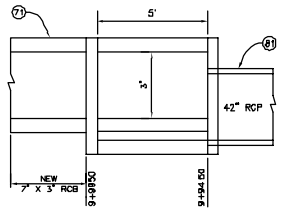
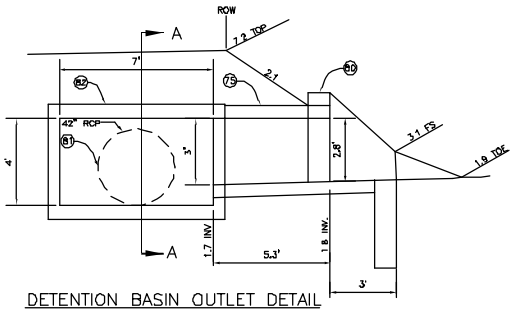
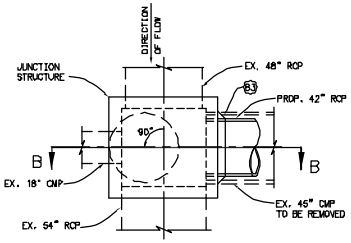


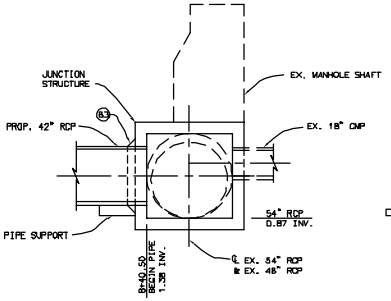
SECTION A-A



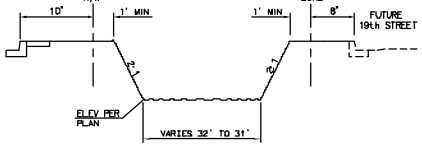
DETENTION BASIN OUTLET DETAIL
N.T.S.



JOIN TO EXISTING MANHOLE DETAIL
N.T.S.



SECTION B-B



SECTION C-C
N.T.S.

REVISIONS TO APPROVED PLANS						
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REGISTERED PROFESSIONAL ENGINEER JEFFREY C. LANGDON No. 42429 Exp. 3-31-04 CIVIL STATE OF CALIFORNIA

PLAN PREPARED BY:
Robert Bain, Wilburn Frost & Associates
PROFESSIONAL ENGINEERS, PLANNERS & SURVEYORS
P.O. BOX 57157 • 14735 ALDER AVENUE, IRVINE, CALIFORNIA 92619-7157
(949) 472-3948 FAX (949) 472-8122

REVIEWED BY:
DATE:
JEFFREY C. LANGDON R.C.E # 42429

LINE A
STA. 10+00.00 TO 17+48.37
STORM DRAIN DETAILS

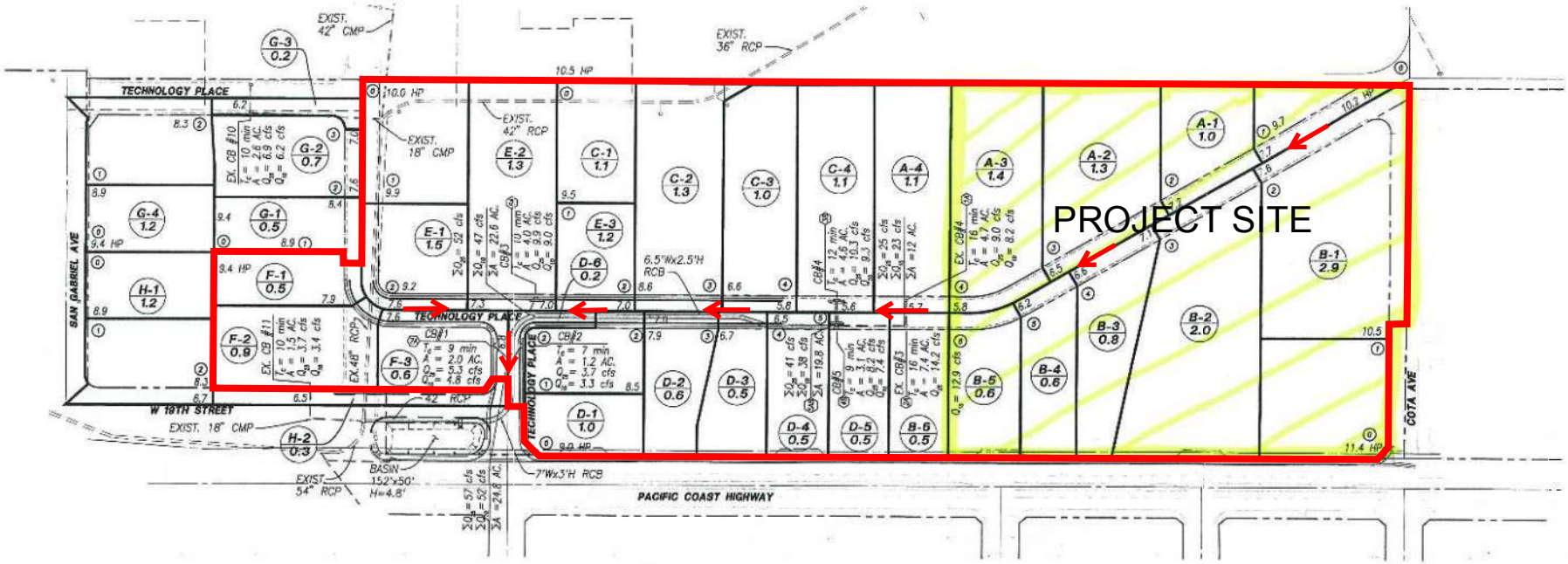
DEPARTMENT OF PUBLIC WORKS, CITY OF LONG BEACH. BUREAU OF ENGINEERING CALIFORNIA

