Appendix F   LID Report and Hydrology and Detention Report

Appendix

This page intentionally left blank.

LOW IMPACT DEVELOPMENT PLAN

CITY OF IRWINDALE, CA.

PROJECT:

INDUSTRIAL BUILDING
4416 Azusa Canyon Road
Azusa, CA

PREPARED FOR:

Rexford Industrial Realty
6222 Wilshire Blvd. Suite 270
Los Angeles, CA 90048
Project Contact Person: Paul Mino
(213) 624-4040

Prepared by:

THE G4 GROUP INC.
Consulting Engineers

5301 North Commerce Ave, Suite H
Moorpark, CA. 93021
(805) 523-0010
Contact: Kevin P. Garrity

May 17, 202
Revised October 4, 2021

TABLE OF CONTENTS

LIST OF APPENDICES

1.0   Vicinity Map
2.0   Geotechnical Infiltration Test Report
3.0   Source Control Matrix
4.0   SWQDf & Q100 Volume Map
5.0   Overall SWQDv Infiltration Map
6.0   Inclusions
        1.   BMP Location Site Plan
        2.   Infiltration Trench Calculations
        3.   Full Capture Runoff and SWQDf Design Calculations
        4.   Bio-Clean Full Capture Filter Details
        5.   Contech CDS Unit Details
        6.   FEMA FIRMette

I.   INTRODUCTION

The proposed development site is located at 4416 Azusa Canyon Road in the city of Irwindale, California. (See Vicinity Map in Appendix 1). The site is currently fully developed with an existing Pepsi bottling and distribution facility with related trucking and office facilities. The project proposes to raise the existing onsite facilities and construct one new, approximately 125,470 square foot commercial/industrial building. In addition to the commercial/industrial building, the project proposes to construct paved drive lanes with associated paved parking and adjacent ornamental landscape areas. The overall property size is approximately 5.82 acres of which will encompass the new development.

I.1 PROJECT APPLICABILITY

This project falls under the jurisdiction of the City of Irwindale, located in the County of Los Angeles. The County of Los Angeles Low Impact Development Standards Manual (LID), 2014 methodology and calculations are adopted by the City of Irwindale. This manual was prepared to meet the requirements contained in Los Angeles Regional Water Quality Control Boards Municipal County of Los Angeles Separate Storm Sewer Systems 2012 (MS4) Permit (Order R4-2012-0175).

As a new development project that will create more than 5,000 square feet of new Impervious surface with more than 25 new parking spaces, it is subject to implementation of post-construction stormwater management control measures that infiltrate/treat the Stormwater Quality Design Volume (SWQDv) part of the proposed development.

Per the LID, all projects that are required meet these measures address the following requirements:

- Conduct a site assessment and identify design considerations, including determining the feasibility of on-site infiltration.
- Apply site design principles and techniques.
- Apply source control measures.
- Implement alternative compliance measures, (Not required for this project).
- Apply treatment control measures.
- Implement hydromodification requirements.
- Develop a Maintenance Plan.

It is the intention of this report to provide detailed information to address all of the applicable requirements identified above.

II.   PROJECT SITE ASSESSMENT

The property is 5.82 acres in size and contains an existing Pepsi bottling and distribution facility with related trucking, vehicle parking and office facilities. The site is located on the northeast corner of Azusa Canyon Road and Los Angeles Street.

Currently, the site drains to the south via surface and subsurface drainage facilities to an existing City maintained 36" diameter storm drain system located in Los Angeles Street.

The proposed development will demolish all existing site facilities and reconstructing one new approximately 125,470 square foot commercial/industrial building. The existing drainage pattern will be maintained in the proposed development with the runoff being ultimately directed into the existing 36" diameter drain through new onsite surface and subsurface storm drain facilities. These new facilities are reflected in the Civil construction documents for the project. The new developed drainage areas are reflected on the attached SWQDv & Q100 Volume Map contained in Appendix 4.

No Environmentally Sensitive Areas exist within the proposed development site.

To determine the feasibility of the use of stormwater infiltration BMP's, a Report of Percolation Testing has been performed on the proposed development by Southern California Geotechnical. The results of this report indicate that infiltration of stormwater is feasible and recommended for this project. The percolation test performed on the property indicate that a tested percolation rate of 10 inches per hour with an applied factor of safety of 3 for a final design rate of 3.33 which has been used in the analysis and works well for the infiltration BMP's proposed for this project. The Report of Percolation Testing is included in Appendix 2.

III. APPLY SITE DESIGN PRINCIPLES AND TECHNIQUES

Site design principles and techniques were used early in the development of this project.

The Project Architect, working with the City of Irwindale Planning Department in the entitlement stage of this project, developed a site plan that not only satisfied the Clients need for more staff parking but also provided the minimum amount of impermeable coverage possible while maximizing the amount of permeable landscape area.

Once the site plan was finalized and the determination was made that a portion of the site was suitable for the use of Infiltration BMP's, it was up to the Design Civil Engineer to develop an efficient and environmentally sound drainage design for the proposed development.

With the favorable infiltration rates established by the Project Soils Engineer, we were able to retain the total SWQDv onsite and infiltrate it into the subsurface soils. This will be done through the utilization of infiltration BMP's.

Pollutants of Concerns (POC) for parking lot developments consist of the following:

- Bacteria, Nutrients, Metals, Pesticides, Toxic Organic Compounds, Sediments, Trash & Debris, Oil and Grease.

This project proposes to mitigate these POC's to the most practicable extend possible by the use of a treatment train design of Source and Treatment Control, Permanente Best Management Practices (BMP's) .

For pre-treatment of stormwater runoff, the design will utilize Bio-Clean In-Catch Basin Filters and Contech Engineered Solutions LLC, CDS separator units for project. The catch basin filters are installed in the proposed onsite curb opening and grated type catch basins and are capable of treating the SWQDf and Full Capture requirement for flows from a 1-year, 1-hour storm event as directed in the California State Water Resources Control Board Trash Policy, Resolution No. 2015-0019. Once the initial flows pass through the filters, the total site flows are directed into the proposed CDS units prior to being directed to the infiltration trenches. The final treatment of the SQVD will be accomplished with the proposed Infiltration Trench facilities. This treatment train system exists in both drainage areas of the site. These BMP's are explained in depth with supporting calculations later in this report.

To verify the performance of the infiltration system, follow up field infiltration tests are to be performed by the geotechnical engineer after grading at a minimum of 2 locations within the excavated infiltration trench locations to verify that the infiltration area has been properly protected and performs as designed. Test results are to be provided to the G4 Group Inc. for verification of design prior to construction of the facility.

Rexford Industrial Building, Azusa, CA 3

IV.   APPLY SOURCE CONTROL MEASURES

Source control measures are designed to prevent pollutants from contacting stormwater runoff or prevent discharge of contaminated stormwater runoff to the storm drain system and/or receiving water.

The following is a list of potential site specific source control measures that could be used for the proposed parking lot expansion, based on the matrix contained in Appendix 3.

STORM DRAIN MESSAGE AND SIGNAGE:

- Yes, all proposed storm drain inlets will be provided with appropriate No Dumping signage.

OUTDOOR MATERIAL STORAGE AREA DESIGN:

- No, outdoor material storage is not proposed as part of this development.

OUTDOOR TRASH STORAGE AREA DESIGN:

- Yes, appropriate outdoor trash storage and waste areas are designed as part of this development.

OUTDOOR LOADING/UNLOADING DOCK AREA DESIGN:

- Yes, appropriate loading docks are designed as part of this development.

OUTDOOR REPAIR/MAINTENANCE BAY DESIGN:

- No, vehicle equipment repair or maintenance is not proposed as part of this development.

OUTDOOR VEHICLE/EQUIPMENT/ACCESSORY WASH AREA:

- No, vehicle washing is not proposed as part of this development.

FUELING AREA DESIGN:

- No, fuel dispensing is not proposed as part of this development.

PROOF OF CONTROL MEASURE MAINTENANCE:

- Yes, the onsite stormwater control measures will be maintained by the property owner as outlined in the separate Operations & Maintenance document.

V.   IMPLEMENT ALTERNATIVE COMPLIANCE MEASURES

This project will infiltrate the total SWQDv onsite, consequently Alternative Compliance Measures are not required.

VI.   APPLY TREATMENT CONTROL MEASURES

The primary SWQDv BMP treatment controls for this project consist of two infiltration trenches, one located at the westerly drainage subarea confluence and the other at the southerly confluence. Per the drainage report prepared by NextGen Engineering dated October 1, 2021, the total site SWQDv is 18,511 cubic feet. This is divided by the two drainage areas of the site with 8,941 cubic feet being directed to the westerly outlet and 9,570 cubic feet draining to the southerly outlet. The proposed westerly trench is 30 foot wide by 121 by 6 foot deep with the southerly trench being 35 foot wide by 116 foot long by 6 foot deep Infiltration trench.

Pretreatment of the SWQDv is accomplished in each catch basin with the installation of Bio-Clean filter inserts and CDS units. The breakdown of the required SWQDf and SWQDv is reflected on the attached maps in Appendix 4.0 and 5.0 respectively.

The following table lists the required and provided treatment capacities of the proposed development BMP's.

Proposed Development Infiltration BMP			
Treatment Control BMP	Required SWQDv	Provided SWQDv	Provided Infiltration Area
Western Infiltration Trench	8,941 cubic feet	13,044 cubic feet	3,725 square feet
Southern Infiltration Trench	9,570 cubic feet	13,377 cubic feet	4,060 square feet

Pretreatment BMP Check by Largest Drainage Subarea		
Treatment Control BMP	Required SWQDf (cfs) by Drainage Area	Provided Treatment Design Capacity (cfs) per
Bio-Clean Filter Inserts Model BC-CURB-FC-24	Area B-6 – 0.469 cfs	2.85 cfs
Bio-Clean Filter Inserts Model BC-GRATE-FC-48-48-18	Area A-4 – 0.352 cfs	2.85 cfs
CDS Unit Model BC-GRATE-FC-12-12-12-	Area A – 0.82 cfs	1.1 cfs

Detailed calculations for the proposed BMP's are located in Appendix 6.3

Rexford Industrial Building, Azusa, CA   5

VII.   IMPLEMENT HYDROMODIFICATION REQUIREMENTS

To reduce the impact of development on existing downstream storm drain infrastructure, the City of Irwindale and the County of Los Angeles require new developments to restrict flows leaving the site such that the 100-year proposed peak flow does not exceed the existing 50-year peak flow condition. This project proposes to accomplish this requirement with the installation of stormwater detention storage galleys located in each of the two drainage areas that will restrict the developed flows to below the existing 50-year peaks flows.

Calculations and discussion for the proposed stormwater detention system are contained under separate cover in the Hydrology and Detention Storage Report prepared by NextGen Engineering report dated October 1, 2021. Design details for the proposed detentions systems are contained in the project civil engineering construction documents.

## VIII.   DEVELOP AN OPERATION & MAINTENANCE PLAN

A separate Operation & Maintenance Plan has been prepared for the project, a copy of which shall remain onsite and in the possession of the designated responsible maintenance individual.

IX.   100 YEAR FLOOD PROTECTION

The finished floor elevations for the proposed building will be set at 4.2' above the existing top of curb elevation at the existing catch basin at the northeast corner Azusa Canyon Road and Los Angeles Street.

The Federal Emergency Management Agency (FEMA) has identified this property to be in the unshaded flood Zone X, defined as, "Area of Minimal Flood Hazard" and determined it to be outside the 0.2% annual floodplain with no base flood elevation established. Since the project is outside of the 0.2% or 500 year floodplain, by default the project is outside the 100-year floodplain. A copy of the FIRMette Map for this property is contained in Appendix 6.6

SITE

4416 Azusa Canyon Road
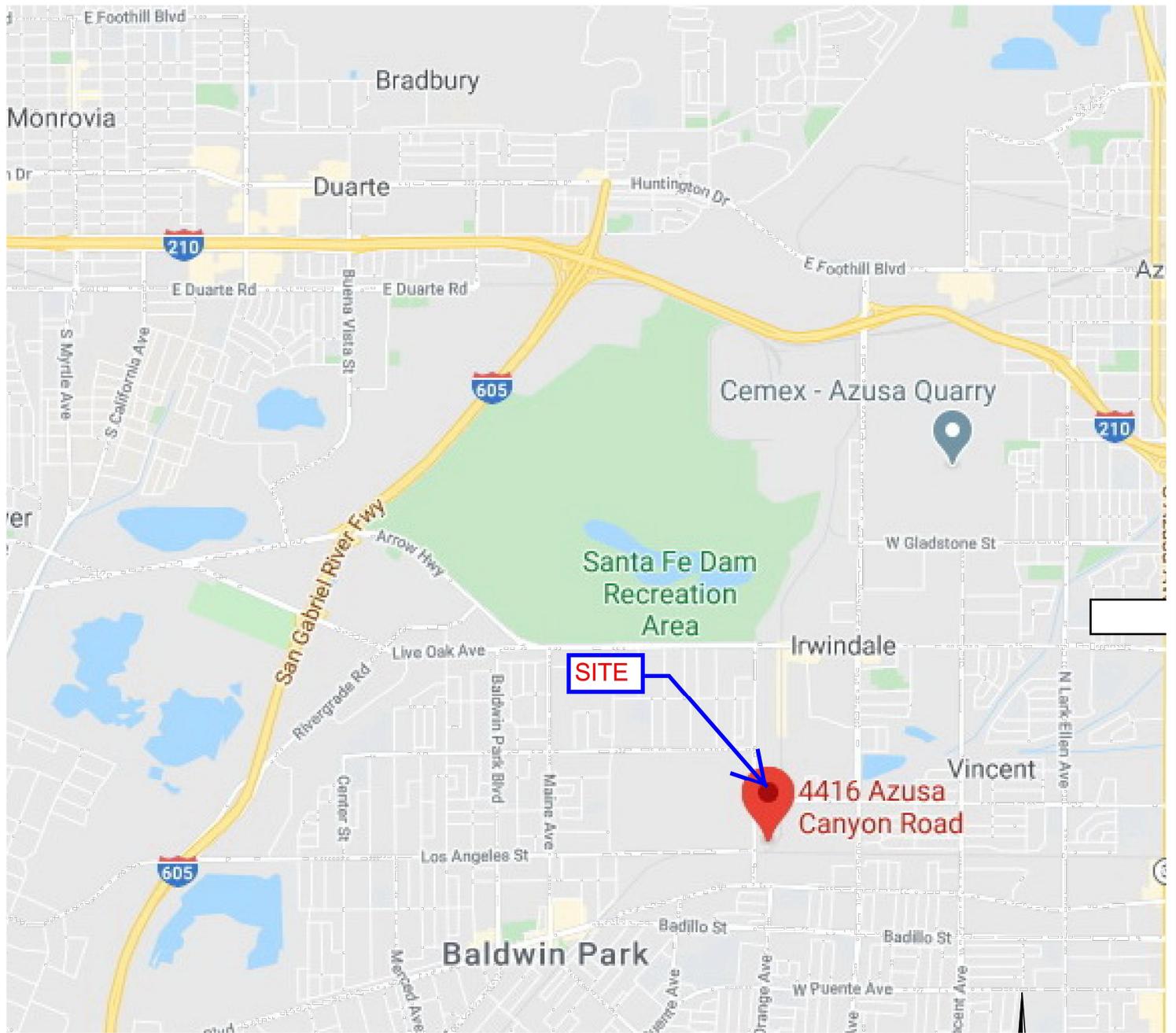
NOT TO SCALE

VICINITY MAP

4416 AZUSA CANYON ROAD
IRWINDALE, CA

F-9

APPENDIX 2

February 13, 2020

Rexford Industrial
11620 Wilshire Boulevard, 10th floor
Los Angeles, California 90025

Attention: Mr. Ricardo Rivas
Construction Manager

Project No.: **20G105-2**

Subject: **Results of Infiltration Testing**
Proposed Warehouse - Infiltration
4416 Azusa Canyon Road
Irwindale, California

Reference: Geotechnical Investigation and Infiltration Testing, Proposed Warehouse, 4416 Azusa Canyon Road, Irwindale, California, prepared for Rexford Industrial, by Southern California Geotechnical, Inc. (SCG), SCG Project No. 20G105-1.

Dear Mr. Rivas:

In accordance with your request, we have conducted infiltration testing at the subject site. We are pleased to present this report summarizing the results of the infiltration testing and our design recommendations.

Scope of Services

The scope of services performed for this project was in general accordance with our Proposal No. 19P370, dated September 25, 2019. The scope of services included site reconnaissance, subsurface exploration, obtaining representative soil samples, laboratory testing, review of relevant geological literature, analysis to determine the infiltration rates of the onsite soils, and preparation of a geotechnical report documenting our findings. The infiltration testing was performed in general accordance with ASTM Test Method D-3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometer.

Site and Project Description

The subject site is located at the northeast corner of Azusa Canyon Road and Los Angeles Street in Irwindale, California. The site is bounded to the north by the Big Dalton Wash, to the west by Azusa Canyon Road, to the south by Los Angeles Street, and to the southeast and east by an existing railroad easement. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of an irregular-shaped parcel, 5.89± acres in size. The site is presently developed with one warehouse, 64,535± ft² in size, in the western half of the site. The warehouse is currently occupied by Pepsi Bottling Group. The building is a single-story structure of concrete tilt-up construction and is assumed to be supported on conventional shallow foundations with a concrete slab-on-grade floor. A loading dock is located along a portion of the northeast building

wall. A modular building, about 1,000 ± ft² in size is present in the east-central portion of the site. This modular building appears to be supported directly on the pavements. The buildings are surrounded by asphaltic concrete pavements in the parking and drive areas, Portland cement concrete pavements in the loading dock areas, and concrete flatwork in limited areas throughout the site. The southeastern area of the site is vacant and undeveloped. The ground surface cover in this area consists of exposed soil with moderate to extensive native grass and weed growth.

Detailed topographic information was not available at the time of this report. Based on visual observations made at the time of the subsurface investigation and from elevation data obtained from Google Earth, the overall site topography generally slopes downward to the southwest at a gradient of 1 to 2± percent.

Proposed Development

A site plan, prepared by GAA Architects, has been provided to our office by the client. Based on this plan, a new warehouse, 130,540± ft² in size, will be constructed in the central area of the site. Dock-high doors will be constructed along a portion of the south building wall. The building will be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the loading dock areas, concrete flatwork and landscape planters throughout.

Detailed structural information has not been provided. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundations 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 100 kips and 3 to 5 kips per linear foot, respectively.

Grading plans for the proposed development were not available at the time of this report. 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 topography, and assuming a relatively balanced site, cuts and fills of 2 to 3± feet are expected to be necessary to achieve the proposed site grades.

Concurrent Study

Southern California Geotechnical, Inc. (SCG) concurrently conducted a geotechnical investigation at the subject site. As part of this study, four (4) trenches were excavated to depths of 6 to 9± feet below the existing site grades. Artificial fill soils were encountered beneath the pavements at all trench locations, extending to depths of 3 to 7½± feet below the existing site grades. At Trench Nos. T-1 and T-4, the fill soils contain occasional clay nodules. The artificial fill soils generally consist of medium dense fine sands, silty sands with varying fine to coarse gravel content and occasional Cobbles. The fill soils possess a disturbed appearance and some samples contain debris, such as glass fragments, resulting in their classification as artificial fill. Native alluvial soils were encountered at all of the trench locations, extending to at least the maximum depth explored of 9± feet below the existing site grades. The native alluvial soils generally consist of gravelly well-graded sands, with some cobbles and occasional boulders.

Groundwater

Free water was not encountered during the drilling of any of the trench locations. Based on the lack of any water within the trenches, and the moisture contents of the recovered soil samples, the static groundwater is considered to have existed at a depth in excess of 10± feet at the time of the subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well in this database is located approximately 300 feet West of the site. Water level readings within this monitoring well indicate groundwater levels of 194± feet below the ground surface in April 2017.

Subsurface Exploration

Scope of Exploration

The subsurface exploration conducted for this project consisted of two (2) backhoe-excavated infiltration trenches to depths of 9 to 10± feet below existing site grades. The trenches were logged during excavation by a member of our staff. The approximate locations of the infiltration trenches (identified as I-1 and I-2) are included in this report as Plate 2.

