CEQA LEVEL
HYDROLOGY & HYDRAULICS
REPORT

PROJECT:

26501 MADISON AVENUE
Murrieta, CA
APN 910-230-003

PREPARED FOR:

Todd Sheller
W. M. Lyles Co.
1210 W. Olive Avenue
Fresno, CA 93728

PREPARED BY:

Kristin L. Greene, P.E.
dk GREENE CONSULTING, INC.
P.O. Box 143
Bonsall, CA 92003
J.N. 19-23

I hereby declare that I am the engineer of work for this project, that I have exercised responsible charge over the design of the project as defined in Section 6703 of the Business and Professions code, and that the design is consistent with current standards.

REGISTERED PROFESSIONAL ENGINEER
KRISTIN LIPSKA GREENE
NO. C57860
CIVIL
STATE OF CALIFORNIA

EXPIRES: JUNE 30, 2022

dk Greene Consulting, Inc.

Kristin L. Greene, P.E. C57860

October 20, 2020
Date

Contents

LIST OF FIGURES

0.0 Declaration of Responsible Charge

I hereby declare that I am the Engineer-of-work for this project, that I have exercised responsible charge over the design of this project as defined in Section 6703 of the Business and Professions Code, and that the design is consistent with current standards.

I understand that the check of project drawings and specifications by the County of San Diego is confined to a review only and does not relieve me, as Engineer-of-work, of my responsibilities for project design.

10-20-2020

Kristin L. Greene, PE   C57860   Date
Exp. 6/30/22

REGISTERED PROFESSIONAL ENGINEER
KRISTIN LIPSKA GREENE
NO. C57860
CIVIL
STATE OF CALIFORNIA
EXPIRES: JUNE 30, 2022

1.0 Project Information

1.1 Site and Project Description

This 5.38-acre net/gross site is located at 26501 Madison Avenue in Murrieta. The lot is currently undeveloped. Access to the site is currently provided from a dirt road off Madison Avenue.

A portion of the northeasterly section of the site will be dedicated to the City of Murrieta for Madison Avenue road improvement. The southerly portion of the site is located in the Warm Springs Creek 100-year flood zone. This area will remain undisturbed and is not considered as part of this project.

This project proposes to develop a two-story commercial office building, a storage building, a storage yard, access driveways, and a biofiltration basin. The site is surrounded by other commercial buildings of similar size to that being proposed, and some undeveloped lots.

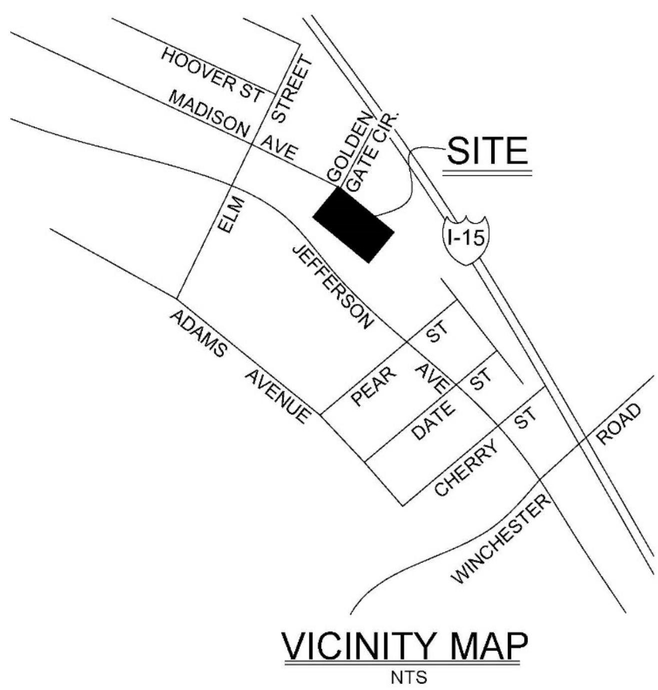


Figure 1-1. Vicinity Map

This report will focus on the hydrology and hydraulics in response to the grading and improvements associated with the development.