SCALE: 1" = 40'
SPEC. NO. #
JOB NO.
SHEET 8 OF 10
DWG NO.

APPENDIX F

LINE "A" EXHIBIT AND CALCULATIONS

LINE "A" DRAINAGE AREAS



LINE "A"
$Area_{total}$ = 26.0 Acres
% Impervious = 91%
Q_{25} = 40.36 cfs
V_{25} = 9.15 ac-ft

LEGEND

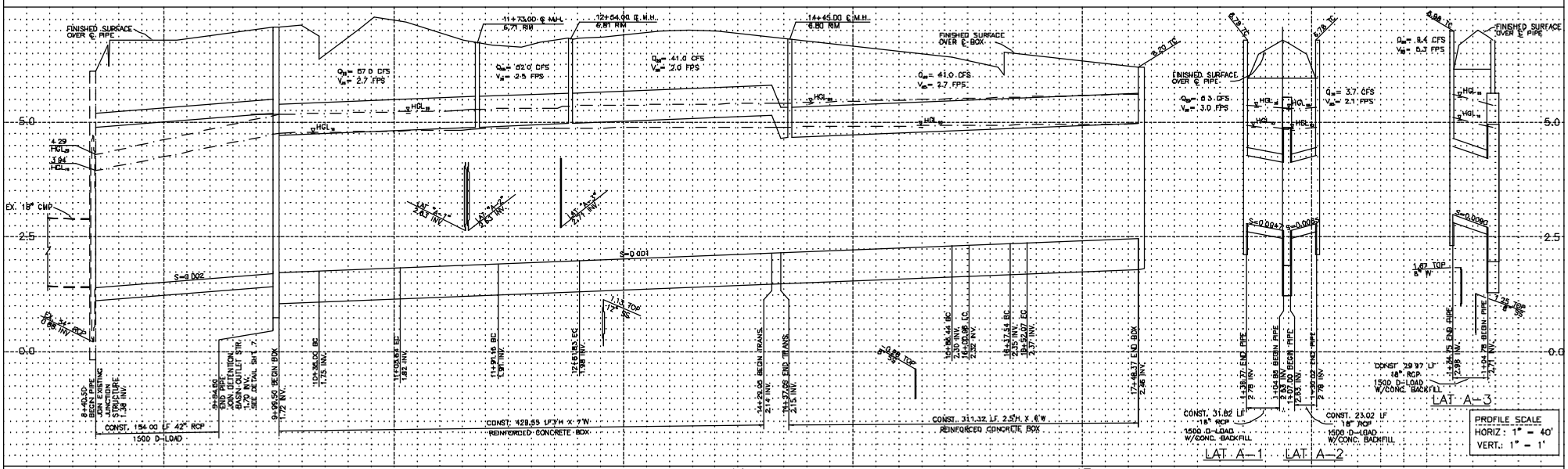
A / 3.2	SUBAREA NO. / SUBAREA AREA – Acre
1	TIME OF CONCENTRATION NODE
1A	TIME OF CONCENTRATION NODE
9.4 HP	HIGH POINT ELEVATION
Q_{25}	25 YEAR STORM CONFLUENCE
———	AREA TRIBUTARY BOUNDARY