Geotechnical Conditions

Artificial fill soils were encountered at the two (2) infiltration test locations and extend to depths of 4 to 6± feet. At Infiltration No. I-1, the fill extends to 6 feet below the existing site grades. The fill soils at this location consist of loose and dry silty fine sands with trace to occasional medium to coarse sand and trace gravel. These soils are underlain by a 1-foot-thick soft and damp silty clay layer between 3 and 4 feet below ground surface. At Infiltration No. I-2, the fill consists of loose and damp silty fine sand with trace gravel. At 1½± feet, little to some soft and damp clay was encountered to the maximum fill depth of 4± feet. The artificial fill soils possess a disturbed appearance and metal fragments were observed within the fill at Infiltration No. I-1.

Native alluvium was encountered beneath the artificial fill soils at all of the infiltration locations, extending to at least the maximum depth explored of 10± feet below existing site grades. The alluvial soils beneath the artificial fill consist of loose and damp gravelly fine to coarse sand with some cobble content at both infiltration test locations. At Infiltration Trench No. I-2, cobble content varies within the alluvium, with extensive cobble content between 5 and 7 feet, and occasional cobbles between 9½ and 10± feet. The Trench Logs, which illustrate the conditions encountered at the infiltration test locations, are presented on plates B-1 and B-2 of this report.

Infiltration Testing

We understand that the results of the testing will be used to prepare a preliminary design for the storm water infiltration systems that will be used at the subject site. As previously mentioned, the infiltration testing was performed in general accordance with ASTM Test Method D-3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometer.

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

F-13

Proposed Warehouse - Infiltration – Irwindale, CA
Project No. 20G105-2
Page 3

Two stainless steel infiltration rings were used for the infiltration testing. The outer infiltration ring is 2 feet in diameter and 20 inches in height. The inner infiltration ring is 1 foot in diameter and 20 inches in height. At each test location, a trench was excavated to the proposed depth of the infiltration system and the outer ring was driven 3± inches into the soil at the base of each trench. The inner ring was centered inside the outer ring and subsequently driven 3± inches into the soil at the base of the trench. The rings were driven into the soil using a ten-pound sledge hammer. The soil surrounding the wall of the infiltration rings was only slightly disturbed during the driving process.

Infiltration Testing Procedure

Infiltration testing was performed at both of the infiltration trench locations. The infiltration testing consisted of filling the inner ring and the annular space (the space between the inner and outer rings) with water, approximately 3 to 4 inches above the soil. To prevent the flow of water from one ring to the other, the water level in both the inner ring and the annular space between the rings was maintained using constant-head float valves. The volume of water that was added to maintain a constant head in the inner ring and the annular space during each time interval was determined and recorded. A cap was placed over the rings to minimize the evaporation of water during the tests.

The schedule for readings was determined based on the observed soil type at the base of each backhoe-excavated trench. Based on the existing soils at the trench locations, the volumetric measurements were made at 1-minute increments at I-1, and 4-minute increments at I-2. The water volume measurements are presented on the spreadsheets enclosed with this report. The infiltration rates for each of the timed intervals are also tabulated on these spreadsheets.

The infiltration rates for the infiltration tests are calculated in centimeters per hour and then converted to inches per hour. The rates are summarized below:

Infiltration Test No.	Depth (feet)	Soil Description	Infiltration Rate (inches/hour)
I-1	9	Light Gray Sandy fine to coarse Gravel, some Cobble content, trace Silt	19.4
I-2	10	Light Gray Gravelly fine to coarse Sand, occasional Cobble content, trace Silt	10.5

Laboratory Testing

Moisture Content

The moisture contents for selected soil samples within the trenches were determined in accordance with ASTM D-2216 and are expressed as a percentage of the dry weight. These test results are presented on the Trench Logs.

SOUTHERN CALIFORNIA GEOTECHNICAL
SoCalGeo

F-14

Proposed Warehouse - Infiltration – Irwindale, CA
Project No. 20G105-2
Page 4

Grain Size Analysis

The grain size distribution of selected soils collected from the base of each infiltration test trench has been determined using a range of wire mesh screens. These tests were performed in general accordance with ASTM D-422 and/or ASTM D-1140. 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 the grainsize analysis are presented on Plates C-1 and C-2 of this report.

Design Recommendations

Two (2) infiltration tests were performed at the subject site. As noted above, the calculated infiltration rates at the infiltration test locations are 19.1 and 10.5 inches per hour. **Based on the results of Infiltration Test Nos. I-1 and I-2, we recommend an infiltration rate of 10 inches per hour be used for the design of the proposed below-grade chamber system located in the east-central region and for the proposed chamber system located in the southwestern region of the site.**

We recommend that a representative from the geotechnical engineer be on-site during the construction of the proposed infiltration systems to identify the soil classification at the base of each chamber system. It should be confirmed that the soils at the base of the proposed infiltration systems correspond with those presented in this report to ensure that the performance of the systems will be consistent with the rates reported herein.

The design of the proposed storm water infiltration systems should be performed by the project civil engineer, in accordance with the City of Irwindale and/or County of Los Angeles guidelines. However, it is recommended that the systems be constructed so as to facilitate removal of silt and clay, or other deleterious materials from any water that may enter the system. The presence of such materials would decrease the effective infiltration rates. **It is recommended that the project civil engineer apply an appropriate factor of safety. The infiltration rates recommended above are based on the assumption that only clean water will be introduced to the subsurface profile. Any fines, debris, or organic materials could significantly impact the infiltration rates.** It should be noted that the recommended infiltration rates are based on infiltration testing at two (2) discrete locations, and the overall infiltration rates of the storm water infiltration systems could vary considerably.

Construction Considerations

The infiltration rates presented in this report are specific to the tested locations and tested depths. Infiltration rates can be significantly reduced if the soils are exposed to excessive disturbance or compaction during construction. Therefore, the subgrade soils within proposed infiltration system areas should not be over-excavated, undercut or compacted in any significant manner. **It is recommended that a note to this effect be added to the project plans and/or specifications.**

Infiltration versus Permeability

Infiltration rates are based on unsaturated flow. As water is introduced into soils by infiltration, the soils become saturated and the wetting front advances from the unsaturated zone to the

saturated zone. Once the soils become saturated, infiltration rates become zero, and water can only move through soils by hydraulic conductivity at a rate determined by pressure head and soil permeability. The infiltration rates presented herein were determined in accordance with the ASTM Test Method D-3385-03 standard and are considered valid for the time and place of the actual test. Changes in soil moisture content will affect these infiltration rates. Infiltration rates should be expected to decrease until the soils become saturated. Soil permeability values will then govern groundwater movement. Permeability values may be on the order of 10 to 20 times less than infiltration rates. The system designer should incorporate adequate factors of safety and allow for overflow design into appropriate traditional storm drain systems, which would transport storm water off-site.

Location of Infiltration Systems

The use of on-site storm water infiltration systems carries a risk of creating adverse geotechnical conditions. Increasing the moisture content of the soil can cause the soil to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Overlying structures and pavements in the infiltration areas could potentially be damaged due to saturation of subgrade soils. **The proposed infiltration systems for this site should be located at least 25 feet away from any structures, including retaining walls.** Even with this provision of locating the infiltration systems at least 25 feet from the buildings, it is possible that infiltrating water into the subsurface soils could have an adverse effect on the proposed or existing structures. It should also be noted that utility trenches which happen to collect storm water can also serve as conduits to transmit storm water toward the structure, depending on the slope of the utility trench. Therefore, consideration should also be given to the proposed locations of underground utilities which may pass near the proposed infiltration system.

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, structural engineer, and/or civil engineer. The design of the infiltration system is the responsibility of the civil engineer. The role of the geotechnical engineer is limited to determination of infiltration rate only. By using the design infiltration rates contained herein, the civil engineer agrees to indemnify, defend, and hold harmless the geotechnical engineer for all aspects of the design and performance of the infiltration system. 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 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 trench locations and testing 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.

SOUTHERN CALIFORNIA GEOTECHNICAL

SoCalGeo

F-16

Proposed Warehouse - Infiltration – Irwindale, CA
Project No. 20G105-2
Page 6

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.

Closure

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,

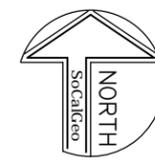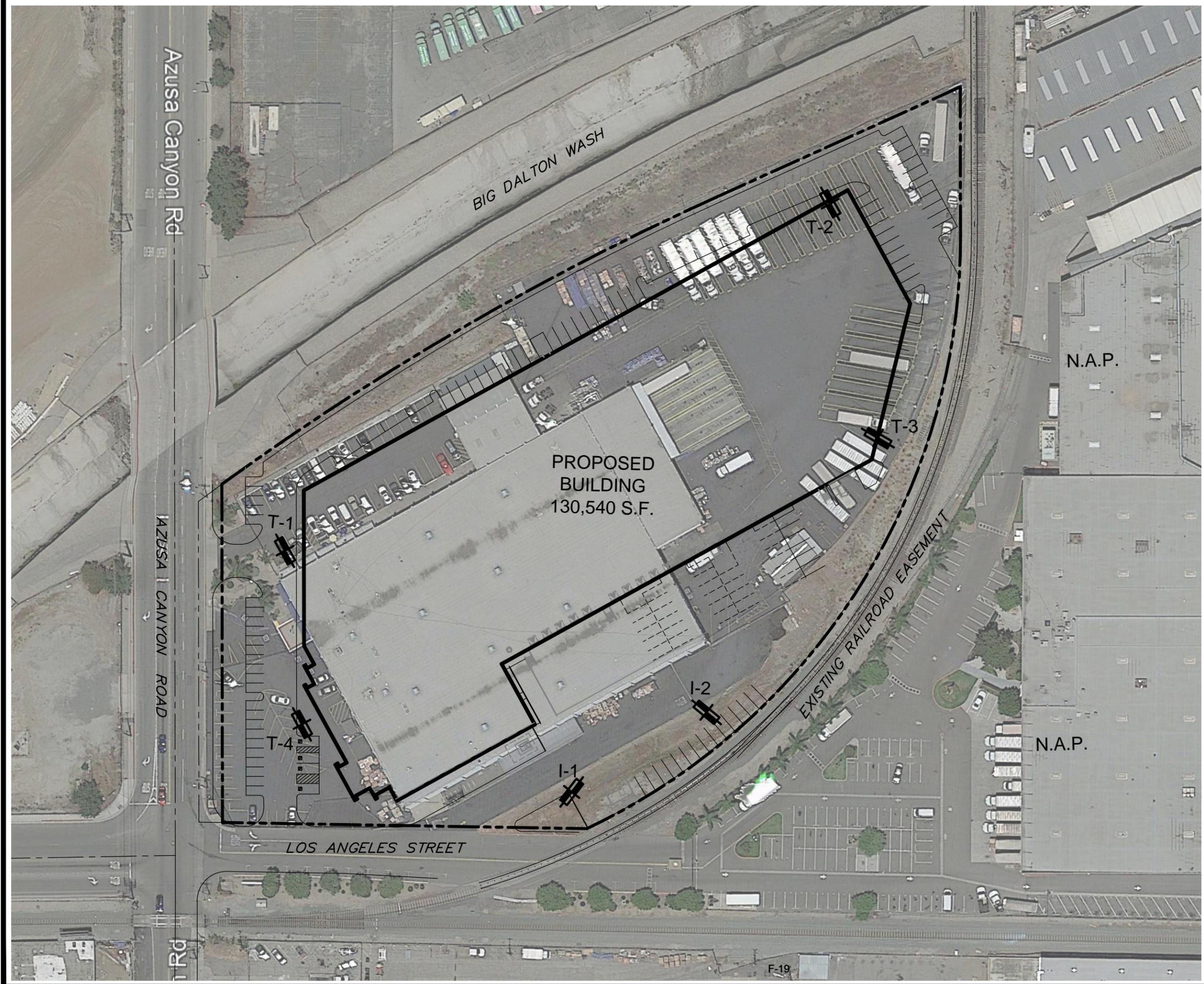SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Ryan Bremer
Staff Geologist

Robert G. Trazo, GE 2655
Principal Engineer

Distribution: (1) Addressee

Enclosures: Plate 1 - Site Location Map
Plate 2 - Infiltration Test Location Plan
Trench Logs (2 pages)
Infiltration Test Results Spreadsheets (2 pages)
Grain Size Distribution Graphs (2 pages)

Irwindale

Vincent

SITE

SOURCE: USGS TOPOGRAPHIC MAP OF THE BALDWIN PARK
QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP		
PROPOSED WAREHOUSE		
IRWINDALE, CALIFORNIA		
SCALE: 1" = 2000'	SoCalGeo	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: RB CHKD: RGT		
SCG PROJECT 20G105-2		
PLATE 1		

F-18

NORTH
SoCalGeo

BIG DALTON WASH

Azusa Canyon Rd

AZUSA CANYON ROAD

T-2

T-3

N.A.P.

PROPOSED
BUILDING
130,540 S.F.

T-1

EXISTING RAILROAD EASEMENT

I-2

T-4

N.A.P.

I-1

LOS ANGELES STREET

Rd

F-19

GEOTECHNICAL LEGEND

APPROXIMATE TRENCH LOCATION

NOTE: CONCEPTUAL SITE PLAN PREPARED BY GAA ARCHITECTS.

TRENCH AND INFILTRATION LOCATION PLAN

PROPOSED WAREHOUSE

IRWINDALE, CALIFORNIA

SCALE: 1" = 80'	
DRAWN: RB	
CHKD: RGT	
SCG PROJECT 20G105-2	
PLATE 2	

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. I-1

JOB NO.: 20G105-2

PROJECT: Proposed Warehouse

LOCATION: Irwindale, CA

DATE: 1/31/20

EQUIPMENT USED: Backhoe

LOGGED BY: Ryan Bremer

ORIENTATION: S 55 W

WATER DEPTH: Dry

SEEPAGE DEPTH: Dry

READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel, abundant fine root fibers, mottled, loose-dry
				B: FILL: Dark brown Silty Clay, occasional fine Sand, trace fine root fibers, soft-damp
5				C: FILL: Brown Silty fine Sand, some medium to coarse Sand, some fine root fibers, loose-damp
				D: ALLUVIUM: Light gray Sandy fine to coarse Gravel, some Cobble content, trace Silt, loose-damp
	b		3	
10				Trench Terminated @ 9 feet
15				

GRAPHIC REPRESENTATION

S 55 W

SCALE: 1" = 5'

A

B

Gravel

C

D

Cobble

TRENCH LOG

PLATE B-1

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. I-2

JOB NO.: 20G105-2

PROJECT: Proposed Warehouse

LOCATION: Irwindale, CA

DATE: 1/28/20

EQUIPMENT USED: Backhoe

LOGGED BY: Ryan Bremer

ORIENTATION: N 45 W

WATER DEPTH: Dry

SEEPAGE DEPTH: Dry

READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Brown Silty fine Sand, trace fine to coarse Gravel, trace Cobble content, some fine root fragments, trace metal, loose-damp
				B: FILL: @ 1.5 feet little to some Clay
				C: ALLUVIUM: Light gray Gravelly fine to coarse Sand, some Cobble content, trace Silt, loose-damp
5				@ 5 to 7 feet Cobbly fine to coarse Sand, some fine to coarse Gravel
	b		5	
10	b		4	@ 9.5 feet occasional Cobble content
				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION

Metal

N 45 W

SCALE: 1" = 5'

Cobble

Gravel

Cobble

A

B

C

KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
    (RELATIVELY UNDISTURBED)

TRENCH LOG

F-21

PLATE B-2

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Irwindale, CA
Project Number	20G105-2
Engineer	Ryan Bremer

Infiltration Test No   I-1

Constants			
	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Anlr. Spac	2	2.36	2189

*Note: The infiltration rate was calculated based on current time interval

Test Interval		Time (hr)	Interval **Elapsed** (min)	Flow Readings				Infiltration Rates			
				Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	9:30 AM	0	0	700	0	5500	140.02	366.71	55.13	144.38
	Final	9:31 AM	**1**	700		5500					
2	Initial	9:33 AM	1	0	600	0	5400	49.34	148.02	19.43	58.28
	Final	9:34 AM	**4**	600		5400					
3	Initial	9:37 AM	1	0	600	0	4400	49.34	120.61	19.43	47.48
	Final	9:38 AM	**8**	600		4400					
4	Initial	9:40 AM	1	0	600	0	4600	49.34	126.09	19.43	49.64
	Final	9:41 AM	**11**	600		4600					
5	Initial	9:43 AM	1	0	550	0	4500	45.23	123.35	17.81	48.56
	Final	9:44 AM	**14**	550		4500					
6	Initial	9:46 AM	1	0	600	0	4400	49.34	120.61	19.43	47.48
	Final	9:47 AM	**17**	600		4400					
7	Initial	9:49 AM	1	0	600	0	4500	49.34	123.35	19.43	48.56
	Final	9:50 AM	**20**	600		4500					
8	Initial	9:51 AM	1	0	600	0	4600	49.34	126.09	19.43	49.64
	Final	9:52 AM	**22**	600		4600					
9	Initial	9:54 AM	1	0	550	0	4600	45.23	126.09	17.81	49.64
	Final	9:55 AM	**25**	550		4600					
10	Initial	9:56 AM	1	0	600	0	4400	49.34	120.61	19.43	47.48
	Final	9:57 AM	**27**	600		4400					

INFILTRATION CALCULATIONS

Project Name	Proposed Warehouse
Project Location	Irwindale, CA
Project Number	20G105-2
Engineer	Ryan Bremer

Infiltration Test No   I-2

Constants

	Diameter (ft)	Area (ft^2)	Area (cm^2)
Inner	1	0.79	730
Anlr. Spac	2	2.36	2189

*Note: The infiltration rate was calculated based on current time interval

Test Interval		Time (hr)	Interval **Elapsed** (min)	Inner Ring (ml)	Ring Flow (cm^3)	Annular Ring (ml)	Space Flow (cm^3)	Inner Ring* (cm/hr)	Annular Space* (cm/hr)	Inner Ring* (in/hr)	Annular Space* (in/hr)
1	Initial	9:52 AM	0	0	1050	0	8000	210.03	533.40	82.69	210.00
	Final	9:56 AM	**4**	1050		8000					
2	Initial	9:57 AM	4	0	1200	0	6500	24.67	44.54	9.71	17.54
	Final	10:01 AM	**9**	1200		6500					
3	Initial	10:02 AM	4	0	1300	0	6300	26.73	43.17	10.52	17.00
	Final	10:06 AM	**14**	1300		6300					
4	Initial	10:07 AM	4	0	1300	0	6400	26.73	43.86	10.52	17.27
	Final	10:11 AM	**19**	1300		6400					
5	Initial	10:12 AM	4	0	1300	0	6300	26.73	43.17	10.52	17.00
	Final	10:16 AM	**24**	1300		6300					
6	Initial	10:17 AM	4	0	1300	0	6500	26.73	44.54	10.52	17.54
	Final	10:21 AM	**29**	1300		6500					
7	Initial	10:22 AM	4	0	1300	0	6500	26.73	44.54	10.52	17.54
	Final	10:26 AM	**34**	1300		6500					
8	Initial	10:28 AM	4	0	1300	0	6400	26.73	43.86	10.52	17.27
	Final	10:32 AM	**40**	1300		6400					
9	Initial	10:33 AM	4	0	1350	0	6400	27.75	43.86	10.93	17.27
	Final	10:37 AM	**45**	1350		6400					
10	Initial	10:39 AM	4	0	1300	0	6500	26.73	44.54	10.52	17.54
	Final	10:43 AM	**51**	1300		6500					

Grain Size Distribution

Sieve Analysis	Hydrometer Analysis
US Standard Sieve Sizes	

US Standard Sieve Sizes (top axis): 2   1   3/4   1/2   3/8   1/4   #4   #8 #10   #16 #20 #30 #40   #50   #100   #200