This report will evaluate the Q_{10} for the existing condition and compare it to the Q_{10} for the proposed condition using the Modified Rational Method and County of Riverside's Hydrology Manual to evaluate peak flows.

1.2 Existing Site Topography and Drainage Condition

Topography of the site was provided by dk Greene Consulting, Dale Greene, L.S. on June 5, 2019.

In the current pre-development condition, the site has four sub-basins and one outlet (outfall) point. Generally, the entire 5.38-acre net/gross project drains southeasterly toward the southeastern corner of the property, then flows directly into Warm Springs Creek.

The high point of the site is located near the northeast corner of the property. There is a small amount of off-site drainage ("run-on"), which sheet flows southeasterly through the property and exits into Warm Springs Creek.

See Appendix A, Existing Hydrology Map.

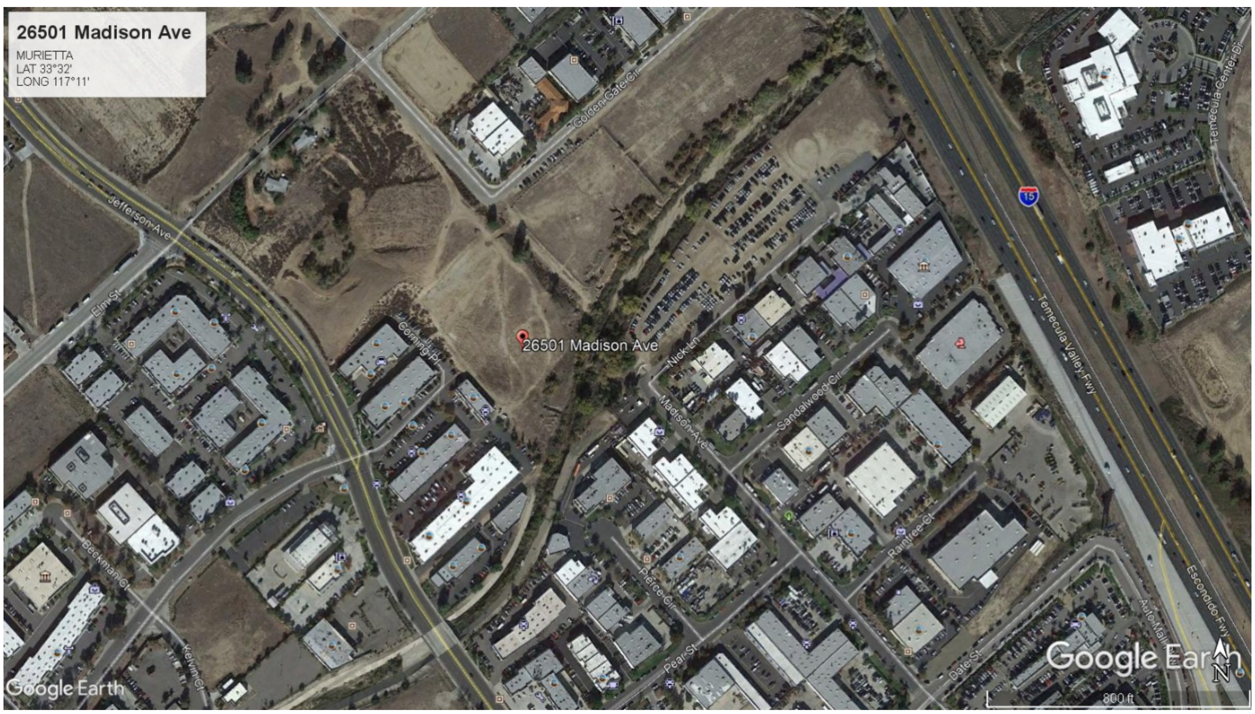


Figure 1-2. Google Earth Plan View Looking Northerly

1.3 Proposed Topography and Drainage Condition

In an effort to minimize grading, this project proposes to create two pads. The office building will be constructed on the upper pad. The storage building and storage yard will be constructed on the lower pad. The total earthwork quantities are:

Earthwork = Cut/Fill 15,000 C.Y

The impervious surface area will increase due to the improvements. The proposed drainage pattern for this site will be generally the same as the existing, historical drainage pattern. The high point will continue to be located near the northeast corner of the property line and the outlet point will continue to be at the southeast corner of the property.