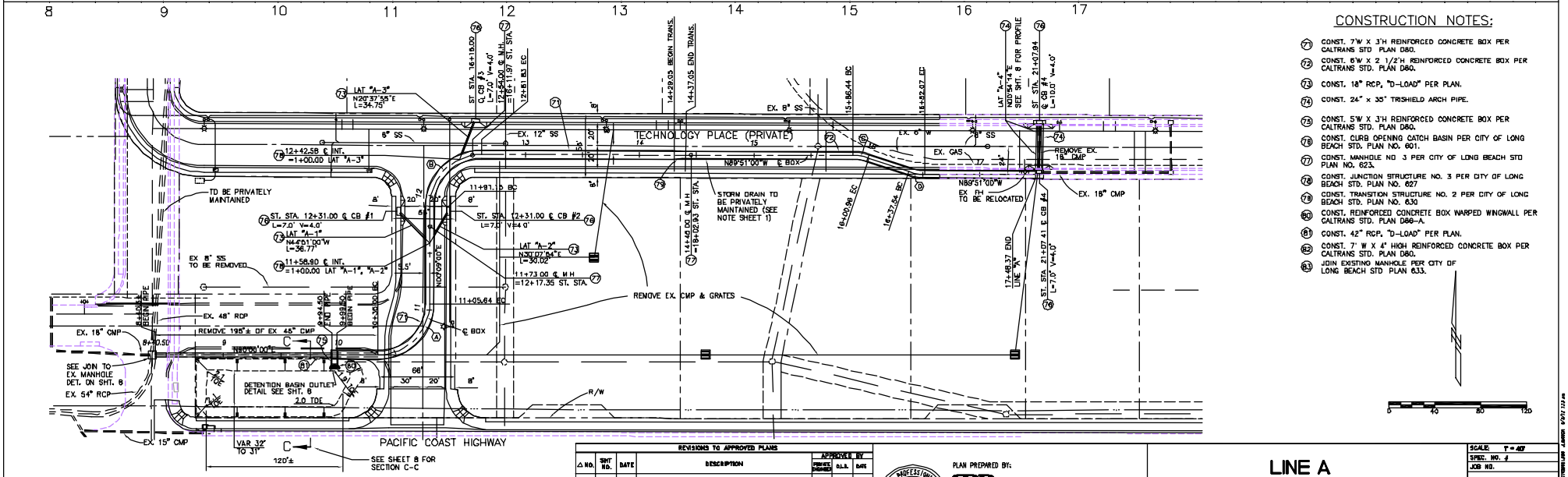
GRAPHIC SCALE: 1"=80'

PLAN PREPARED BY: Robert Bein, William Frost & Associates
REVIEWED BY: JEFFREY C. LANGDON, R.C.E. # 42429

HYDROLOGY MAP
PROPOSED
TENTATIVE TRACT 52467

DEPARTMENT OF PUBLIC WORKS, CITY OF LONG BEACH, BUREAU OF ENGINEERING, CALIFORNIA





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REVIEWED BY: DATE:
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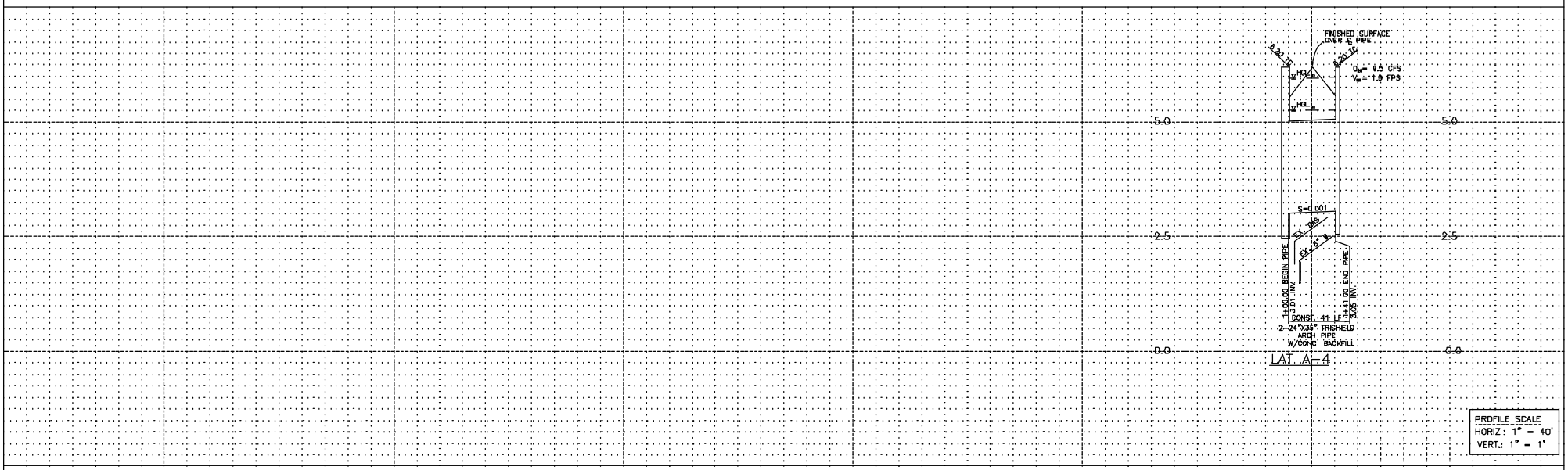
LINE A
STA. 10+00.00 TO 17+48.37
STORM DRAIN PLAN