Percent Passing by Weight (y-axis): 100, 90, 80, 70, 60, 50, 40, 30, 20, 10, 0

Grain Size in Millimeters (x-axis): 100, 10, 1, 0.1, 0.01, 0.001

Coarse Gravel	Fine Gravel	Crs. Sand	Med. Sand	Fine Sand	Fines (Silt and Clay)
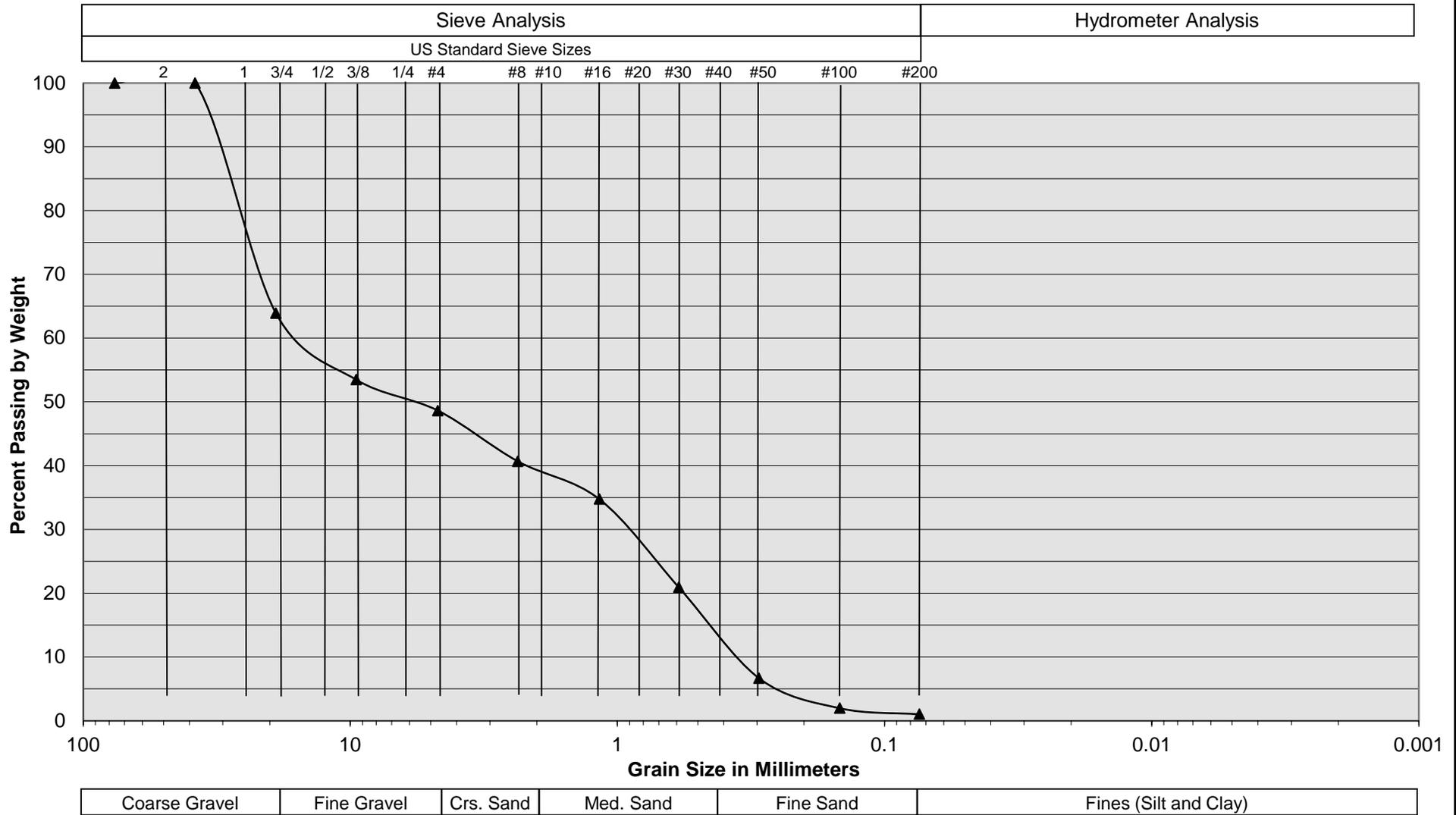
Sample Description	I-1 @ 9 feet
Soil Classification	Light Gray Sandy fine to coarse Gravel, occasional coarse Sand, trace Silt

Proposed Warehouse
Irwindale, California
Project No. 20G105-2
PLATE C-1

F-24

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Grain Size Distribution

Sieve Analysis	Hydrometer Analysis

US Standard Sieve Sizes

Sieve labels: 2 1 3/4 1/2 3/8 1/4 #4 #8 #10 #16 #20 #30 #40 #50 #100 #200

Percent Passing by Weight (y-axis: 0 to 100)

Grain Size in Millimeters (x-axis: 100 to 0.001)

Coarse Gravel	Fine Gravel	Crs. Sand	Med. Sand	Fine Sand	Fines (Silt and Clay)
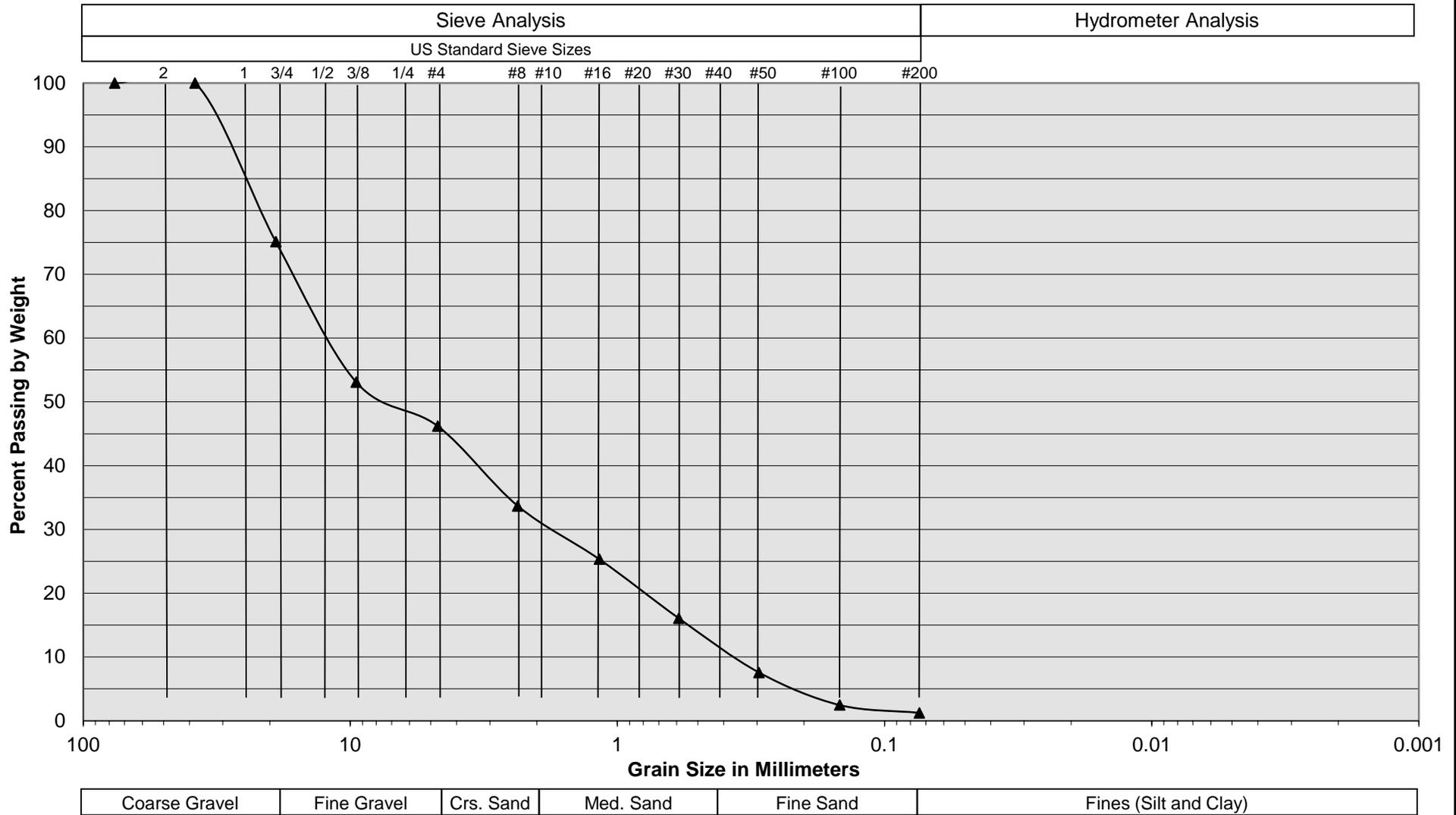
Sample Description	I-2 @ 9.5 feet
Soil Classification	Light Gray Sandy fine to coarse Gravel, occasional fine Sand

Proposed Warehouse
Irwindale, California
Project No. 20G105-2
PLATE C-2

F-25

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

APPENDIX 3

Low Impact Development Standards Manual

Table 5-1. Source Control Measures Selection Matrix

Project Type	Storm Drain Message and Signage (S-1)	Outdoor Material Storage Area (S-2)	Outdoor Trash Storage/Waste Handling Area (S-3)	Outdoor Loading/Unloading Dock Area (S-4)	Outdoor Vehicle/Equipment Repair/Maintenance Area (S-5)	Outdoor Vehicle/Equipment Accessory Wash Area (S-6)	Fuel & Maintenance Area (S-7)	Landscape Irrigation Practices (S-8)	Building Materials (S-9)	Animal Care and Handling Facilities (S-10)	Outdoor Horticulture Areas (S-11)
Designated Projects – New Development											
Development ≥1 acre and ≥10,000 ft² new impervious area	R	R[1]	R[1]	R[1]	R[1]	R[1]	R[1]	R	R	R[1]	R[1]
Industrial parks (≥10,000 ft²)	R	R[1]	R[1]	R[1]	R[1]	R[1]	R[1]	R	R	–	–
Commercial malls (≥10,000 ft²)	R	R[1]	R[1]	R[1]	R[1]	R[1]	–	R	R	R[1]	R[1]
Retail gasoline outlets (≥5,000 ft²)	R	R[1]	R[1]	R[1]	R[1]	R[1]	R[1]	R	R	–	–
Restaurants (≥5,000 ft²)	R	R[1]	R[1]	R[1]	–	–	–	R	R	–	–
Parking lots (≥5,000 ft² or ≥25 parking spaces)	R	R[1]	R[1]	R[1]	R[1]	R[1]	R[1]	R	R	R[1]	R[1]
Automotive service facilities (5,000 ft²)	R	R[1]	R[1]	R[1]	R[1]	R[1]	R[1]	R	R	–	–
Projects in/around Significant Ecologic Areas	R	R[1]	R[1]	R[1]	R[1]	R[1]	R[1]	R	R	R[1]	R[1]
Projects potentially impacting sensitive biological species or habitats	R	R[1]	R[1]	R[1]	R[1]	R[1]	R[1]	R	R	R[1]	R[1]
Projects adding ≥2,500 ft² of impervious area	R	R[1]	R[1]	R[1]	R[1]	R[1]	R[1]	R	R	R[1]	R[1]

R = required; R[1] = required if outdoor activity area is included in project; R[2] = required for multi-family dwellings
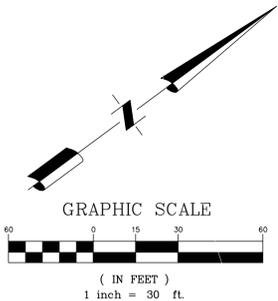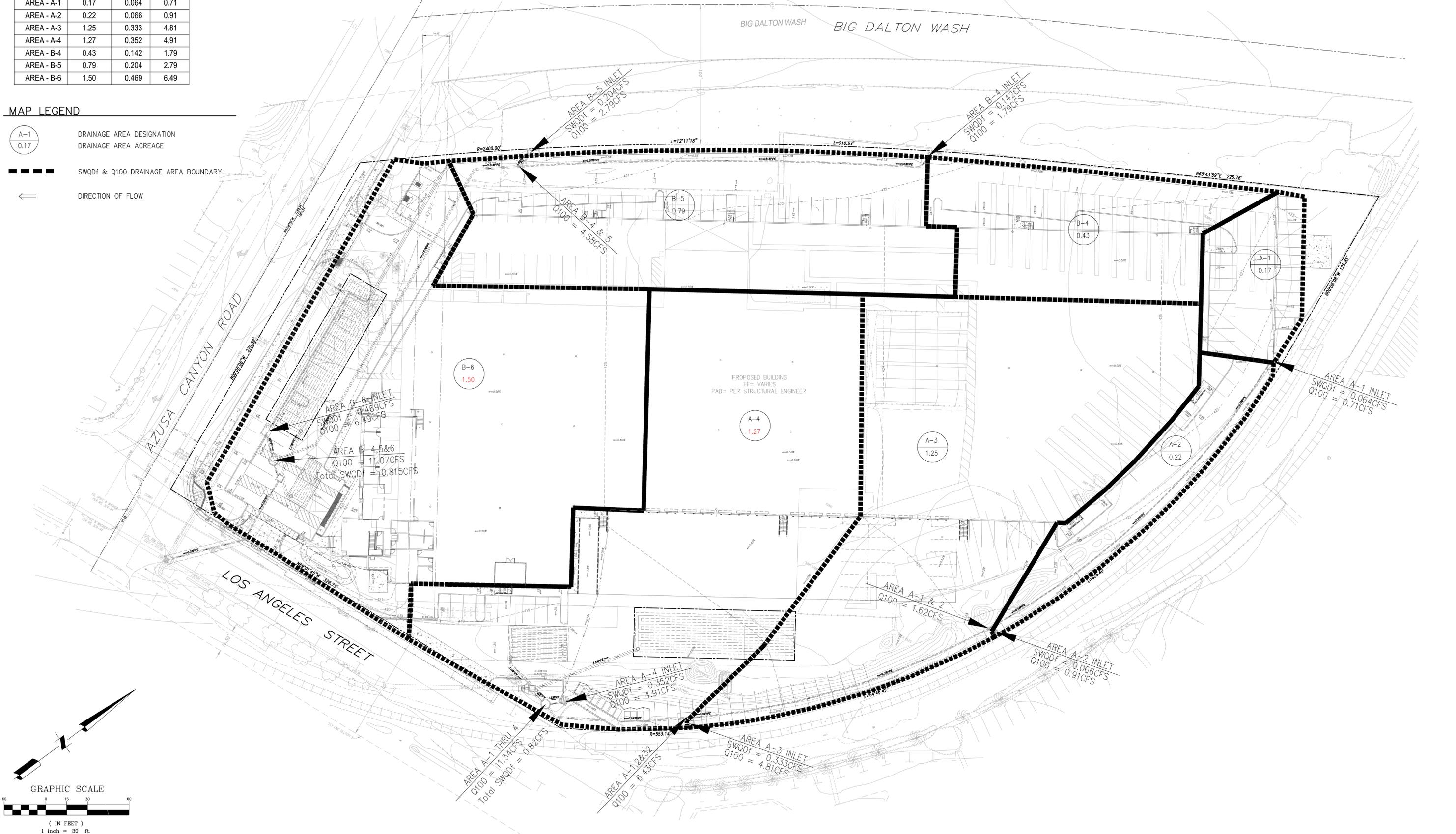
County of Los Angeles

February 2014

APPENDIX 4

SWQDf AND Q100 SUBAREA TABULATION

SUBAREA	ACRES (Ac)	SWQDf (cfs)	Q100 (cfs)
AREA - A-1	0.17	0.064	0.71
AREA - A-2	0.22	0.066	0.91
AREA - A-3	1.25	0.333	4.81
AREA - A-4	1.27	0.352	4.91
AREA - B-4	0.43	0.142	1.79
AREA - B-5	0.79	0.204	2.79
AREA - B-6	1.50	0.469	6.49

MAP LEGEND

A-1 / 0.17   DRAINAGE AREA DESIGNATION
              DRAINAGE AREA ACREAGE

– – – – –   SWQDf & Q100 DRAINAGE AREA BOUNDARY

⟵   DIRECTION OF FLOW

BIG DALTON WASH   BIG DALTON WASH

AREA B-5 INLET
SWQDf = 0.204CFS
Q100 = 2.79CFS

AREA B-4 INLET
SWQDT = 0.142CFS
Q100 = 1.79CFS

B=2400.00'   L=12'11'18"   L=510.54'   N65°43'59"E 225.76'

AREA B-4 & 5
Q100 = 4.58CFS

B-5 / 0.79

B-4 / 0.43

A-1 / 0.17

AREA B-6 INLET
SWQDf = 0.469CFS
Q100 = 6.49CFS

B-6 / 1.50

PROPOSED BUILDING
FF= VARIES
PAD= PER STRUCTURAL ENGINEER

AREA A-1 INLET
SWQDf = 0.064CFS
Q100 = 0.71CFS

AREA B- 4,5&6
Q100 = 11.07CFS
Total SWQDf = 0.815CFS

A-4 / 1.27

A-3 / 1.25

A-2 / 0.22

AZUSA CANYON ROAD

LOS ANGELES STREET

AREA A-1 & 2
Q100 = 1.62CFS

AREA A-2 INLET
SWQDf = 0.066CFS
Q100 = 0.91CFS

AREA A-1 THRU 4
Q100 = 11.34CFS
Total SWQDf = 0.82CFS

AREA A-4 INLET
SWQDf = 0.352CFS
Q100 = 4.91CFS

AREA A-1,2&32
Q100 = 6.43CFS

R=553.74'

AREA A-3 INLET
SWQDf = 0.333CFS
Q100 = 4.81CFS

GRAPHIC SCALE

(IN FEET)
1 inch = 30 ft.

CIVIL ENGINEER:

REGISTERED PROFESSIONAL ENGINEER
MARK ROGERS
No. 42848
Exp. 03/22
CIVIL
STATE OF CALIFORNIA

42848
MARK ROGERS   R.C.E. NO.   DATE

PREPARED BY:
THE G4 GROUP INC.
Consulting Engineers
5301 NORTH COMMERCE AVE., SUITE H
MOORPARK, CA 93021
PHONE: (805) 523-0010 FAX: (805) 553-0626

PREPARED FOR:
REXFORD INDUSTRIAL
REALTY, INC.
11620 WILSHIRE BLVD., 10TH FLR.
LOS ANGELES, CA
CONTACT: MARK SALEH
424−281−7361
F-29

DESCRIPTION OF REVISION	DATE	NO.

SWQDf AND Q100 VOLUME MAP

4416 AZUSA CANYON ROAD, IRWINDALE

PORTION OF LOTS 9 AND 10, ORANGE BELT TRACT, MR 37−67
IN THE CITY OF IRWINDALE, COUNTY OF LOS ANGELES, STATE OF CALIFORNIA

1 of 1

JOB NO. 1946
OCTOBER 4, 2021

APPENDIX 5

BIG DALTON WASH   BIG DALTON WASH

31' WIDE x 121' LONG INFILTRATION TRENCH
3,630 SQFT BASIN FOOTPRINT AREA
SWQDv 13,043 CF PROVIDED

OVERALL INFILTRATION
SUBAREA B
REQUIRED SWQDv = 8,685 C.F.
2.72 ACRES

OVERALL AREA B
SQDV = 8,941CF

PROPOSED BUILDING
FF= VARIES
PAD= PER STRUCTURAL ENGINEER

OVERALL INFILTRATION SUBAREA A
REQUIRED SWQDv = 9,255 C.F.
2.91 ACRES

35' WIDE x 116' LONG INFILTRATION TRENCH
4,060 SQFT BASIN FOOTPRINT AREA
SWQDv 13,377 CF PROVIDED

AZUSA CANYON ROAD

LOS ANGELES STREET

OVERALL AREA A
SQDV = 9,570 CF

GRAPHIC SCALE
(IN FEET)
1 inch = 30 ft.

CIVIL ENGINEER:

MARK ROGERS   R.C.E. NO.   DATE
42848

REGISTERED PROFESSIONAL ENGINEER
MARK ROGERS
No. 42848
Exp. 03/22
CIVIL
STATE OF CALIFORNIA

PREPARED BY:
THE G4 GROUP INC.
Consulting Engineers
5301 NORTH COMMERCE AVE., SUITE H
MOORPARK, CA 93021
PHONE: (805) 523-0010 FAX: (805) 553-0626

PREPARED FOR:
REXFORD INDUSTRIAL
REALTY, INC.
11620 WILSHIRE BLVD., 10TH FLR.
LOS ANGELES, CA
CONTACT: MARK SALEH
424-281-7361
F-31

DESCRIPTION OF REVISION	DATE	NO.