The site will continue to have four sub-basins and one main outfall point. The property will continue to outfall directly into Warm Springs Creek. A summary of the proposed drainage strategy for each sub-basin follows:

Sub-basin ID	Description	Proposed Drainage Strategy
A	Madison Avenue Road Improvements	The road improvements for Madison Avenue will include a berm to prevent off-site run-on from entering the property.
B	Off-site Runon	The offsite runon will be captured in a lined ditch to prevent it from entering the developed property. The runoff will be directed toward the street improvements.
C	Development Area - Upper Pad	The roofs of the commercial office building will drain into the parking spaces (constructed of permeable asphalt), then will sheet flow to the asphalt driveway surrounding the building. The asphalt driveway will be sloped to the perimeter gravel area. Catch basins (18” Brooks Boxes) will be constructed in the gravel areas to receive overflow runoff. The stormwater will then be piped to the Biofiltration Basin, design for hydromodification and pollutant control, then piped directly to Warm Springs Creek.
	Development Area – Lower Pad	The outer driveway, which provides access to the storage building/storage yard, will sheet flow to the gravel area at the southern portion of the site. The roof of the storage building will drain to the gravel area toward the southeast corner of the building.

		Catch Basins will be constructed in this gravel area that will collect the overflow drainage and then pipe it into to the Biofiltration basin, and ultimately be outlet to Warm Springs Creek, which is the historical drainage pattern.
D	Natural Slope	The natural slope will remain and will sheet flow to the creek, as is the existing condition.

For more information, see Appendix A, Proposed Hydrology Map.

2.0 Hydrology and Hydraulic Calculations

2.1 Method of Calculation

The methodology used for the hydrologic and hydraulic analysis for this project is the Modified Rational Method according to the Riverside County Flood Control and Water Conservation District (RCFCWCD). The 10-year storm events will be used in the calculations. CivilD software was used to calculate the peak storm discharge for the drainage areas for the existing and proposed conditions.

Since this site is less than 10 acres, the difference between the existing and the proposed conditions dictate the amount of detention necessary. See Section 2.2 for calculations used to determine the amount of detention required for this project.

Hydraulic analysis of all pipes was conducted using Open Channel Flow Calculator by Lamar University. The Open Channel Flow calculations are based on Manning's Equation.

The soil report has been prepared for this project by LGC Geo-Environmental, Inc., and is dated April 25, 2019.

According to the Soils Map from the NRCS Websoilsurvey online tool, the site is classified as containing soils from Soil Groups A, C, and D. However, because the development occurs in Soil Group C, that soil group has been selected for calculation purposes. See Appendix B, Soils Map.

The "C" value for the existing condition will be based on 0% imperviousness. When the site is fully developed, the imperviousness will increase to approximately 35%.

Detailed "C" values will be used for calculations during the final engineering phase of this development.

2.2 Results of Hydrology Study

The hydrology was performed for this project assuming the following conditions:

SUMMARY OF HYDROLOGIC CONDITIONS

	Sub-area ID	Area (ac.)	Elevation Difference (ft.)	Flow Path Length (ft.)	Flow Path Slope (VHT/HFT)
Existing Condition	A	0.20	26	309	0.08
	B	1.57	9	242	0.04
	C	3.56	27	639	0.04
	D	0.33	33	448	0.07
Proposed Condition	A	0.20	11	295	0.04
	B	0.57	18	351	0.05
	C1-C2	3.56	13	701	0.02
	D	0.33	33	639	0.04