DEPARTMENT OF PUBLIC WORKS, CITY OF LONG BEACH. BUREAU OF ENGINEERING CALIFORNIA

SCALE: 1" = 40'
SPEC. NO. #
JOB NO.
SHEET 7 OF 10
DWG NO.



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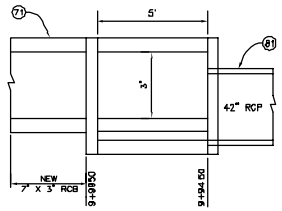
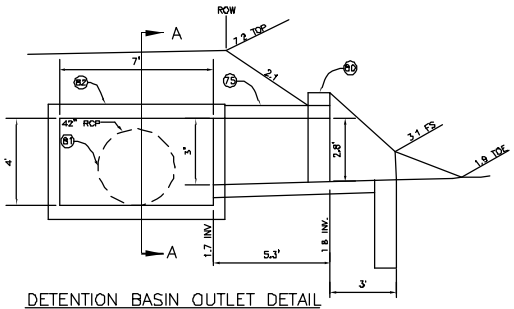
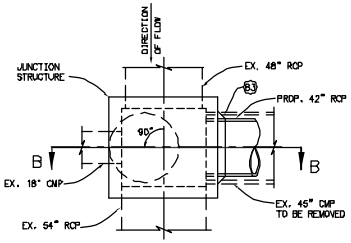


SECTION A-A



DETENTION BASIN OUTLET DETAIL
N.T.S.



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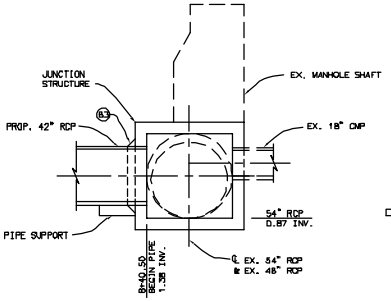
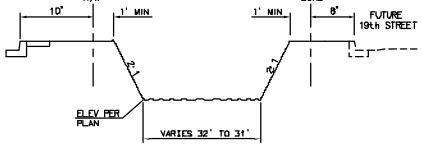


SECTION B-B



SECTION C-C
N.T.S.

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REVIEWED BY:
DATE:
JEFFREY C. LANGDON R.C.E # 42429

LINE A
STA. 10+00.00 TO 17+48.37
STORM DRAIN DETAILS

DEPARTMENT OF PUBLIC WORKS, CITY OF LONG BEACH. BUREAU OF ENGINEERING CALIFORNIA

SCALE: 1" = 40'
SPEC. NO. #
JOB NO.
SHEET 8 OF 10
DWG NO.