OVERALL SWQDv INFILTRATION MAP
4416 AZUSA CANYON ROAD, IRWINDALE
PORTION OF LOTS 9 AND 10, ORANGE BELT TRACT, MR 37-67
IN THE CITY OF IRWINDALE, COUNTY OF LOS ANGELES, STATE OF CALIFORNIA

1 of 1

JOB NO. 1946
OCTOBER 4, 2021

APPENDIX 6

Appendix Cover Sheet.doc

APPENDIX 6.1

Appendix Cover Sheet.doc
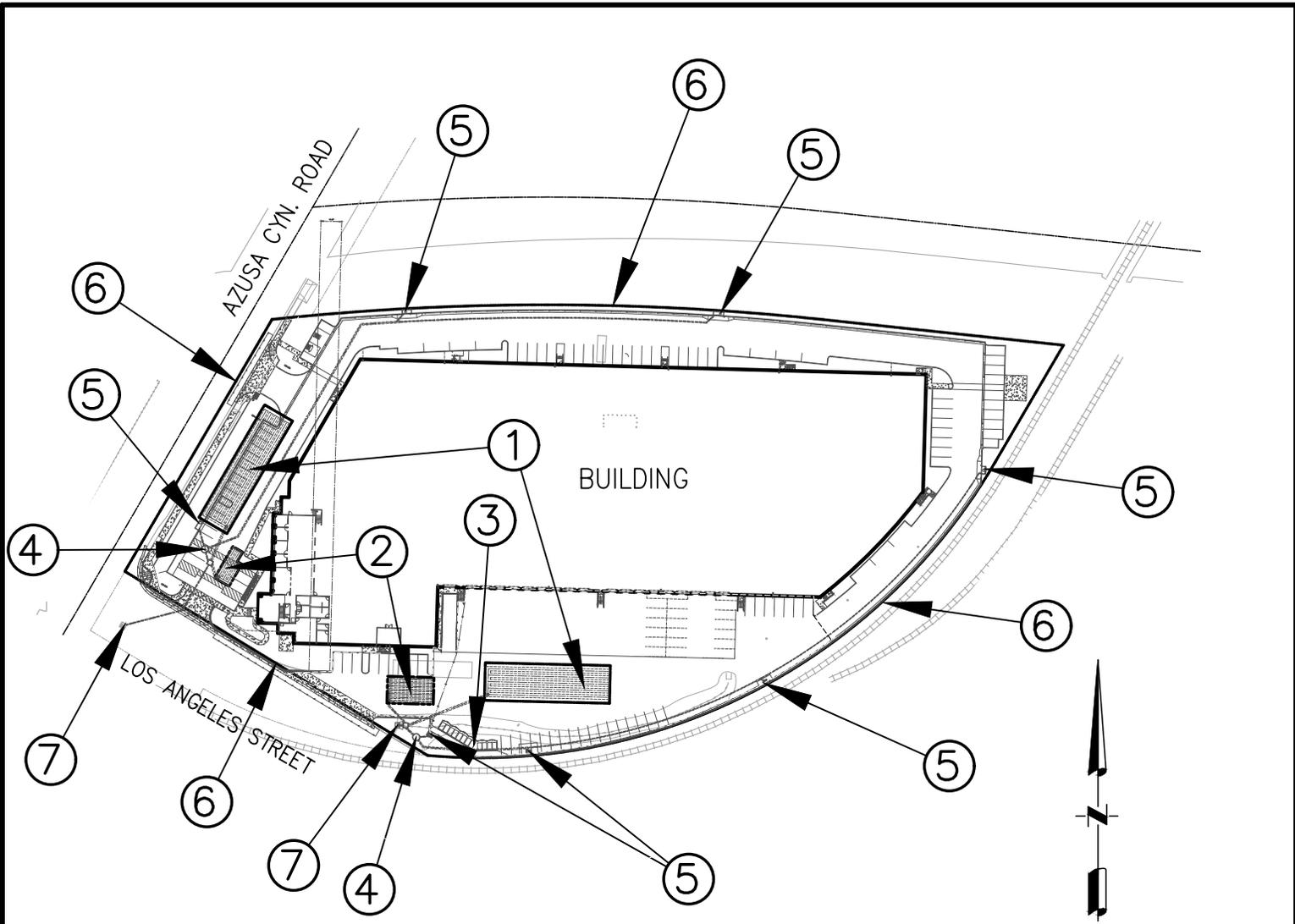
LEGEND

1 — UNDERGROUND INFILTRATION
TRENCH BMP

2 — UNDERGROUND DETENTION
STORAGE SYSTEM

3 — TRASH ENCLOSURE

4 — PT-1: CONTECH ENGINEERING SOLUTIONS
CDS MODEL NO. CDS2020-5-C

5 — STORMWATER INLET WITH BIO-CLEAN FULL
CAPTURE CATCH BASIN FILTER INSERTS

6 — EXISTING PROPERTY LINE

7 — STORMWATER DISCHARGE
CONNECTION POINT TO PUBLIC
STORM DRAIN SYSTEM

SCALE 1" = 150'

STROMWATER QUALITY BMP LOCATION SITE PLAN

4416 AZUSA CANYON ROAD
IRWINDALE, CA

F-34

APPENDIX 6.2

Appendix Cover Sheet.doc

SWQDv INFILTRATION TRECH DESIGN CALCULATIONS

DESIGN DATA:

SWQDv per NextGen Engineering Report Dated Feb. 22, 2021

Total Site SWQDv for Subarea A (Southern Outlet) = 9,570 cubic feet

Total Site SWQDv for Subarea B (Western Outlet) = 8,941 cubic feet

Tested Infiltration Rate per Geotechnical Engineer = 10in/hr

Safety Factor Applied = 3

f design= 3.33 in/hr

t = 96 hours

dt = 6 feet

nt = 40%

Subarea A (Southern Outlet) Design

Determine Minimum Required Infiltration Trench Surface Area

Determine Trench Depth:

d max = {f design / 12} x t

$\qquad$ 3.33 / 12 x 96 = 26.7 feet

dt = 6 < 26.7
Use 6' trench depth.

Min. Trench Area:

$A\ min = \dfrac{SWQD}{dt\ x\ nt}$

- $\dfrac{9,570}{6\ x\ 0.40}$

- Amin = 3,987.5 square feet

<u>Check Proposed Corrugated Pipe Storage Volume for 9,5705 cubic feet</u>

Use 48 inch dia. C.M.P. pipe = 12.6 cubic feet / linear foot

- Use 5 rows of 48 inch dia. pipe x 112 linear feet long

- Use ½ header end section of 48 inch dia. pipe x 28 linear feet long

- 5 x 112 l.f. x 12.6 = 7,056 cubic feet.

- Header end section – 28 L.F. x 12.6 / 2 = 176.4 cubic feet.

- Total Provided Storage In pipes = 7,232.4 cubic feet.

<u>Check Proposed Trench Rock Storage Volume</u>

- T R V = 116 feet long. x 35 feet wide x 6 feet high = 24,360 cubic feet

- T R V – Pipe V = 24,360 – 7,232.4 = 17,036.6

- 17,036.6 x 40% rock voids = 6,144.6 cubic feet

- Total Storage (Rock + Pipe) = 13,377 cubic feet

- 13,377 > 9,570 **Pipes OK**

<u>Check Proposed Infiltration Trench Surface Area</u>

Proposed Infiltration Trench Footprint = 116 feet long. x 35 feet wide = 4,060 SQFT

- 4,060 > 3,987.5 **Use Proposed Layout**

Subarea B (Western Outlet) Design

Determine Minimum Required Infiltration Trench Surface Area

Determine Trench Depth:

d max = {f design / 12} x t

3.33 / 12 x 96 = 26.7 feet

dt = 6 < 26.7
Use 6' trench depth.

Min. Trench Area:

A min = SWQD
       dt x nt

- 8,941
  6 x 0.40

- Amin = 3,725.4 square feet

Check Proposed Corrugated Pipe Storage Volume for 8,941 cubic feet

Use 48 inch dia. C.M.P. pipe = 12.6 cubic feet / linear foot

- Use 5 rows of 48 inch dia. pipe x 112 linear feet long

- Use ½ header end section of 42 inch dia. pipe x 26 linear feet long

- 5 x 112 l.f. x 12.6 = 7,056 cubic feet.

- Header end section – 26 L.F. x 12.6 / 2 = 163.8 cubic feet.

- Total Provided Storage In pipes = 7,219.8 cubic feet.

Check Proposed Trench Rock Storage Volume

- T R V = 121 feet long. x 30 feet wide x 6 feet high = 21,780 cubic feet

- T R V – Pipe V = 21,780 – 7,219.8 = 14,560.2

- 14,560.2 x 40% rock voids = 5,824.1 cubic feet

- Total Storage (Rock + Pipe) = 13,043.9 cubic feet

- 13,043.9 > 8,941 **Pipes OK**

<u>Check Proposed Infiltration Trench Surface Area</u>

Proposed Infiltration Trench Footprint = 121 feet long. x 31 feet wide = 3,751 SQFT

- 3,751 > 3,725.4 **Use Proposed Layout**

APPENDIX 6.3

FULL CAPTURE / SWQDf DESIGN CALCULATIONS

Full Capture Design

Subarea Sizes Reflected on Attached SWQDf & Q100 Volume Map Contained in Appendix 4

Subarea A-1 – 0.17 acres
Subarea A-2 – 0.22 acres
Subarea A-3 – 1.25 acres
Subarea A-4 – 1.27 acres

Subarea B-4 – 0.43 acres
Subarea B-5 – 0.79 acres
Subarea B-6 – 1.50 acres

Q = CIA

Runoff Coefficient

C = 0.90

1-year, 1-hour Rainfall Intensity Per NOAA Atlas 14 (see attached)

 = 0.504

Full Capture Calculations

For Subarea A-1
1-Y,1-H = (0.90) (0.504) (0.17) = 0.08 cfs

For Subarea A-2
1-Y,1-H = (0.90) (0.504) (0.22) = 0.10 cfs

For Subarea A-3
1-Y,1-H = (0.90) (0.504) (1.25) = 0.57 cfs

For Subarea A-4
1-Y,1-H = (0.90) (0.504) (1.27) = 0.58 cfs

For Subarea B-4
1-Y,1-H = (0.90) (0.504) (0.43) = 0.20 cfs

For Subarea B-5
1-Y,1-H = (0.90) (0.504) (0.79) = 0.36 cfs

For Subarea B-6
1-Y,1-H = (0.90) (0.504) (1.50) = 0.68 cfs

CHECK FILTER INSERTS FOR FULL CAPTURE 1-YEAR, 1-HOUR FLOW

Check Bio-Clean Model BC-CURB-FC-24 Catch Basin Filter Insert for Subarea B-6 (Largest Tributary Area)

Manufacturers Treatment Flow filter capacity = 2.85 cfs

Subarea B-6
2.85cfs > 0.68cfs
Filter Unit is OK

Check Bio-Clean Model BC-GRATE-FC-48-48-18 Catch Basin Filter Insert for Subarea A-4 (Largest Tributary Area)

Manufacturers Treatment Flow filter capacity = 25.59 cfs

Subarea A-3
25.59cfs > 0.58cfs
Filter Unit is OK

CHECK FILTER INSERTS FOR HIGH FLOW BYPASS

Check Bio-Clean Model BC-CURB-FC-24 Catch Basin Filter Insert for Subarea B-6 (Largest Area)

Subarea B-6, Q100 flow per NextGen Report = 6.24cfs

Manufacturers High Flow Bypass filter capacity = Unlimited cfs

Subarea B-6
Unlimited cfs > 6.24cfs
Filter Unit is OK

Check Bio-Clean Model BC-GRATE-FC-48-48-18 Catch Basin Filter Insert for Subarea A-3

Subarea A-3, Q100 flow per NextGen Report = 9.08cfs

Manufacturers High Flow Bypass filter capacity = 10.13cfs

Subarea A-3
10.13cfs > 9.08cfs
Filter Unit is OK

SWQDf Design

Subarea SWQDf flows per NextGen Engineering Report Dated 10-1-21
also reflected on Attached SWQDf & Q100 Volume Map Contained in Appendix 4

Subarea A-1 – 0.064cfs
Subarea A-2 – 0.066cfs
Subarea A-3 – 0.333cfs
Subarea A-4 – 0.352cfs

Subarea B-4 – 0.142cfs
Subarea B-5 – 0.204cfs
Subarea B-6 – 0.469cfs

CHECK FILTER INSERTS FOR SWQDf

Check Bio-Clean Model BC-CURB-FC-24 Catch Basin Filter Insert for Subarea B-6 (Largest Tributary Area)

Manufacturers Treatment Flow filter capacity = 2.85 cfs

Subarea B-6
2.85cfs > 0.469cfs
Filter Unit is OK

Check Bio-Clean Model BC-GRATE-FC-48-48-18 Catch Basin Filter Insert for Subarea A-3 (Largest Tributary Area)

Manufacturers Treatment Flow filter capacity = 25.59 cfs

Subarea A-4
25.59cfs > 0.352cfs
Filter Unit is OK

CHECK CDS UNITS FOR TOTAL SWQDf

Check CDS Model 2020-5-0 Filter for Total Subarea a (Largest Tributary Areas)

Manufacturers Treatment Flow filter capacity = 1.1 cfs

Total SWQDf for Subareas A = 0.82cfs

1.1>0.82
CDS Unit OK

NOAA Atlas 14, Volume 6, Version 2
Location name: Baldwin Park, California, USA*
Latitude: 34.0934°, Longitude: -117.9419°
Elevation: 421.59 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

Duration	PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]									
	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	1.78 (1.49-2.15)	2.27 (1.90-2.75)	2.94 (2.45-3.58)	3.50 (2.89-4.30)	4.32 (3.43-5.47)	4.97 (3.86-6.43)	5.64 (4.28-7.51)	6.37 (4.69-8.74)	7.40 (5.22-10.6)	8.24 (5.60-12.2)
10-min	1.28 (1.07-1.54)	1.63 (1.36-1.97)	2.11 (1.75-2.56)	2.51 (2.08-3.08)	3.09 (2.46-3.92)	3.56 (2.77-4.61)	4.04 (3.07-5.38)	4.57 (3.37-6.26)	5.31 (3.74-7.60)	5.90 (4.02-8.77)
15-min	1.03 (0.860-1.24)	1.31 (1.10-1.59)	1.70 (1.41-2.06)	2.03 (1.67-2.48)	2.49 (1.98-3.16)	2.87 (2.23-3.72)	3.26 (2.48-4.34)	3.68 (2.71-5.05)	4.28 (3.02-6.13)	4.76 (3.24-7.07)
30-min	0.706 (0.590-0.854)	0.900 (0.752-1.09)	1.17 (0.970-1.42)	1.39 (1.15-1.71)	1.71 (1.36-2.17)	1.97 (1.53-2.55)	2.24 (1.70-2.98)	2.53 (1.86-3.47)	2.94 (2.07-4.21)	3.27 (2.22-4.85)
60-min	0.504 (0.421-0.610)	0.643 (0.536-0.779)	0.833 (0.693-1.01)	0.994 (0.820-1.22)	1.22 (0.973-1.55)	1.41 (1.09-1.82)	1.60 (1.21-2.13)	1.81 (1.33-2.48)	2.10 (1.48-3.00)	2.33 (1.59-3.47)
2-hr	0.372 (0.310-0.450)	0.470 (0.392-0.570)	0.605 (0.503-0.735)	0.719 (0.593-0.881)	0.882 (0.702-1.12)	1.01 (0.788-1.31)	1.15 (0.873-1.53)	1.30 (0.956-1.78)	1.51 (1.06-2.16)	1.68 (1.14-2.49)
3-hr	0.313 (0.261-0.378)	0.394 (0.328-0.477)	0.505 (0.420-0.613)	0.599 (0.494-0.734)	0.734 (0.584-0.931)	0.842 (0.656-1.09)	0.956 (0.726-1.27)	1.08 (0.795-1.48)	1.25 (0.884-1.79)	1.39 (0.949-2.07)
6-hr	0.226 (0.189-0.273)	0.284 (0.237-0.344)	0.364 (0.303-0.442)	0.432 (0.356-0.529)	0.529 (0.421-0.671)	0.607 (0.472-0.787)	0.689 (0.523-0.917)	0.777 (0.573-1.07)	0.903 (0.637-1.29)	1.01 (0.684-1.49)
12-hr	0.148 (0.124-0.180)	0.189 (0.158-0.229)	0.245 (0.203-0.297)	0.292 (0.241-0.358)	0.359 (0.286-0.456)	0.413 (0.322-0.536)	0.471 (0.357-0.627)	0.532 (0.392-0.729)	0.620 (0.437-0.887)	0.691 (0.470-1.02)
24-hr	0.097 (0.086-0.112)	0.127 (0.112-0.146)	0.167 (0.147-0.193)	0.201 (0.176-0.235)	0.249 (0.211-0.301)	0.288 (0.239-0.355)	0.330 (0.267-0.415)	0.374 (0.294-0.484)	0.436 (0.330-0.589)	0.487 (0.356-0.680)
2-day	0.059 (0.052-0.068)	0.079 (0.070-0.091)	0.106 (0.093-0.123)	0.129 (0.113-0.150)	0.161 (0.136-0.194)	0.187 (0.155-0.230)	0.215 (0.174-0.271)	0.244 (0.192-0.316)	0.286 (0.216-0.385)	0.319 (0.233-0.446)
3-day	0.044 (0.039-0.050)	0.059 (0.052-0.068)	0.080 (0.071-0.093)	0.098 (0.086-0.115)	0.124 (0.105-0.149)	0.144 (0.119-0.177)	0.166 (0.134-0.209)	0.189 (0.148-0.244)	0.221 (0.167-0.298)	0.248 (0.181-0.346)
4-day	0.036 (0.031-0.041)	0.048 (0.043-0.056)	0.066 (0.058-0.077)	0.081 (0.071-0.095)	0.102 (0.087-0.124)	0.120 (0.099-0.147)	0.138 (0.111-0.174)	0.157 (0.124-0.204)	0.185 (0.140-0.249)	0.207 (0.151-0.289)
7-day	0.023 (0.021-0.027)	0.032 (0.028-0.037)	0.044 (0.039-0.051)	0.054 (0.047-0.063)	0.068 (0.058-0.082)	0.080 (0.066-0.099)	0.093 (0.075-0.117)	0.106 (0.084-0.138)	0.125 (0.095-0.169)	0.141 (0.103-0.197)
10-day	0.018 (0.016-0.020)	0.024 (0.021-0.028)	0.033 (0.029-0.038)	0.041 (0.036-0.048)	0.052 (0.044-0.063)	0.061 (0.051-0.076)	0.071 (0.058-0.090)	0.082 (0.064-0.106)	0.097 (0.073-0.131)	0.110 (0.080-0.153)
20-day	0.010 (0.009-0.012)	0.014 (0.013-0.017)	0.020 (0.018-0.023)	0.025 (0.022-0.029)	0.032 (0.027-0.038)	0.038 (0.031-0.046)	0.044 (0.035-0.055)	0.050 (0.040-0.065)	0.060 (0.045-0.081)	0.068 (0.050-0.095)
30-day	0.008 (0.007-0.009)	0.011 (0.010-0.013)	0.016 (0.014-0.018)	0.020 (0.017-0.023)	0.025 (0.021-0.030)	0.030 (0.025-0.037)	0.035 (0.028-0.044)	0.040 (0.032-0.052)	0.048 (0.036-0.065)	0.055 (0.040-0.076)
45-day	0.006 (0.006-0.007)	0.009 (0.008-0.010)	0.012 (0.011-0.014)	0.015 (0.013-0.018)	0.020 (0.017-0.024)	0.023 (0.019-0.029)	0.027 (0.022-0.034)	0.031 (0.025-0.041)	0.038 (0.028-0.051)	0.043 (0.031-0.060)
60-day	0.006 (0.005-0.006)	0.008 (0.007-0.009)	0.010 (0.009-0.012)	0.013 (0.011-0.015)	0.017 (0.014-0.020)	0.020 (0.016-0.024)	0.023 (0.019-0.029)	0.026 (0.021-0.034)	0.032 (0.024-0.043)	0.036 (0.026-0.050)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical

F-44
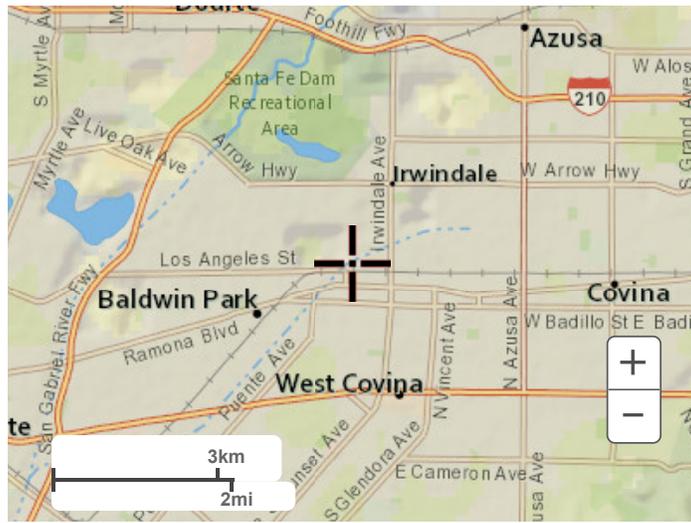
Large scale terrain

Large scale map

Large scale aerial

APPENDIX 6.4

Appendix Cover Sheet.doc

BIO CLEAN FULL CAPTURE FILTER WITH TROUGH SYSTEM

FOR USE IN CURB INLETS

STAINLESS STEEL DRIVE PIN

TROUGH

WEIR

FILTER BASKET

FIGURE 1:
DETAIL OF PARTS

TROUGH POSITIONS FILTER BASKET DIRECTLY UNDER MANHOLE OPENING FOR EASY ACCESS

CURB OPENING

WIDTH OF INLET WILL VARY

FIGURE 2:
DETAIL OF CONFIGURATION

CURB OPENING

MANHOLE

WEIR

30" MINIMUM

FILTER BASKET

OUTLET PIPE
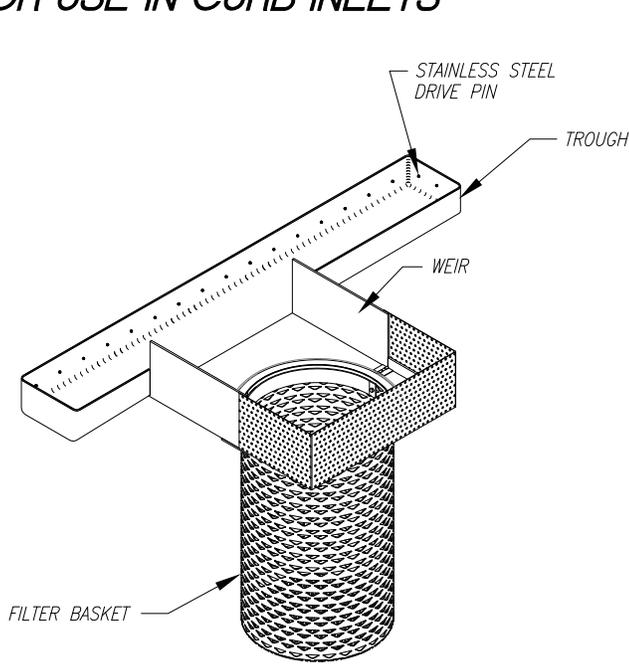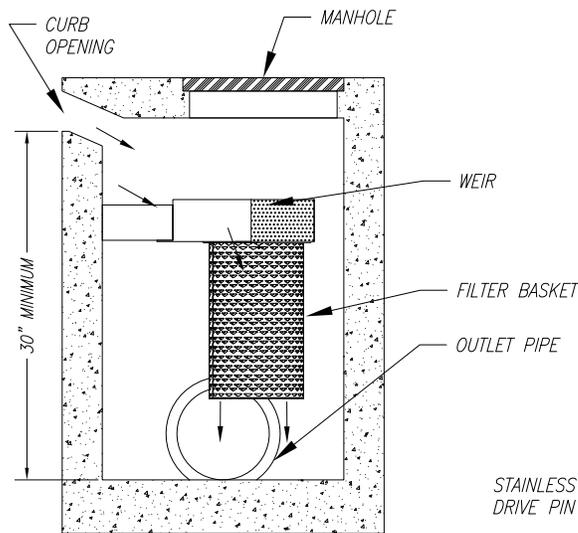
FIGURE 4:
DETAIL OF PROFILE

TROUGH

STAINLESS STEEL DRIVE PIN
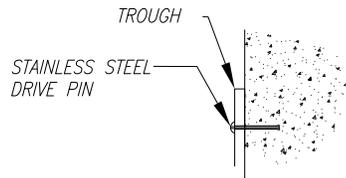
FIGURE 3:
DETAIL OF MOUNTING

NOTES:

1. TROUGH SYSTEM PROVIDES FOR ENTIRE COVERAGE OF INLET OPENING SO TO DIVERT ALL FLOW TO FILTER.
2. TROUGH SYSTEM MANUFACTURED FROM MARINE GRADE FIBERGLASS, GEL COATED FOR UV PROTECTION.
3. SYSTEM ATTACHED TO THE CATCH BASIN WITH NON-CORROSIVE HARDWARE.
4. FILTER MANUFACTURED OF 100% STAINLESS STEEL.
5. FILTER MADE OF NON-CLOGGIN SCREEN WITH 4.7 MM OPENINGS AND MEETS FULL CAPTURE REQUIREMENTS.
6. FILTER CAN BE FITTED WITH HYDROCARBON ABSORBENT BOOM
7. FILTER IS LOCATED DIRECTLY UNDER THE MANHOLE FOR EASY REMOVAL AND MAINTENANCE.
8. LENGTH OF TROUGH CAN VARY FROM 2' TO 30'
9. OTHER STANDARD AND CUSTOM MODEL SIZES AVAILABLE – CONTACT BIO CLEAN FOR MORE INFORMATION.
10. CONSIDERS A SAFETY FACTOR OF 2.0
11. BYPASS IS FACILITATED VIA OVERFLOW OF THE TROUGH SYSTEM AND IS EQUAL TO THE CAPACITY OF THE CURB OPENING
12. STORAGE CAPACITY BASED ON THE BASKET HALF FULL.
13. ADDITIONAL TREATMENT AND STORAGE CAPACITY CAN BE ACHIEVED BY UTILIZING MULTIPLE FILTER BASKETS.

MODEL NUMBER	TREATMENT FLOW (cfs)*	SOLIDS STORAGE CAPACITY (cu ft)
BC-CURB-FC-30	2.85	2.21
BC-CURB-FC-24	2.85	1.77
BC-CURB-FC-18	2.85	1.33
BC-CURB-FC-12	2.85	0.88

*SEE PAGE 2 FOR EXPLANATION OF FLOW RATES

DRAWING: BIO CLEAN CURB INLET FILTER DETAILS	MEETS FULL CAPTURE REQUIREMENTS
TREATMENT FLOW RATE: 2.85 cfs	MODEL #: BC-CURB-FC
WARRANTY: 5 YEAR MANUFACTURERS	PROJECT:

BIO CLEAN ENVIRONMENTAL SERVICES, INC.
398 VIA EL CENTRO, OCEANSIDE CA 92058
PHONE:760-433-7640   FAX:760-433-3176

DATE: 10/12/2017   SCALE: NTS

DRAFTER: M.C.P.   UNITS = INCHES

REVISIONS: DATE:
REVISIONS: DATE:
REVISIONS: DATE:
REVISIONS: DATE:

F-47

Bio ⬦ Clean
A Forterra Company

PAGE 1
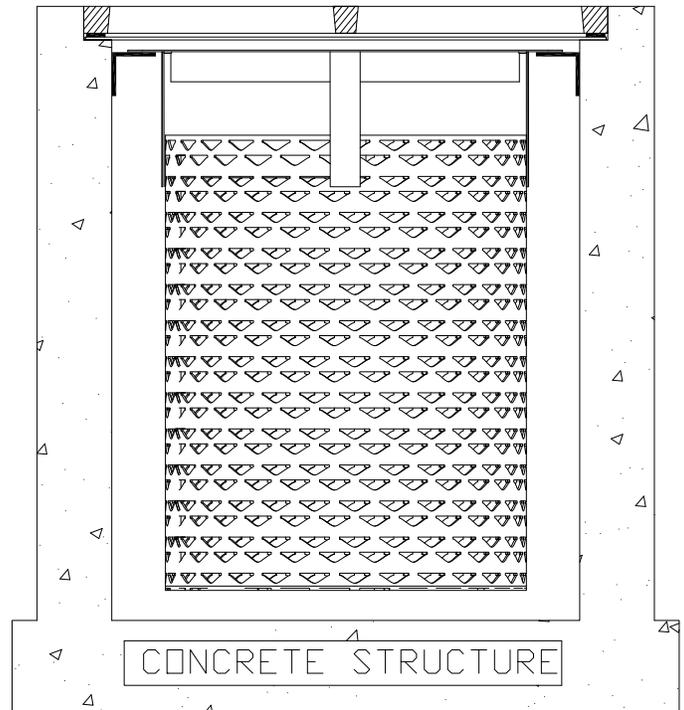
BIO CLEAN FULL CAPTURE FILTER
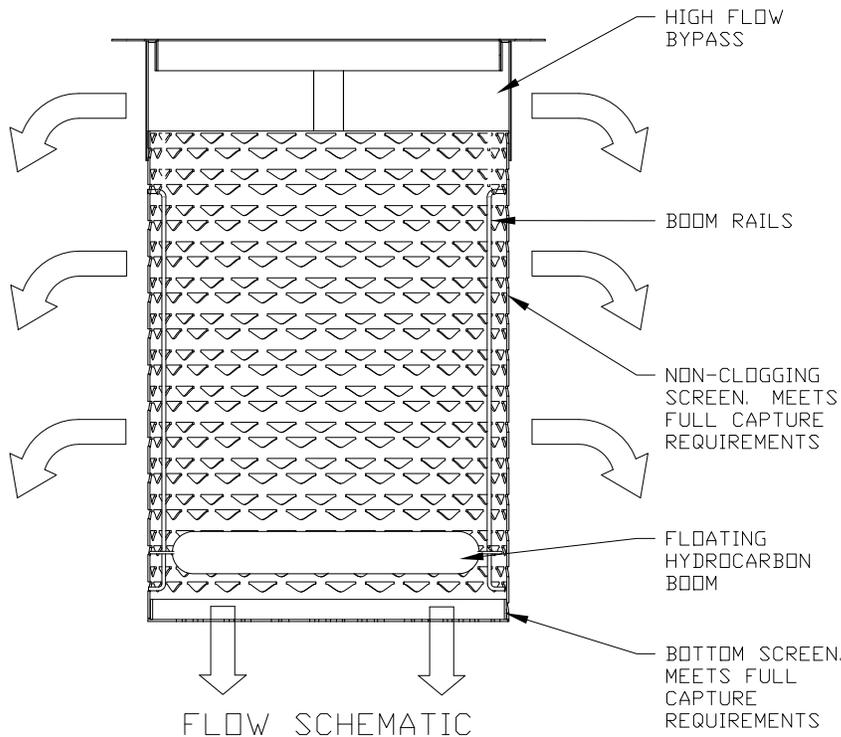
FOR USE IN GRATE INLETS
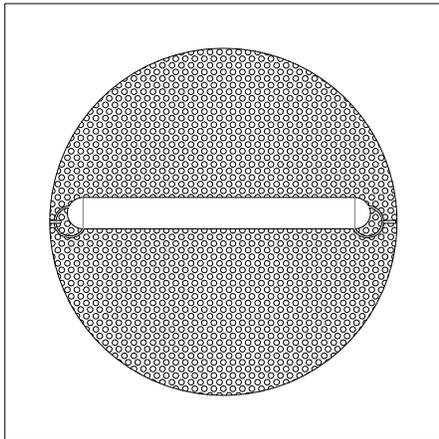
TOP VIEW

NOTES:

1. ALL HARDWARE, FLANGE, FRAME, SCREENS SHALL BE STAINLESS STEEL
2. HYDROCARBON BOOM SHALL BE 2" DIAMETER AND CONNECTED, MECHANICALLY TO THE FILTER FRAME WITH RAILS ALLOWING IT TO FLOAT ON THE WATER SURFACE REGARDLESS OF HEIGHT
3. SEE PERFORMANCE REPORTS IN MANUFACTURES SPECIFICATIONS
4. OTHER STANDARD AND CUSTOM MODEL SIZES AVAILABLE — CONTACT BIO CLEAN FOR MORE INFORMATION.
5. BASED ON 37% OPEN AREA.
6. CONSIDERS A SAFETY FACTOR OF 2.0.
7. CONSIDERS A LOCAL DEPRESSION PONDIING DEPTH OF 6 INCHES.
8. STORAGE CAPACITY BASED ON THE BASKET HALF FULL.

CONCRETE STRUCTURE

FLOW SCHEMATIC

- HIGH FLOW BYPASS
- BOOM RAILS
- NON-CLOGGING SCREEN. MEETS FULL CAPTURE REQUIREMENTS
- FLOATING HYDROCARBON BOOM
- BOTTOM SCREEN. MEETS FULL CAPTURE REQUIREMENTS

MODEL #	TREATMENT FLOW (CFS)	BYPASS FLOW (CFS)	SOLIDS STORAGE CAPACITY (CF)
BC-GRATE-FC 12-12-12	1.55	1.55	0.27
BC-GRATE-FC 18-18-18	4.32	3.68	1.05
BC-GRATE-FC 24-24-24	7.67	4.83	2.41
BC-GRATE-FC 30-30-24	12.97	6.21	3.98
BC-GRATE-FC 25-38-24	13.53	6.59	4.16
BC-GRATE-FC 36-36-24	19.64	7.60	5.94
BC-GRATE-FC 48-48-18	25.59	10.13	7.92

Bio Clean
A Forterra Company

F-48

PAGE 1

APPENDIX 6.5

PLAN VIEW B-B
N.T.S.
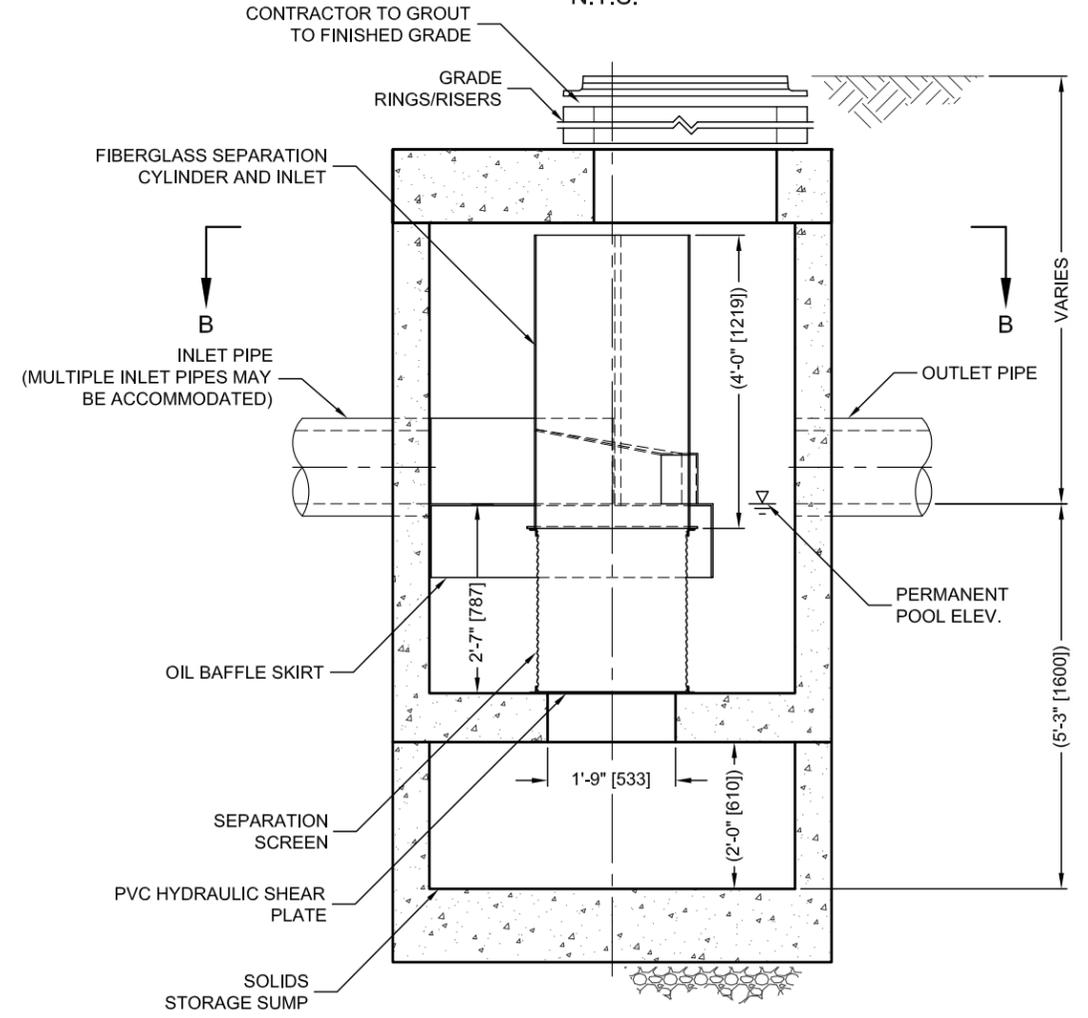
FIBERGLASS SEPARATION CYLINDER AND INLET

CENTER OF CDS STRUCTURE, SCREEN AND SUMP OPENING

FLOW

A

A

FLOW

+/-135° MAX.

PVC HYDRAULIC SHEAR PLATE

TOP SLAB ACCESS (SEE FRAME AND COVER DETAIL)

60" [1524] I.D. MANHOLE STRUCTURE

+/-65° MAX.

ELEVATION A-A
N.T.S.

CONTRACTOR TO GROUT TO FINISHED GRADE

GRADE RINGS/RISERS

FIBERGLASS SEPARATION CYLINDER AND INLET

B

B

INLET PIPE (MULTIPLE INLET PIPES MAY BE ACCOMMODATED)

OUTLET PIPE

VARIES

(4'-0" [1219])

PERMANENT POOL ELEV.

OIL BAFFLE SKIRT

2'-7" [787]

(5'-3" [1600])

1'-9" [533]

(2'-0" [610])

SEPARATION SCREEN

PVC HYDRAULIC SHEAR PLATE

SOLIDS STORAGE SUMP

CDS2020-5-C DESIGN NOTES

CDS2020-5-C RATED TREATMENT CAPACITY IS 1.1 CFS [31.2 L/s], OR PER LOCAL REGULATIONS. MAXIMUM HYDRAULIC INTERNAL BYPASS CAPACITY IS 14.0 CFS [396 L/s]. IF THE SITE CONDITIONS EXCEED 14.0 CFS [396 L/s], AN UPSTREAM BYPASS STRUCTURE IS REQUIRED.

THE STANDARD CDS2020-5-C CONFIGURATION IS SHOWN. ALTERNATE CONFIGURATIONS ARE AVAILABLE AND ARE LISTED BELOW. SOME CONFIGURATIONS MAY BE COMBINED TO SUIT SITE REQUIREMENTS.

CONFIGURATION DESCRIPTION

CONFIGURATION DESCRIPTION
GRATED INLET ONLY (NO INLET PIPE)
GRATED INLET WITH INLET PIPE OR PIPES
CURB INLET ONLY (NO INLET PIPE)
CURB INLET WITH INLET PIPE OR PIPES
SEPARATE OIL BAFFLE (SINGLE INLET PIPE REQUIRED FOR THIS CONFIGURATION)
SEDIMENT WEIR FOR NJDEP / NJCAT CONFORMING UNITS
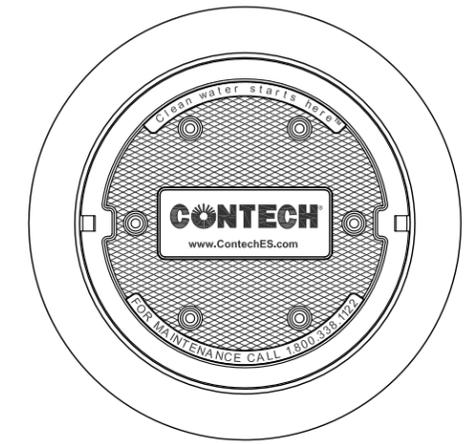
FRAME AND COVER
(DIAMETER VARIES)
N.T.S.

C⌀NTECH
www.ContechES.com

SITE SPECIFIC DATA REQUIREMENTS

STRUCTURE ID	
WATER QUALITY FLOW RATE (CFS OR L/s)	*
PEAK FLOW RATE (CFS OR L/s)	*
RETURN PERIOD OF PEAK FLOW (YRS)	*
SCREEN APERTURE (2400 OR 4700)	*

PIPE DATA:	I.E.	MATERIAL	DIAMETER
INLET PIPE 1	*	*	*
INLET PIPE 2	*	*	*
OUTLET PIPE	*	*	*

RIM ELEVATION		*

ANTI-FLOTATION BALLAST	WIDTH	HEIGHT
	*	*

NOTES/SPECIAL REQUIREMENTS:

* PER ENGINEER OF RECORD

GENERAL NOTES
1. CONTECH TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.
2. DIMENSIONS MARKED WITH ( ) ARE REFERENCE DIMENSIONS. ACTUAL DIMENSIONS MAY VARY.
3. FOR FABRICATION DRAWINGS WITH DETAILED STRUCTURE DIMENSIONS AND WEIGHTS, PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS LLC REPRESENTATIVE. www.ContechES.com
4. CDS WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING.
5. STRUCTURE SHALL MEET AASHTO HS20 AND CASTINGS SHALL MEET HS20 (AASHTO M 306) LOAD RATING, ASSUMING GROUNDWATER ELEVATION AT, OR BELOW, THE OUTLET PIPE INVERT ELEVATION. ENGINEER OF RECORD TO CONFIRM ACTUAL GROUNDWATER ELEVATION.
6. PVC HYDRAULIC SHEAR PLATE IS PLACED ON SHELF AT BOTTOM OF SCREEN CYLINDER. REMOVE AND REPLACE AS NECESSARY DURING MAINTENANCE CLEANING.