FLOW AND VOLUME SUMMARY FOR EXISTING AND PROPOSED CONDITIONS

	Existing Condition		Proposed Condition			
Sub-area ID	T_c (min.)	10-year Flow (cfs)	T_c (min.)	10-year Flow (cfs)	ΔQ (cfs)	Required Volume (cf)
A	8.41	0.41	5.63	0.58	+0.17	29
B	9.19	1.23	10.0	1.17	0	0
C1-C2	13.22	6.23	6.40	8.65	+2.25	373
D	9.2	0.71	9.2	0.71	0	0

The existing and proposed development conditions 10-year flow rates are shown above. To mitigate the increase in flow created by the impervious surfaces due to the development of the project, a biofiltration basin will be constructed for flow control (hydromodification) and pollutant control compliance. This basin will provide 5,800 c.f. of storage which will also satisfy the increase in hydrologic flow.

Maps of the existing and proposed hydrology are provided in Appendix A. Calculations are provided in Appendix D.

2.3 Results of Hydraulic Analysis

Hydraulic analysis of the pipes was conducted (for the 10-year storm event) using Open Channel Flow Calculator by Lamar University.

The results of the hydraulic analysis indicate that a 15” HDPE pipe is recommended to convey the storm water overflow to the Biofiltration Basin from the upper pad (C1) and the lower pad access driveway (C2).

2.4 Conclusions

This proposed project encompasses the development of a commercial office, storage building, storage yard, and driveway on approximately 5.38 acres net/gross.

One biofiltration basin (PCBMP #1) will be constructed at the southern portion of the development area, near the outlet point. The basin will be sized for hydromodification and pollutant control compliance. The majority of runoff generated by the new development will flow toward the biofiltration basin and then outlet directly into Warm Springs Creek. The pipes and orifice sizes are calculated to allow the runoff to flow at a lower rate than occurs for the existing condition.

This project includes several run-off collection areas and pervious surfaces as follows: landscape areas, gravel areas, and permeable asphalt. These pervious areas will allow for appropriate levels of infiltration and will then sheet flow. Overflow will be safely conveyed to the biofiltration basin.

Because the increase in overall outflow is mitigated via the biofiltration basin, the development of this site will result in a decrease in Q_{10} when compared to the existing condition.

In my professional opinion, the proposed project will not substantially alter the existing drainage pattern of the area. The project has been designed to maintain the historical drainage pattern. With the use of a biofiltration basin and low-impact development features, there will be no runoff to off-site parcels, and therefore no downstream flooding will occur due to the development this project. The proposed improvements will not increase the volume or velocity of surface flows to the detriment of downstream landowners and/or facilities.