3' x 7' RCB

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.00100	ft/ft
Height	3.00	ft
Bottom Width	7.00	ft
Discharge	40.36	ft^3/s

Results

Normal Depth	1.53	ft
Flow Area	10.71	ft^2
Wetted Perimeter	10.06	ft
Hydraulic Radius	1.06	ft
Top Width	7.00	ft
Critical Depth	1.01	ft
Percent Full	51.0	%
Critical Slope	0.00344	ft/ft
Velocity	3.77	ft/s
Velocity Head	0.22	ft
Specific Energy	1.75	ft
Froude Number	0.54	
Discharge Full	78.41	ft^3/s
Slope Full	0.00377	ft/ft
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	51.00	%
Downstream Velocity	Infinity	ft/s

3' x 7' RCB

GVF Output Data

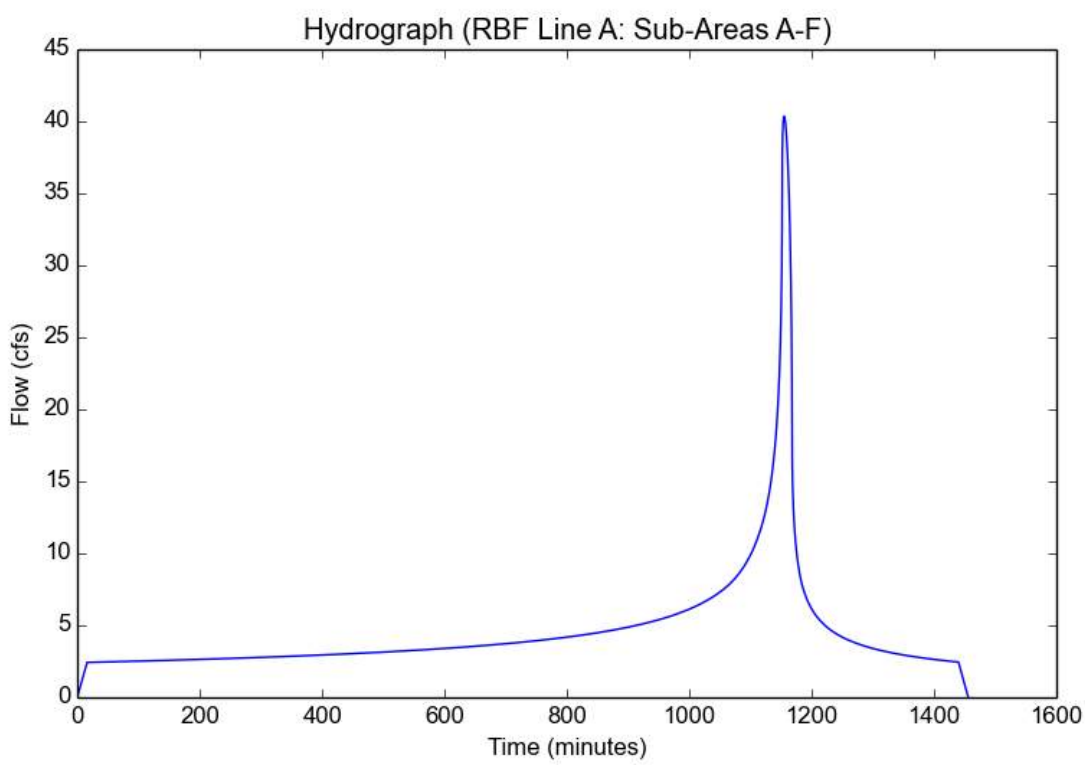
Upstream Velocity	Infinity	ft/s
Normal Depth	1.53	ft
Critical Depth	1.01	ft
Channel Slope	0.00100	ft/ft
Critical Slope	0.00344	ft/ft

Peak Flow Hydrologic Analysis

File location: //fuscoe.corp/IRV/Panzura/Projects/309/019/_Support Files/Reports/EIR Tech Reports/MND/Hydrology and WQ Report/HydroCalc/RBF Lin
Version: HydroCalc 1.0.2

Input Parameters

Project Name	RBF Line A
Subarea ID	Sub-Areas A-F
Area (ac)	26.0
Flow Path Length (ft)	1600.0
Flow Path Slope (vft/hft)	0.01
50-yr Rainfall Depth (in)	5.8
Percent Impervious	0.91
Soil Type	6
Design Storm Frequency	25-yr
Fire Factor	0
LID	False

Output Results

Modeled (25-yr) Rainfall Depth (in)	5.0924
Peak Intensity (in/hr)	1.7588
Undeveloped Runoff Coefficient (Cu)	0.7058
Developed Runoff Coefficient (Cd)	0.8825
Time of Concentration (min)	16.0
Clear Peak Flow Rate (cfs)	40.3557
Burned Peak Flow Rate (cfs)	40.3557
24-Hr Clear Runoff Volume (ac-ft)	9.153
24-Hr Clear Runoff Volume (cu-ft)	398704.6076