INSTALLATION NOTES
A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE CDS MANHOLE STRUCTURE (LIFTING CLUTCHES PROVIDED).
C. CONTRACTOR TO ADD JOINT SEALANT BETWEEN ALL STRUCTURE SECTIONS, AND ASSEMBLE STRUCTURE.
D. CONTRACTOR TO PROVIDE, INSTALL, AND GROUT PIPES. MATCH PIPE INVERTS WITH ELEVATIONS SHOWN.
E. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO ASSURE UNIT IS WATER TIGHT, HOLDING WATER TO FLOWLINE INVERT MINIMUM. IT IS SUGGESTED THAT ALL JOINTS BELOW PIPE INVERTS ARE GROUTED.

CDS

C⌀NTECH®
ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122   513-645-7000   513-645-7993 FAX

CDS2020-5-C
INLINE CDS
STANDARD DETAIL

F-50

APPENDIX 6.6

Appendix Cover Sheet.doc

National Flood Hazard Layer FIRMette

FEMA

34°5'50.73"N
117°56'53.49"W

CITY OF IRWINDALE
060129
AREA OF MINIMAL FLOOD HAZARD
Zone X

06037C1700F
eff. 9/26/2008

CITY OF WEST COVINA
060666

0.2 PCT ANNUAL CHANCE FLOOD HAZARD
Zone X

CITY OF BALDWIN PARK
060100

LOS ANGELES COUNTY
065043

USGS The National Map: Orthoimagery. Data refreshed April, 2019.

117°56'16.03"W
34°5'20.94"N

Feet					
0	250	500	1,000	1,500	2,000

1:6,000 F-52

MEMORANDUM

NextGen
ENGINEERING

To: **The G4 Group**
From: **Bill O'Brien, PE; Tariq Tariq, EIT - NextGen Engineering**
Date: **October 1, 2021**
Subject: **Hydrology and Detention for 4416 Azusa Canyon Rd.**
Irwindale, CA 91706
APN 8414004006

Executive Summary

In response to the California Environmental Quality Act (CEQA) comments, this technical memorandum revises the previous technical memo (dated 5-17-21) and summarizes the hydrologic analysis, water quality requirements, and detention volume calculations for the re-development at the subject property (see Figure 1), located in Irwindale (City), in Los Angeles County (County).

Figure 1. Location Map (with subject parcel in red)

A hydrologic analysis was performed for the subject property under two conditions: existing and proposed. The property was divided into sub-areas that were defined by topographical and drainage features, then measurements and sub-basin characteristics were taken utilizing AutoCAD. After the contributing sub-basin were hydrologically characterized, LA County's HydroCalc model was run for each sub-area. The HydroCalc-generated hydrographs for individual sub-areas were then combined into an equivalent outflow hydrograph (simple addition without routing), which served as the basis for detention volume calculations. Utilizing the hydrologic characterization of the sub-basins, separate HydroCalc runs were performed to obtain the stormwater quality design volumes and flows (SWQDV-F).

The hydrologic analysis determined that the most extreme condition for detention volume corresponds to mitigating the 100-year proposed conditions peak flow (22.97 cfs) to the 50-year

374 Poli St., Suite 207, Ventura, California, 93001   TEL. (805) 798-7664   www.NGenEng.com
Mail: 5142 E. Holmes St., Tucson, AZ 85711

F-53

existing conditions peak flow (16.01 cfs); which corresponds to a detention volume of 1,702 cubic feet. This detention volume ensures that the 100-year proposed conditions peak flow is outlet at the 50-year existing condition peak flow.

In addition to the detention requirements, HydroCalc was utilized for obtaining the SWQDV, which was determined to correspond to the 85th percentile rainfall event, with a corresponding SWQDV of 18,511 cubic feet. This volume is provided in underground infiltration chambers at two locations on the site (see Proposed Conditions Hydrology Map in Attachment 1).

Project Background

The property is 5.82 acres in size and is currently used for a Pepsi bottling facility. The existing lot is mostly impervious and is composed of the main facility building (70,595 sq. ft.) in addition to parking spaces, and truck loading/unloading docks.

The proposed development constitutes re-grading the lot, increasing the building footprint and additional parking spaces.

The site is situated on the northeast corner of the intersection of Azusa Canyon Road and Los Angeles Street. The site is spanned by railroad tracks on the southeast side and sits on the southern overbank of Big Dalton wash, a major drainage canal in the area.

For both existing and proposed conditions drainage, the site utilizes both surface and subsurface drainage for handling stormwater. This memo focuses on the surface drainage component of the site, with assumption about sub-surface drainage.

For hydrology, County's Hydrology Manual (2006) was referenced and their public hydrology software, HydroCalc, was utilized (see Hydrology sections).

It is understood that the County's mitigation criteria for the re-development project limits the proposed conditions 100-year peak discharge (Q100) to the existing conditions 50-year peak discharge (Q50).

Existing Conditions Hydrology

The natural topography of the area and field investigation confirm that the existing developed watershed is confined to the subject property boundary, without any run-on flows. Onsite-generated stormwater is handled by surface and subsurface drainage mechanisms. An existing conditions hydrology map (see Exhibit 1 in Attachment 1) was prepared to visually demonstrate how the site drains. As shown on the map, a small portion on the north side of the property (Sub-area A-1) drains to Big Dalton wash through overland sheet flow. The remaining portion of the property concentrates at two locations, and ultimately discharge to the existing, half-filled 36" RCP storm drainpipe under Los Angeles Street through underground storm drainpipes. This area was subdivided into two sub-areas (Sub-areas B-2 and B-3), each discharging to unique onsite points of concentration, as shown on Exhibit 1 in Attachment 1.

County's online hydrology GIS map was utilized to get an average rainfall depth for the entire property. The 50-year, 24-hr rainfall depth was estimated as 6.9 inches for the property (see Figure 2). For determining the soil properties of each sub-area, the County's soil shapefiles were imported into AutoCAD and overlaid on top of the property boundary, as shown in Figure 3.

The remaining HydroCalc inputs were obtained by taking measurements from the AutoCAD file that created the hydrology map. Table 1 summarizes the HydroCalc inputs for each sub-area.
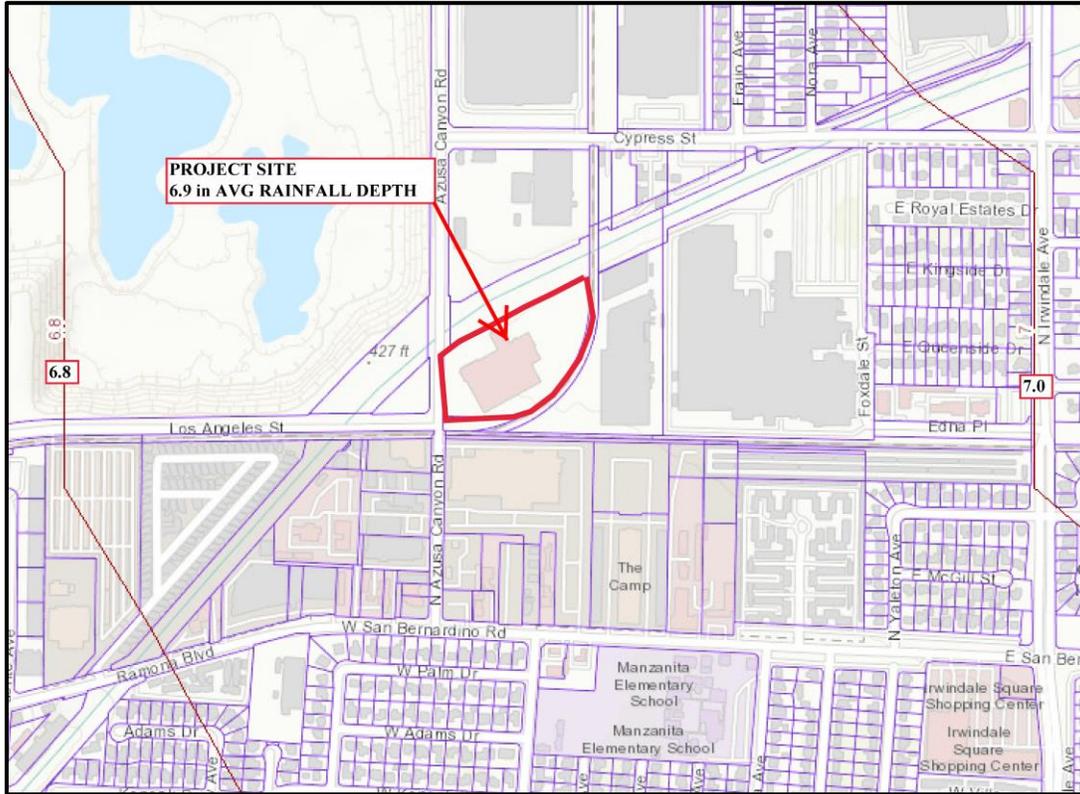
Figure 2. 50-yr, 24-hr Rainfall Contours Map (with subject parcel in red)
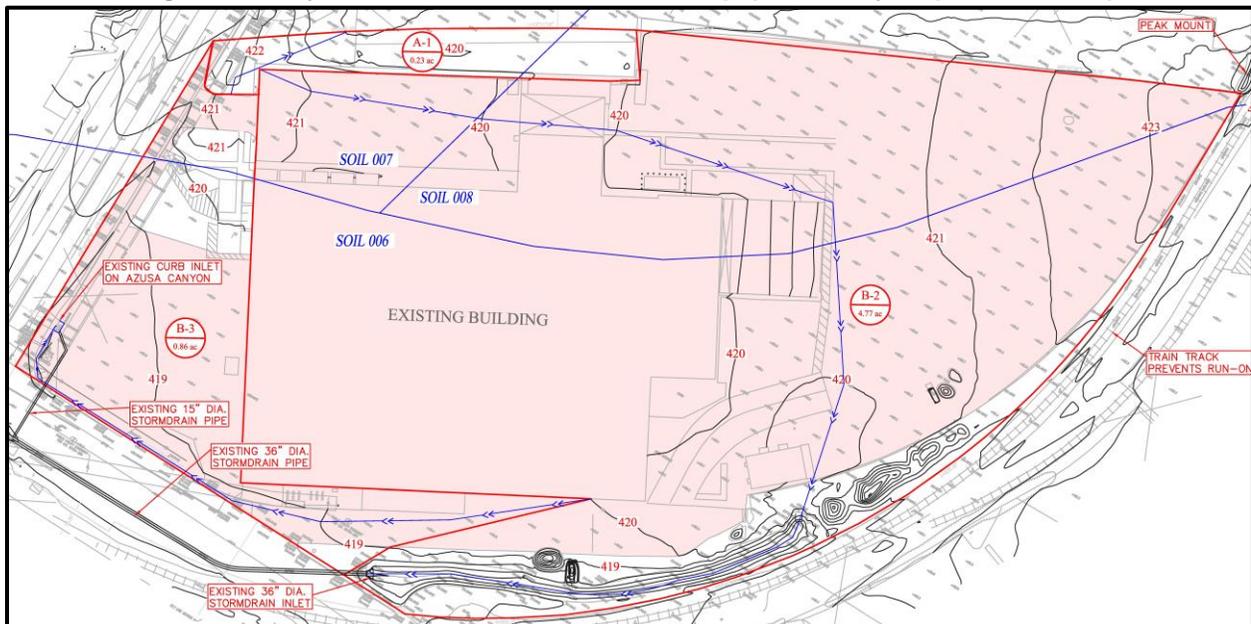
Figure 3. Existing Conditions Soil-Type Map (with sub-areas shown in red)

Table 1. HydroCalc Inputs for Existing Conditions Hydrology

Subarea (ID)	Area (ac)	Flow Path (ft)	Flow Path Slope (vft/hft)	Impervious Area (dec. %)	Soil Type	Fire Factor	24-hr, 50 -yr Rainfall Depth (in)
A-1	0.23	98	0.0347	0.02	007		
B-2	4.77	978	0.0108	0.91	006	0.71	6.9
B-3	0.86	465	0.0043	0.82	006		

*The fire factor was determined from Table 6.3.3 in the County's Hydrology Manual (2006), recognizing that the property is part of the San Gabriel River Watershed.
*HydroCalc automatically calculates the 100-year rainfall depth from the 50-year, 24-hr rainfall depth.

HydroCalc run was performed twice for each sub-area to obtain the 50-year and 100-year hydrographs at the outlet of each sub-area. Given that sub-area A-1 discharges into Dalton Wash through overland sheet flow, the hydrograph obtained for A-1 demonstrates the outflow hydrograph for the north portion of the site (see Figure 4). For the remaining portion of the property, the hydrographs for the two sub-areas were simply combined into an equivalent outflow hydrograph (Figure 4), representing the flow rates discharged into the existing storm drain system.
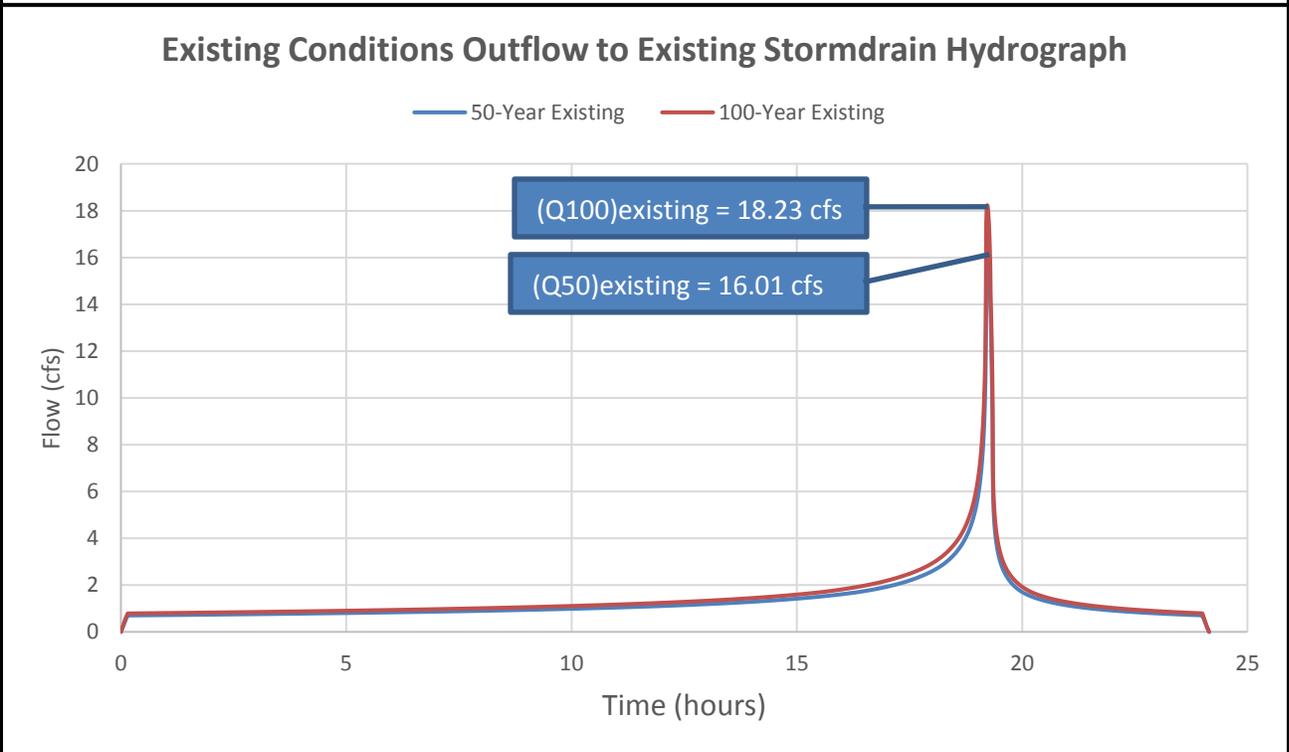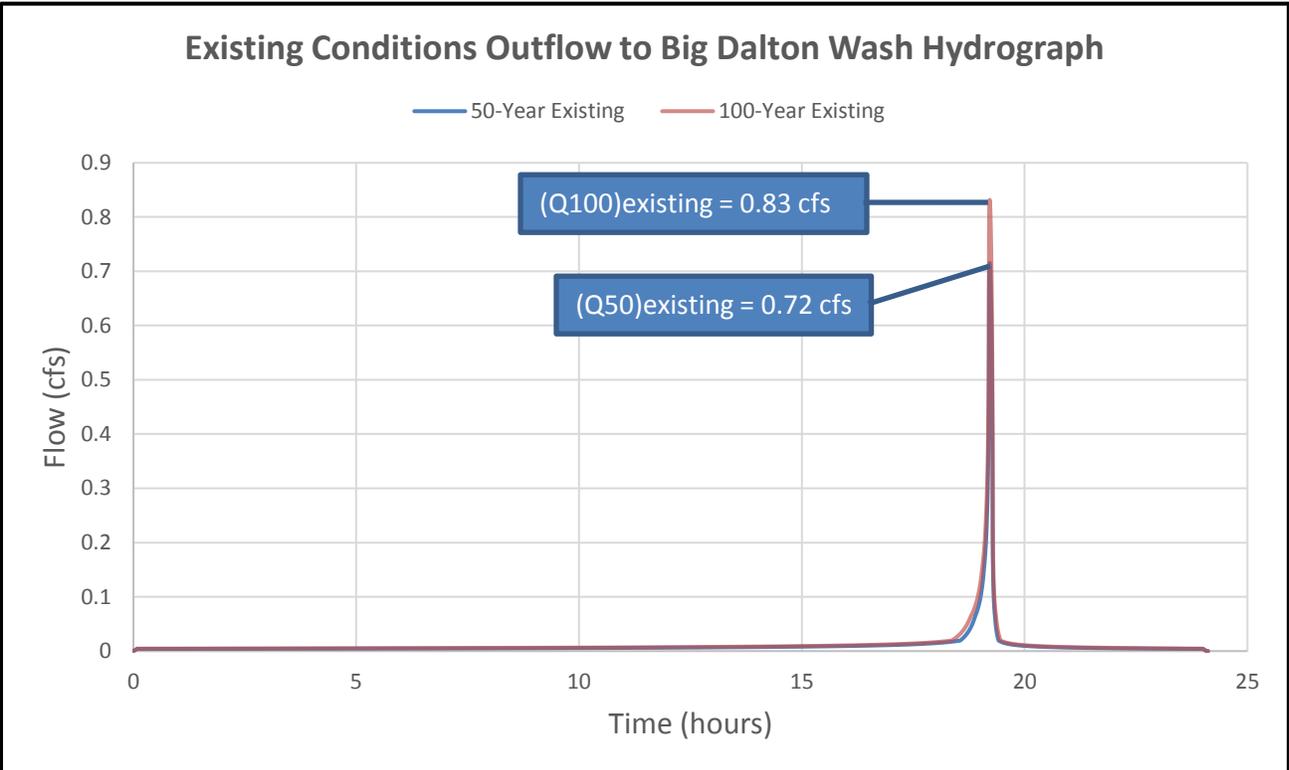
Figure 4. Existing Conditions Outflow Hydrographs to Big Dalton Wash (top) and to Existing Storm Drain on Los Angeles Street (bottom)

As shown on Figure 4, the existing conditions peak Q50 is 0.72 cfs to Big Dalton Wash and 16.01 cfs to the existing storm drain system under Los Angeles Street. The existing conditions peak Q100 is 0.83 cfs to Big Dalton Wash and 18.23 cfs to the existing storm drain system under Los Angeles Street.

Proposed Conditions Hydrology

The re-development project proposes to make further use of subsurface storm drainage, adding seven (7) new onsite stormdrain inlets that collect all of the onsite stormwater and re-directs it to the existing, half-filled 36" RCP storm drainpipe under Los Angeles Street through subsurface drainage pipes. A proposed conditions hydrology map was prepared to visually demonstrate how stormwater drains for the proposed site conditions (see Exhibit 2 in Attachment 1). The property was subdivided into eight sub-areas using proposed topography, terminating each sub-area at the location where water concentrates at a proposed catch basin/grated inlet.