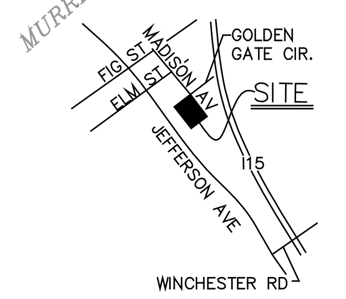
APPENDICES

Appendix A

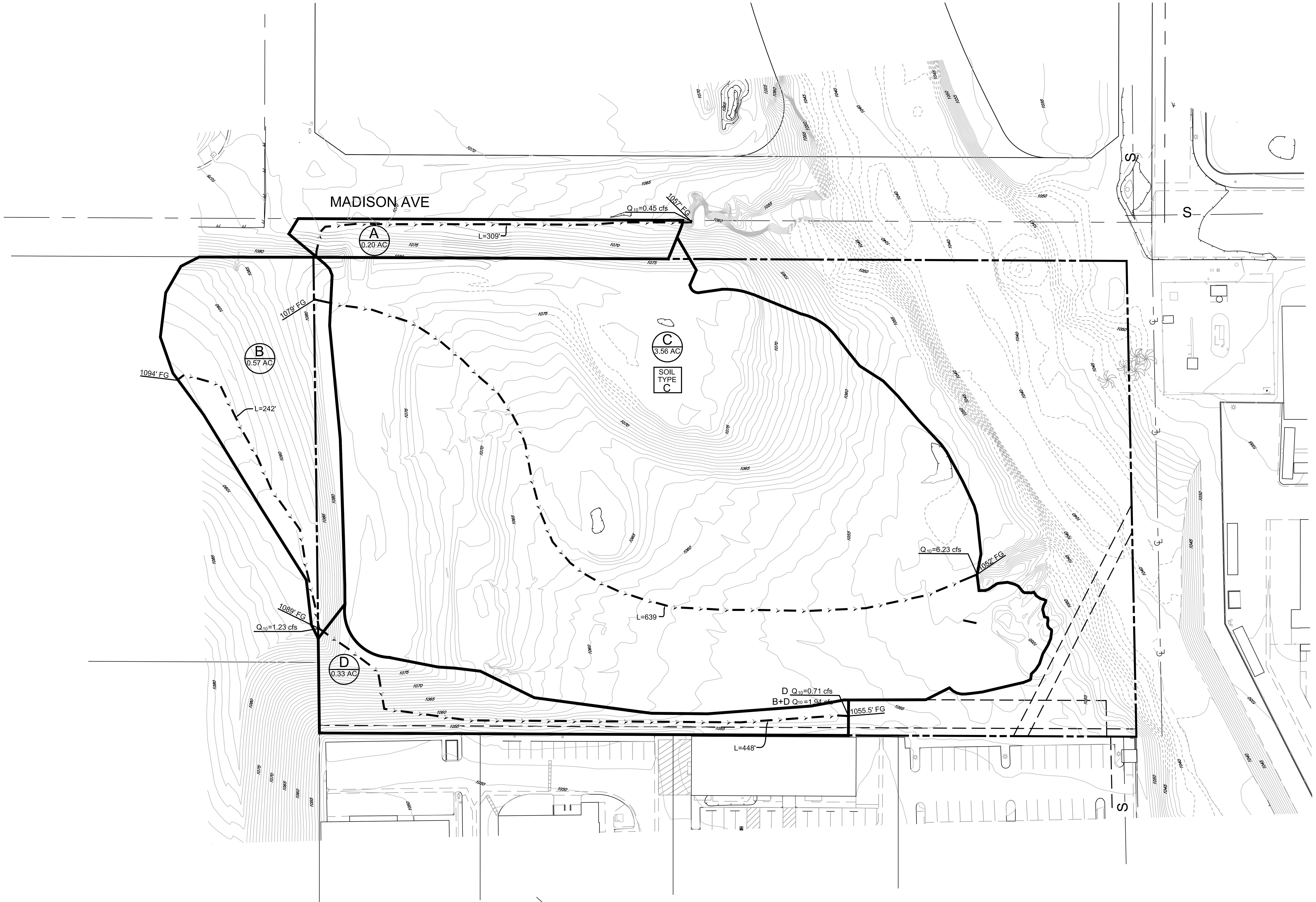
Existing Hydrology Map

Proposed Hydrology Map

EXISTING HYDROLOGY



VICINITY MAP
NTS



DRAINAGE LEGEND

FLOW PATH
DRAINAGE BOUNDARY
PROPERTY LINE
OUTFALL 1 FLOW LENGTH L=215
SUB AREA I.D. (A)
SUB AREA SIZE (IN ACRES) (XX AC)

NOTES:

UNDERLYING HYDROLOGIC SOIL GROUP=C

APPROXIMATE DEPTH TO GROUND WATER=GREATER THAN 20'