Rainfall depth map remains unchanged from the existing conditions hydrology. Figure 5 shows the soil type map for proposed conditions hydrology.
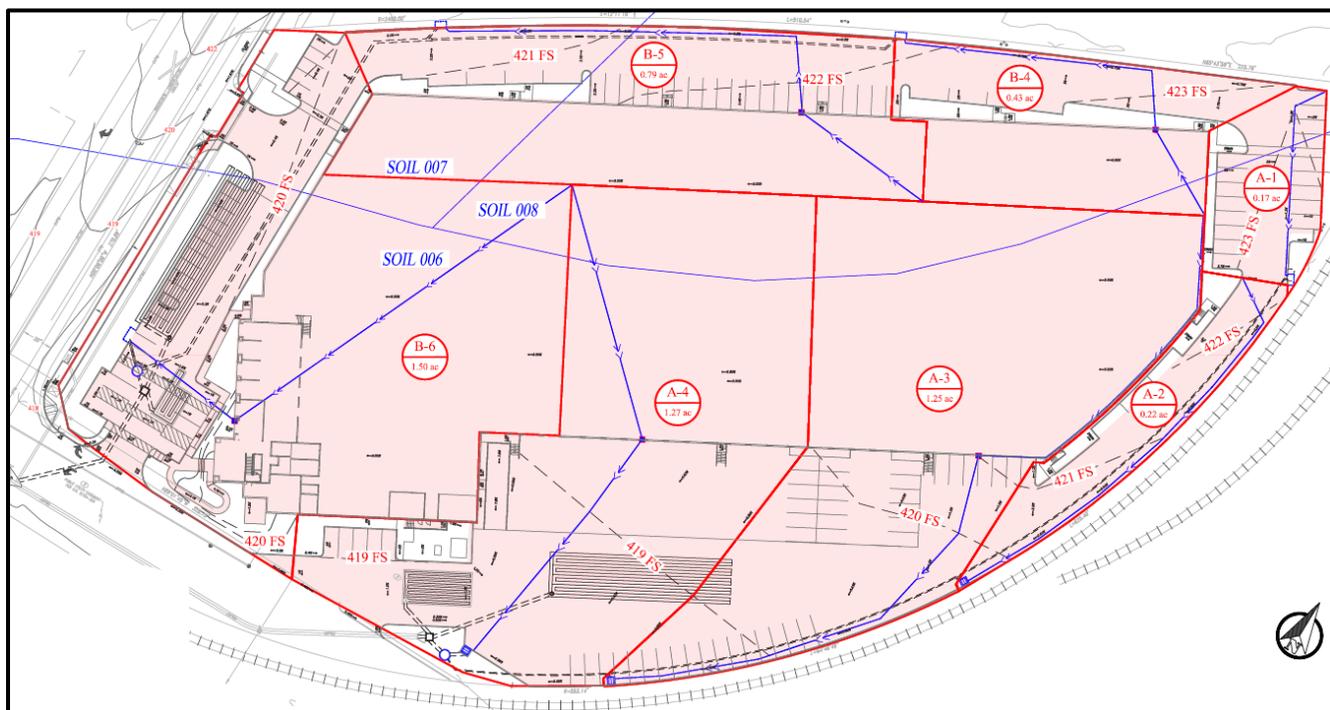
Figure 5. Proposed Conditions Soils Type Map (with sub-areas shown in red)

Following similar methodology as described in the Existing Conditions Hydrology section of the memo, Table 2 was prepared, summarizing the characteristics of each sub-area under proposed conditions.

Table 2. HydroCalc Inputs for Proposed Conditions Hydrology

Subarea (ID)	Area (ac)	Flow Path (ft)	Flow Path Slope (vft/hft)	Impervious Area (decimal)	Soil Type	Fire Factor	24-hr, 50 -yr Rainfall Depth (in)
A-1	0.17	130	0.0102	0.892	006		
A-2	0.22	280	0.0119	0.916	006		
A-3	1.25	531	0.0144	1.000	006		
A-4	1.27	336	0.0135	0.943	006	0.71	6.9
B-4	0.43	257	0.0069	0.893	008		
B-5	0.79	363	0.0067	0.959	008		
B-6	1.50	340	0.0062	0.952	006		

Following similar procedure as used for modelling existing conditions hydrology, two HydroCalc runs were performed for each sub-area, representing the 50-year and 100-year design frequency events. The proposed grading plan for the re-development project eliminates onsite drainage to Big Dalton wash; hence, the entire property drains to the existing storm drain system under Los Angeles Street under proposed conditions.

For determining the detention storage needed to achieve the mitigation criteria, equivalent outflow hydrographs were prepared (Figure 6) by combining the hydrographs of the individual sub-areas.

Outflow from Developed Site to Existing Stormdrain Hydrograph

Legend: 50-Year Proposed — 100-Year Proposed — 50-Year Existing — 100-Year Existing

(Q100)proposed = 22.97 cfs (un-mitigated)
(Q50)proposed = 20.46 cfs (un-mitigated)
(Q100)existing = 18.23 cfs
(Q50)existing = 16.01
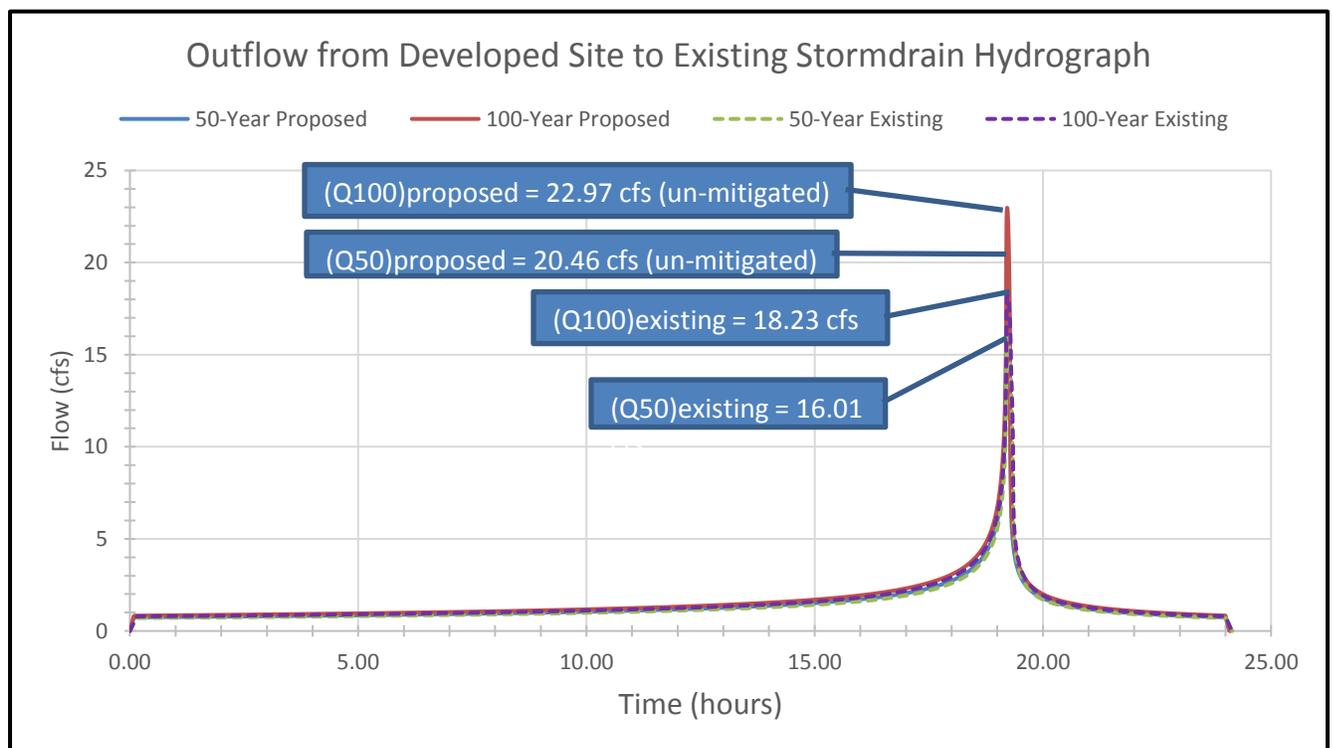
Figure 6. Existing vs. Proposed Conditions Outflow from Site (Un-mitigated)

Table 3 summarizes the peak flows entering the existing storm drain system under Los Angeles Street for existing and proposed conditions hydrology.

Table 3. Peak Flow Results

Condition	Peak Q50 (cfs)	Peak Q100 (cfs)
Existing	16.01	18.23
Proposed	20.46	22.97

For individual sub-area peak flow rates, see appendix A.

Detention Storage

The following table (Table 4) summarizes the detention volume needed for the four mitigation criteria considered: limiting proposed ("developed") conditions Q100 to existing conditions Q100; limiting proposed conditions Q50 to existing conditions Q50; limiting proposed conditions Q100 to proposed conditions Q50; and limiting proposed conditions Q100 to existing conditions Q50.

Table 4. Detention Volume Results

Mitigation Criteria	Detention Volume (cubic feet)
Q50 proposed to Q50 existing	927
Q100 proposed to Q100 existing	971
Q100 proposed to Q50 proposed	495
Q100 proposed - Q50 existing	**1702**

*For calculations of the detention volumes, refer to Appendix A.

Given that the proposed development provides two drainage paths to the existing storm drain system under Los Angeles Street, two detention chambers at two different locations are proposed. For sizing these detention chambers, a linear scaling of the total detention volume was utilized. Attachment 2 shows how the above detention volume is provided for the development.

Stormwater Quality Design Volume and Flow (SWQDV and SWQDF)

In accordance with the County Public Works Department Low Impact Development Manual (2014), the SWQDV shall be the greater of the runoff volume produced from either ¾" inch or 85th percentile rainfall events.

HydroCalc was utilized for obtaining the runoff volumes, using the previous inputs with the design storm frequency set to the ¾" and 85th percentile rainfall events. Individual HydroCalc runs were performed for each sub-area for the two storm events in consideration and the runoff volumes were obtained. Adding up the runoff volumes for each sub-area, the total SWQDV was determined to be 18,511 cubic feet and the SWQDF was 1.659 cfs. Given that the proposed site provides two drainage paths to the existing storm drain system under Los Angeles Street, the stormwater control is provided at two locations: at the eastern side (A-type sub-areas) and the western side (B-type sub-areas). The SWQDV for the eastern side is 9,570 cubic feet and the SWQDF is 0.87 cfs. The SWQDV for the eastern side is 8,941 cubic feet and the SWQDF is 0.79 cfs.

374 Poli St., Suite 207, Ventura, California, 93001   TEL. (805) 798-7664   www.NGenEng.com
Mail: 5142 E. Holmes St., Tucson, AZ 85711

8
F-60

For individual sub-area runoff volumes, flow, and total SWQDV, see Appendix B.

Proposed stormwater quality control measures will be further addressed by The G4 Group.

Conclusions and Discussion

As shown in Table 4, the most constraining mitigation volume is 1,702 cubic feet, corresponding to limiting the proposed conditions Q100 to the existing conditions Q50. This mitigation volume was obtained assuming that the hydrographs for all sub-areas peaked simultaneously. Hence, this volume is a conservative quantity, leading to a conservative design for detention.

As shown in Attachment 2, the development provides two detention chambers with a total storage capacity of 1,861 cubic feet, approximately 459 cubic feet more than the required detention storage of 1,702 cubic feet.

With the addition of the above detention storage, it is ensured that, for any given storm frequency, the outflow from the site under proposed conditions is less than that of the existing conditions. In addition, the nuisance flow to Big Dalton Wash under existing conditions is eliminated in the proposed conditions.

To ensure that the detention provides the desired flow mitigation, an appropriate outflow control is needed at the outflow locations. As such, flow restriction orifices are provided at the two outflow culverts; designed to limit the proposed Q100 to the existing Q50. Appendix C contains the calculations for the sizing of the orifice and the outflow culverts. It was determined that the required outflow configurations consist of a 17.9" diameter orifice on the existing half-filled 36" diameter culvert for the eastern outflow and a 6.6" diameter orifice on an 8" diameter culvert for the western outflow.

References

Los Angeles County Department of Public Works, (2006). *Hydrology Manual*. Public Works Department, County of Los Angeles California.

Los Angeles County Department of Public Works, (2014). *Low Impact Development Standards Manual*. Public Works Department, County of Los Angeles California.

Los Angeles County GIS. *LA County Hydrology Map*. https://dpw.lacounty.gov/wrd/hydrologygis/. Accessed 10/29/20. Public Works Department, County of Los Angeles California.

Los Angeles County GIS, (2013). *GIS-NET Public*. https://rpgis.isd.lacounty.gov/Html5Viewer/index.html?viewer=GISNET_Public.GIS-NET_Public. Accessed 10/29/20. Department of Regional Planning, County of Los Angeles California.

King, H.W. & Brater, E.F. Handbook of Hydraulics (1996). McGraw-Hill Company.

Jerome M. Norman et al. Culverts – Hydrology & Hydraulics (1980). Lehigh Univeristy.

374 Poli St., Suite 207, Ventura, California, 93001   TEL. (805) 798-7664   www.NGenEng.com
Mail: 5142 E. Holmes St., Tucson, AZ 85711

10
F-62

Appendix A: Detention Volume Calculations and Individual Sub-Area Flow Rates

The following table only shows the outflow rates for the time period where volume mitigation is needed (utilizing the peak flow rates in Table 3 and mitigation goals listein in Table 4).

Time (minutes)	Outflow (cfs)			
	50-Year Proposed	100-Year Proposed	50-yr Existing	100-yr Existing
1151.4	13.616	15.290	11.330	12.869
1151.6	14.394	16.165	11.847	13.462
1151.8	15.490	17.397	12.559	14.280
1152.0	18.417	20.677	14.379	16.391
1152.2	19.754	22.179	15.243	17.389
1152.4	20.127	22.598	15.524	17.709
1152.6	20.325	22.820	15.699	17.908
1152.8	20.426	22.934	15.818	18.041
1153.0	20.461	22.973	15.901	18.132
1153.2	20.446	22.956	15.955	18.190
1153.4	20.390	22.893	15.989	18.223
1153.6	20.297	22.789	16.005	18.234
1153.8	20.173	22.650	16.005	18.228
1154.0	20.018	22.476	15.993	18.207
1154.2	19.835	22.270	15.968	18.171
1154.4	19.623	22.032	15.933	18.121
1154.6	19.383	21.762	15.887	18.060
1154.8	19.113	21.458	15.832	17.986
1155.0	18.811	21.120	15.768	17.901
1155.2	18.477	20.744	15.695	17.806
1155.4	18.105	20.326	15.613	17.699
1155.6	17.692	19.862	15.523	17.581
1155.8	17.229	19.344	15.424	17.453
1156.0	16.707	18.760	15.318	17.313
1156.2	16.112	18.094	15.202	17.161
1156.4	15.418	17.315	15.078	16.996
1156.6	14.575	16.368	14.945	16.818
1156.8	13.460	15.114	14.803	16.623

The following table shows the incremental volume calculations during the time period where mitigation is required (proposed conditions flow rate is greater than the peak existing conditions flow rate for respective rain events) and the corresponding total detention volume. For the full table, see the separate attached excel sheet.

Time (minutes)	Incremental Detention Volume (cubic feet)			
	Q50 prop - Q50 ex	Q100 prop - Q100 ex	Q100 prop - Q50 prop	Q100 prop - Q50 ex
1151.4	0	0	0	0
1151.6	0	0	0	51.818
1151.8	0	0	0	58.061
1152.0	48.448	51.425	27.119	75.566
1152.2	54.122	57.474	29.099	83.222
1152.4	55.244	58.667	29.653	84.897
1152.6	55.509	58.947	29.946	85.455
1152.8	55.289	58.708	30.096	85.384
1153.0	54.724	58.099	30.146	84.870
1153.2	53.885	57.200	30.125	84.010
1153.4	52.807	56.045	30.041	82.848
1153.6	51.512	54.661	29.905	81.416
1153.8	50.010	53.056	29.720	79.730
1154.0	48.307	51.234	29.491	77.798
1154.2	46.401	49.193	29.218	75.620
1154.4	44.285	46.928	28.904	73.189
1154.6	41.946	44.425	28.547	70.494
1154.8	39.367	41.664	28.147	67.514
1155.0	36.524	38.619	27.700	64.225
1155.2	33.385	35.256	27.204	60.589
1155.4	29.907	31.528	0	56.561
1155.6	26.031	27.371	0	52.073
1155.8	21.655	22.696	0	47.035
1156.0	16.678	17.373	0	41.313
1156.2	10.920	0	0	34.705
1156.4	0	0	0	26.839
1156.6	0	0	0	17.069
1156.8	0	0	0	0
Sum ∑ =	927	971	495	1702

The following table shows the individual sub-area flow rates for the 100-year proposed conditions rain event to be used for design of on-site stormdrain pipe.

Sub-Area	Proposed Q100 (cfs)
A-1	0.71
A-2	0.91
A-3	4.77
A-4	5.28
B-4	1.79
B-5	3.28
B-6	6.24

Design of on-site stormdrain pipes will be covered by The G4 Group.

Appendix B: SWQDV-F HydroCalc Results

374 Poli St., Suite 207, Ventura, California, 93001   TEL. (805) 798-7664   www.NGenEng.com
Mail: 5142 E. Holmes St., Tucson, AZ 85711

15
F-67

Design Storms for SWQDV

3/4" Storm	0.75	inch	
85th Percentile	1.05	inch	(see Figure below)

HydroCalc Results for Stormwater Quality

Subarea (ID)	3/4" Storm SWQDV (cubic feet)	85th Precentile SWQDV (cubic feet)	85th Precentile SWQDF (cfs)
A-1	373.4	523.2	0.064
A-2	494.7	692.6	0.066
A-3	3037.5	4252.5	0.358
A-4	2929.8	4101.6	0.382
B-4	1044.9	1462.9	0.140
B-5	1849.8	2593.2	0.227
B-6	3489.5	4885.3	0.422
Sub-Group A	6835	**9570**	**0.870**
Sub-Group B	6384	**8941**	**0.789**
Total	13220	**18511**	**1.659**

In accordance to Los Angeles County Pub Public Works Department Low Impact Development Manual (2014), the greater SWQDV should be used for design. The proposed site is divided into two infiltration areas, sub-group A (east) and sub-group B (west). **The Stormwater Quality Design Volumes is 9,570 cubic feet for sub-group A and 8,941 cubic feet for sub-group B.**
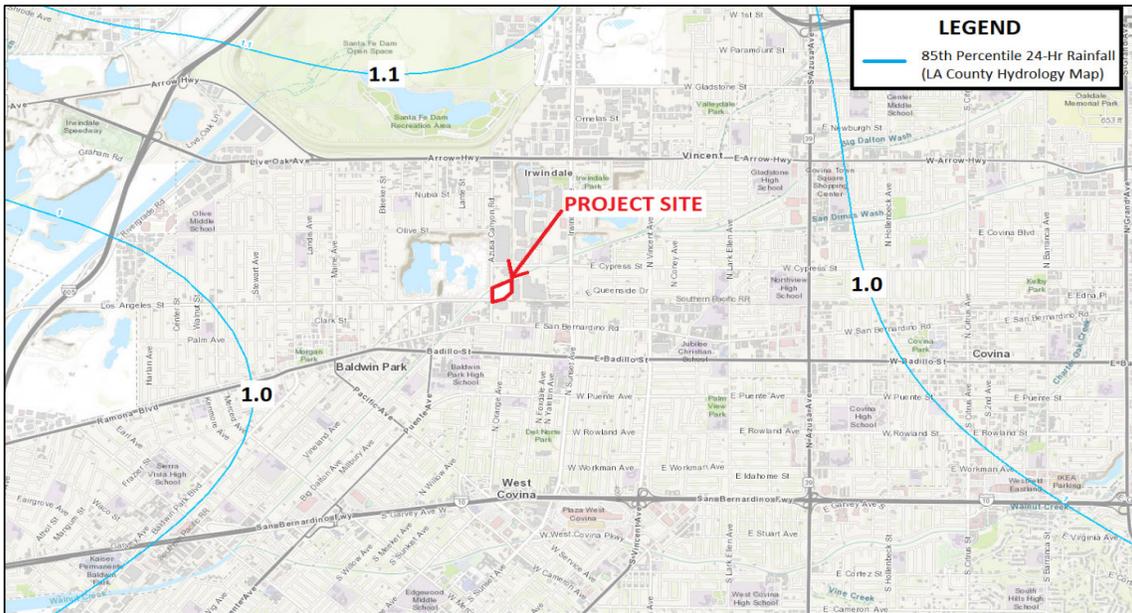
85th Percentile Rainfall Contours

Appendix C: Outflow Culvert and Orifice Restrictor Calculation Sheet

374 Poli St., Suite 207, Ventura, California, 93001   TEL. (805) 798-7664   www.NGenEng.com
Mail: 5142 E. Holmes St., Tucson, AZ 85711

17
F-69

This section describes the design of the flow restriction orifices and outflow culverts provided at the outflow catch basins to ensure that the peak discharge from the proposed conditions site is mitigated adequately.