PROPERTY OWNER

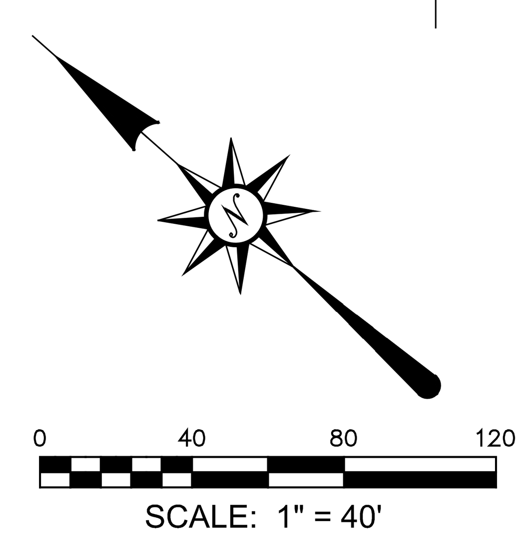
W.M. LYLES CO
1210 WEST OLIVE AVENUE
FRESNO, CA 93728

APN

910-230-003

LEGAL DESCRIPTION

PARCEL 2 OF PARCEL MAP NO. 7065, IN THE CITY OF MURRIETA, COUNTY OF RIVERSIDE, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 26, PAGE 50 OF PARCEL MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.



SEAL: REGISTERED PROFESSIONAL ENGINEER KRISTIN LIPSKA GREENE NO. C57860 CIVIL STATE OF CALIFORNIA

SCALE
HORIZONTAL
1"=40'
VERTICAL
N/A

dk Greene Consulting, Inc.
P.O. BOX 143
BONSALL, CA 92003
(760) 310-9408

PREPARED BY — DATE 10/13/20
KRISTIN L. GREENE
C NO. 57860 EXP. DATE 6/30/22

DATE	INITIAL	REVISION DESCRIPTION	SHT. NO.	DATE	INITIAL
ENGINEER OF WORK				CITY APPROVAL	

SHEET 1 — CITY OF MURRIETA, ENGINEERING DEPARTMENT — SHEETS 1

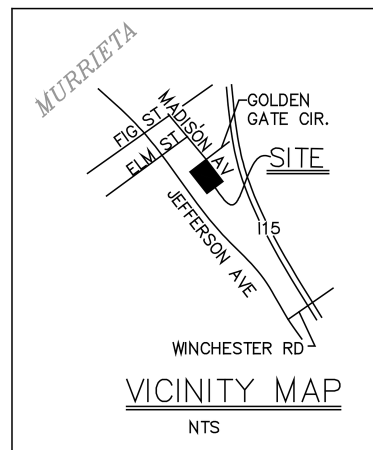
EXISTING HYDROLOGY
W.M. LYLES CO.
26501 MADISON AVE
PARCEL 2, P.M. 7065, BK 26, PG 50

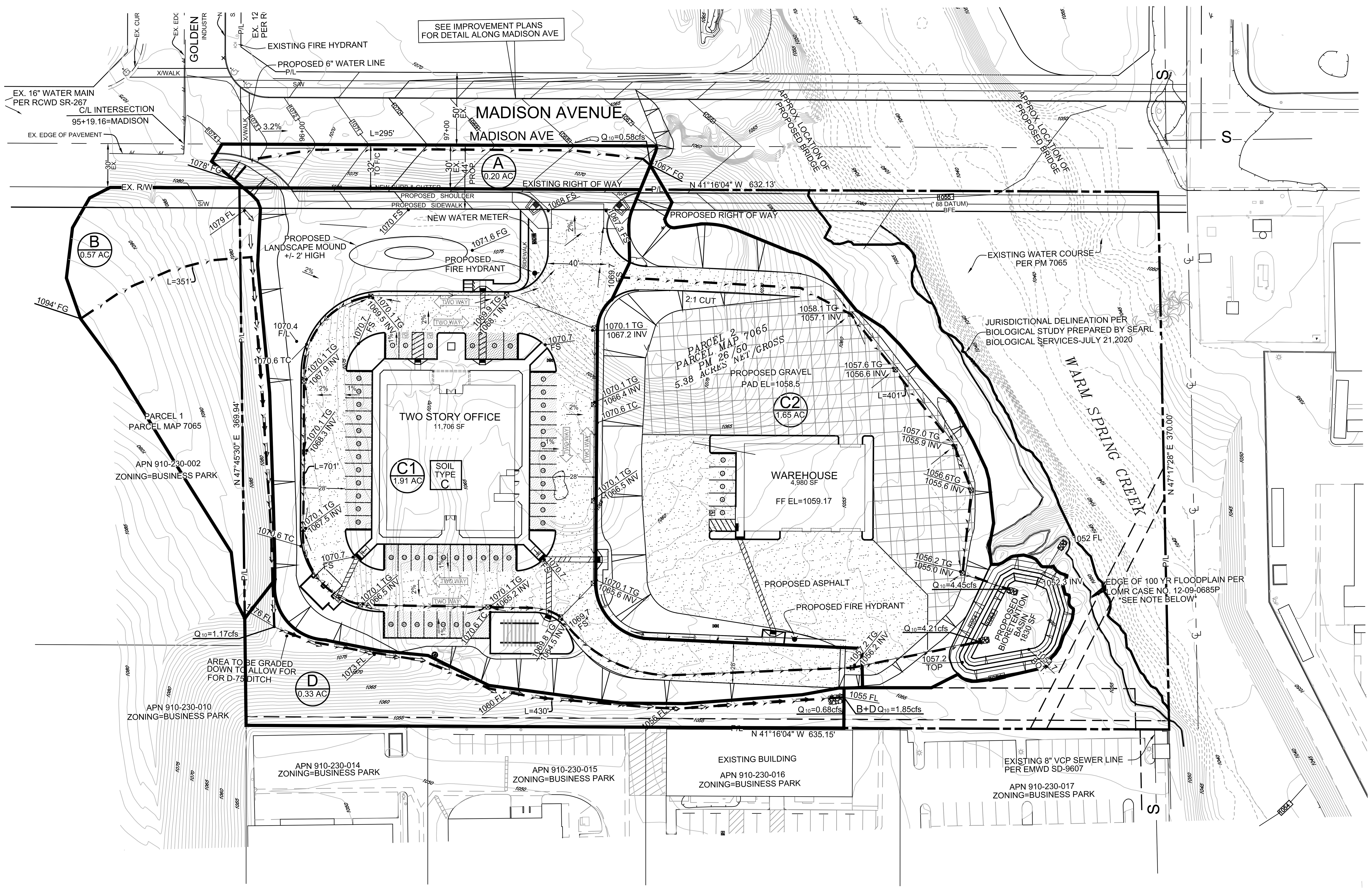
APPROVED
ROBERT K. MOEHLING ________ DATE ______
CITY ENGINEER RCE 63056

DWN BY:
CHKD BY:
FIELD BK:
PROJECT NO.
DRAWING NO.

JOB NO. 19-23

PROPOSED HYDROLOGY





DRAINAGE LEGEND

FLOW PATH	
DRAINAGE BOUNDARY	
PROPERTY LINE	
OUTFALL 1 FLOW LENGTH	L=215
SUB AREA I.D.	A
SUB AREA SIZE (IN ACRES)	XX AC

NOTES:

UNDERLYING HYDROLOGIC SOIL GROUP=C

APPROXIMATE DEPTH TO GROUND WATER=GREATER THAN 20'