Basis for Design

The flow restriction orifice will be designed assuming worts-case scenario. This scenario is assumed to correspond to the 100-year event, limiting the proposed conditions Q100 to the existing conditions Q50. The proposed conditions site discharges to the existing stormdrain pipe under Los Angeles street at two locations as shown in Figure 1. Both of these locations will be provided with a flow restriction orifice for controlling the peak discharge from the proposed conditions site.
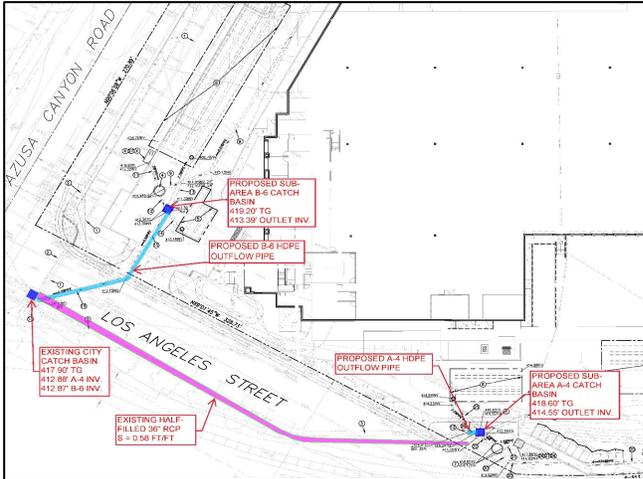

Figure 1. Plan View of Outflow from Developed Site

The following lists the assumptions used for the design of the flow restriction orifices.

1) The basis for design of the outflow restrictor orifice is to restrict the proposed peak Q100 to the existing peak Q50 per CEQA Peer Review Comments.

2) During the worst-case scenario, the proposed on-site catch basins are nearly inundated (water level nearly at top of catch basin, however, gravity-fed flow).

3) Due to lack of data/as-built information on the HGL within the existing City catch basin, it assumed that the City catch basin is nearly inundated (water level nearly at top of catch basin, however, gravity-fed flow).

4) The impact of discharge from sub-area A-4 (east side) does not imact the calculations for discharge from B-6 (west side).

Design

The flow through a circular orifice can be computed as follows using Brater and King Handbook of Hydraulics.

$$Q = Ca(2gh)^{0.5}$$

Where;

        C - discharge coefficient (0.60 recommended typical)
        a - submemged area (full circular area given full submergence during wors-case scenario)
        g - gravitational constant (32.2 ft/s^2)
        h - effective head (ft)

For fully submerged vertical orifice, the effective head is given by:

$$h = H - \max(r, TW)$$

Where;

        H - head above invert (ft)
        r - radius of orifice (ft)
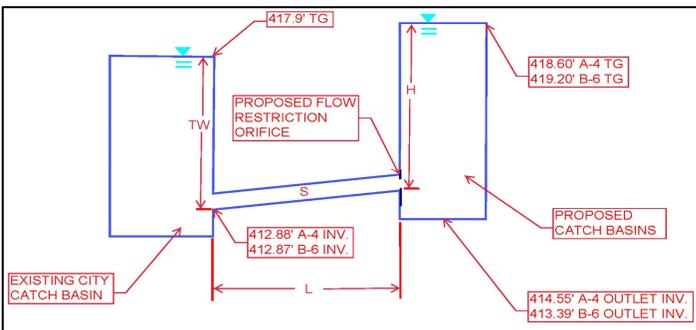   TW - tailwater depth above outlet invert of orifice (ft)

Figure 2. Typical Setup for Outflow from Developed Site

The following lists key variables used in flow calculations.

TW = City Catch Basin TG - Outlet Inv.

Sub-area A-4 (East Outflow)			
TW =	0.7	ft	
H =	4.05	ft	
Orifice:	**r =**	**17.9**	**in**
		1.49	ft
	a =	1.75	ft^2
	h =	2.56	ft

Sub-area B-6 (West Outflow)		
TW =	1.3	ft
H =	5.81	ft
Orifice:	**r (in) =**	**6.6**
	r (ft) =	0.55
	a (ft^2) =	0.24
	h (ft) =	4.51

Using the above parameters, the peak discharge through the orifices are:

Sub-area	Peak Discharge (cfs)	
	Orifice Q$_{100}$	Existing Q$_{50}$
A-4 (East Outflow)	13.46	13.56
B-6 (West Outflow)	2.43	2.45

*Existing peak discharges are calculated using pro-ration of the overall existing peak Q50 of 16.01 cfs.

As shown in the table above, the peak outflows from the developed site during the 100-year storm event is lower than the existing conditions peak outflows.

<u>Check Outflow Controls</u>
The orifice is provided as an inlet control for the outflow culvert. The actual peak discharge from the site will be the lesser of the discharges from either the orifice or the outflow culvert. The following provides the required check.

The flow through a circular culvert whose inlet and outlet are submerged (outlet control) can be computed as follows using Culverts - Hydrology and Hydraulics by Jerome Norman:

$$V^2 = \frac{H - D + SL}{\frac{Ke + 1}{2g} + \frac{n^2 L}{CR^{\frac{4}{3}}}}$$

Where;
V - velocity (ft/s)
H - head above inlet inv. (ft)
D - depth of flow (ft, greater of culvert diameter or TW)
S - slope (ft/ft)
L - length (ft)
Ke - entrance loss coeff (0.2 for recommended typical)
g - gravitational constant (32.2 ft/s^2)
n - manning's coefficient
    n = 0.010 for proposed HDPE
    n = 0.011 for existing concrete
C - conversion factor (2.22 for English units)
R - hydraulic radius (ft)
A - cross-sectional area (ft^2)

For a fully submerged culvert, the hydraulic radius is given by the following.

$$R = \frac{A}{Wetted\ Perimeter} = \frac{\frac{\pi * Diameter^2}{4}}{\pi * Diameter} = \frac{Diameter}{4}$$

The peak discharge can be computed from the velocity as follows.
$$Q = VA$$

Where;
Q - peak discharge (cfs)

Parameters	Sub-Area		
	A-4 Proposed Culvert	A-4 Existing Culvert	B-6
H (ft)		4.05	5.81
Diameter (ft)		**3**	**0.67**
TW (ft)		0.7	1.3
D (ft)		3	1.3
S (ft/ft)		0.0058	0.005
L (ft)	Existing culvert will be	276.4	102.5
n	kept in place	0.011	0.010
R (ft)		0.75	0.325
A (ft^2)		7.07	1.33
V (ft/s)		8.07	11.31
Q (cfs)		28.52	15.01

*Q for the existing culvert on A-4 consider 50% capacity (half-filled).

The following table summarizes the calculateed flows and compares with existing flows.
.

Sub-area	Peak Discharge (cfs)		
	Culvert Q_{100}	Orifice Q_{100}	Existing Q_{50}
A-4 (East Outflow)	28.52	13.46	15.42
B-6 (West Outflow)	15.01	2.43	2.81

As shown in the table above, the peak discharges in the culverts are greater than the orifice peak discharges. **Hence, orifice controls outflow.**

Summary

For the outflow from the developed site to be adequately mitigated, the outflow system must be composed of the following:
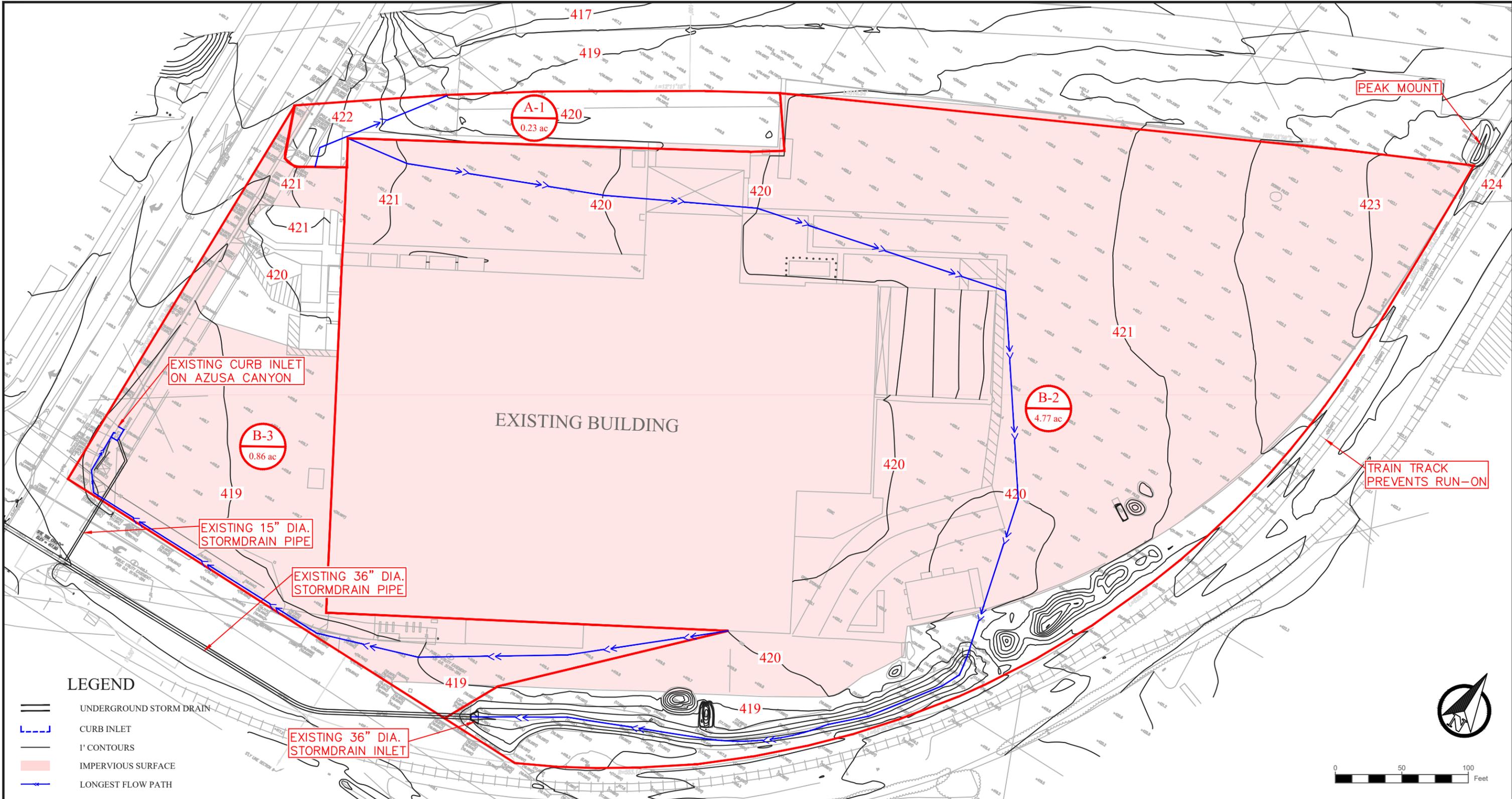
Sub-area A-4 (East Outflow):   **36" diameter culvert** (half-filled) @ 0.58% slope with a
17.9" diameter orifice placed at the culvert invert.

Sub-area B-6 (West Outflow):   **8" diameter culvert** @ 0.5% slope with a
6.6" diameter orifice placed at the culvert invert.

The above outflow configuration ensures that the proposed Q100 peak discharge is not greater than the existing Q50 peak discharge.
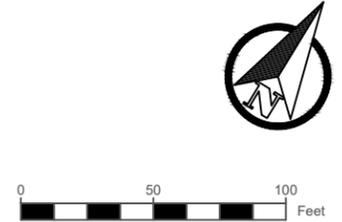
Attachment 1: Hydrology Maps
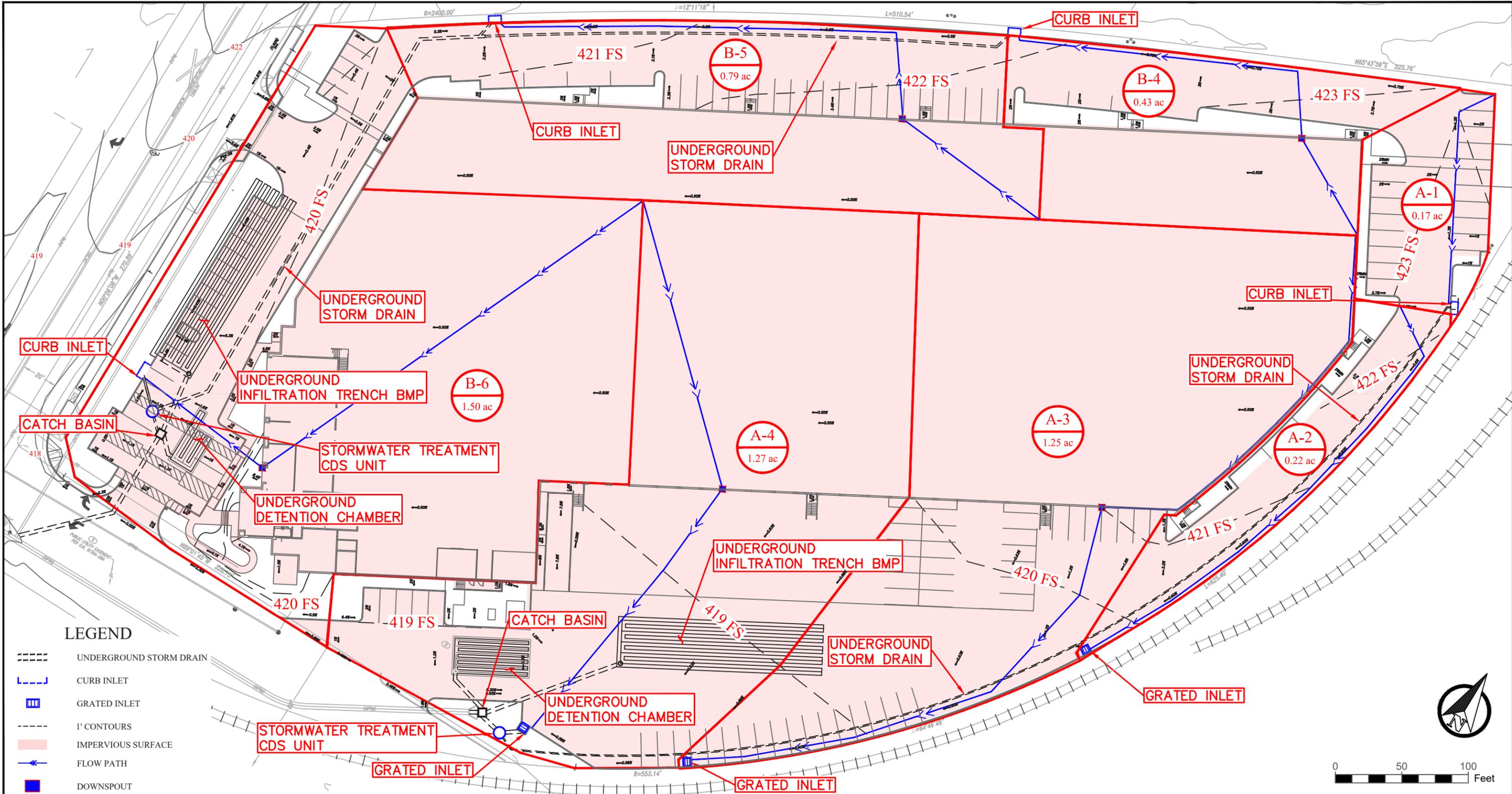
Exhibit 1. Existing Conditions Hydrology Map

PEAK MOUNT

A-1
0.23 ac

EXISTING CURB INLET
ON AZUSA CANYON

B-3
0.86 ac

B-2
4.77 ac

EXISTING BUILDING

TRAIN TRACK
PREVENTS RUN—ON

EXISTING 15" DIA.
STORMDRAIN PIPE

EXISTING 36" DIA.
STORMDRAIN PIPE

EXISTING 36" DIA.
STORMDRAIN INLET

LEGEND

	UNDERGROUND STORM DRAIN
	CURB INLET
	1' CONTOURS
	IMPERVIOUS SURFACE
	LONGEST FLOW PATH

0   50   100 Feet

D		PREPARED BY:	2424 E. Broadway Blvd., Suite 200 Tucson, AZ 85719 (520) 561-6467 ngeneng.com	DESIGNED BY: Tariq Tariq	PROJECT LOCATION: 4416 AZUSA CANYON ROAD IIRWINDALE, CA 91706 APN: 8414004006	SPEC. NO.	IRWINDALE DRAINAGE	SHEET 1
C								OF 2
B		NextGen ENGINEERING		DRAWN BY: Tariq Tariq		PROJ. NO. 87-004-2020	HYDROLOGY MAP - EXISTING	
A				REVIEWED BY: William O'Brien				DRAWING NO.
Δ								

U:\87-004-2020 G4 Group - Irwindale Property\AutoCad\HydrologyMapping_existing.dwg

Exhibit 2. Proposed Conditions Hydrology Map

CURB INLET

B-5
0.79 ac

421 FS

422 FS

B-4
0.43 ac

.423 FS

CURB INLET

UNDERGROUND STORM DRAIN

A-1
0.17 ac

420 FS

UNDERGROUND STORM DRAIN

423 FS

CURB INLET

CURB INLET

UNDERGROUND INFILTRATION TRENCH BMP

B-6
1.50 ac

UNDERGROUND STORM DRAIN

422 FS

CATCH BASIN

A-4
1.27 ac

A-3
1.25 ac

A-2
0.22 ac

STORMWATER TREATMENT CDS UNIT

UNDERGROUND DETENTION CHAMBER

UNDERGROUND INFILTRATION TRENCH BMP

421 FS

420 FS

420 FS

419 FS

CATCH BASIN

419 FS

UNDERGROUND STORM DRAIN

GRATED INLET

LEGEND

	UNDERGROUND STORM DRAIN
	CURB INLET
	GRATED INLET
	1' CONTOURS
	IMPERVIOUS SURFACE
	FLOW PATH
	DOWNSPOUT

STORMWATER TREATMENT CDS UNIT

UNDERGROUND DETENTION CHAMBER

GRATED INLET

GRATED INLET

0   50   100
Feet

D
C
B
A
△

U:\87-004-2020 G4 Group - Irwindale Property\AutoCad\HydrologyMapping_proposed_9-21-21.dwg

F-75

Attachment 2: Underground Detention Chamber Details

DETENTION STORAGE CAPACITY CALCULATIONS

Total Required Site Storm Water Detention Volume = 1,702 cubic feet

Southern Outlet Detention Storage Design

Percentage of Site Drainage Directed to Southern Outlet = 49%

49% of total required volume = 834 cubic feet.

Use 24" C.M.P. = 3.14 C.F./L.F.

Check Proposed Pipe Capacity:

Use 7 rows of 24" C.M.P. x 40 L.F. = 280 L.F.
With ½ Header Pipe 23 L.F.

280 x 3.14 = 879.2 C.F.
23 x 3.14 / 2 = 36.1 C.F.

Total provided Storage = 915.3 C.F.

915.3 > 834 System OK

Western Outlet Detention Storage Design

Percentage of Site Drainage Directed to Southern Outlet = 51%

51% of total required volume = 868 cubic feet.

Use 42" C.M.P. = 9.6 C.F/L.F.

Check Proposed Pipe Capacity:

Use 2 row of 42" C.M.P. x 30 L.F. + 1 rows of 42" C.M.P. x 20 L.F. = 80 L.F.
With ½ Header Pipe 13.5 L.F.

80 x 9.6 = 768 C.F
13.5 x 9.6 / 2 = 68.8 C.F.

Detention Storage Fill Pipe
34.9 LF od 24" HDPE pipe x 3.14 C.F./L.F. = 109.6 C.F.

Total provided Storage = 946.4 C.F.

946.4 > 868 System OK