PROPERTY OWNER

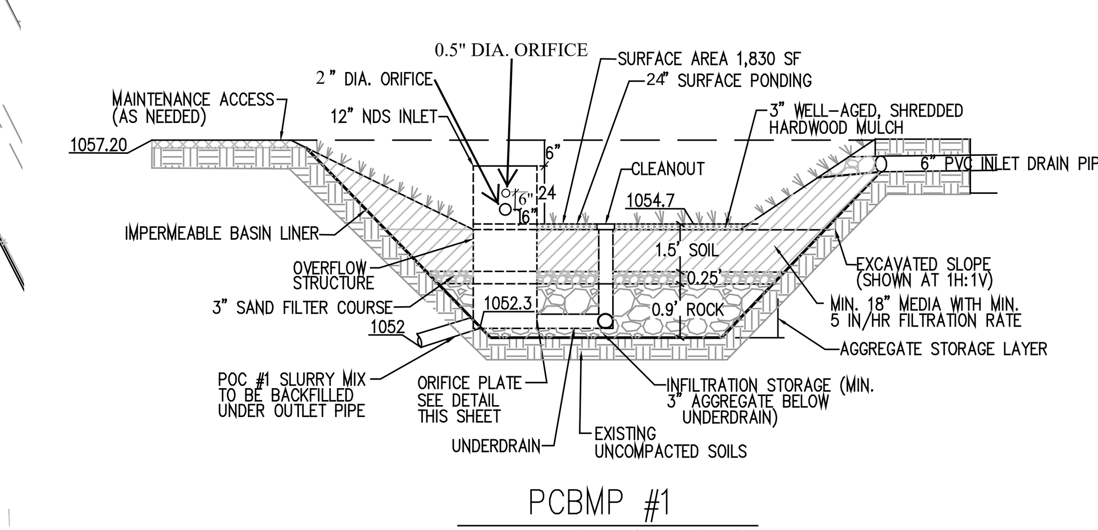
W.M. LYLES CO
1210 WEST OLIVE AVENUE
FRESNO, CA 93728

APN

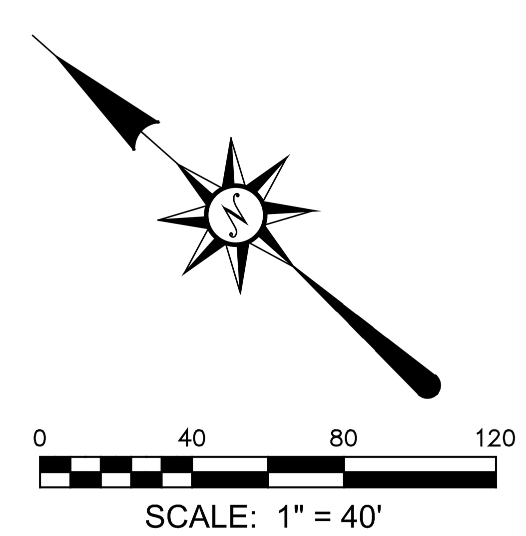
910-230-003

LEGAL DESCRIPTION

PARCEL 2 OF PARCEL MAP NO. 7065, IN THE CITY OF MURRIETA, COUNTY OF RIVERSIDE, STATE OF CALIFORNIA, AS PER MAP RECORDED IN BOOK 26, PAGE 50 OF PARCEL MAPS, IN THE OFFICE OF THE COUNTY RECORDER OF SAID COUNTY.



PCBMP #1
BIOFILTRATION BASIN
NOT TO SCALE



SEAL: REGISTERED PROFESSIONAL ENGINEER KRISTIN LIPSKA GREENE NO. C57860 CIVIL STATE OF CALIFORNIA

dk Greene Consulting, Inc.
P.O. BOX 143
BONSALL, CA 92003
(760) 310-9408

SCALE	
HORIZONTAL	1"=40'
VERTICAL	N/A

PREPARED BY _______ DATE 10/13/20
KRISTIN L. GREENE
C NO. 57860 EXP. DATE 6/30/22

DATE	INITIAL	REVISION DESCRIPTION	SHT. NO.	DATE	INITIAL
ENGINEER OF WORK				CITY APPROVAL	

SHEET 1 — CITY OF MURRIETA, ENGINEERING DEPARTMENT — SHEETS 1

PROPOSED HYDROLOGY
W.M. LYLES CO.
26501 MADISON AVE
PARCEL 2, P.M. 7065, BK 26, PG 50

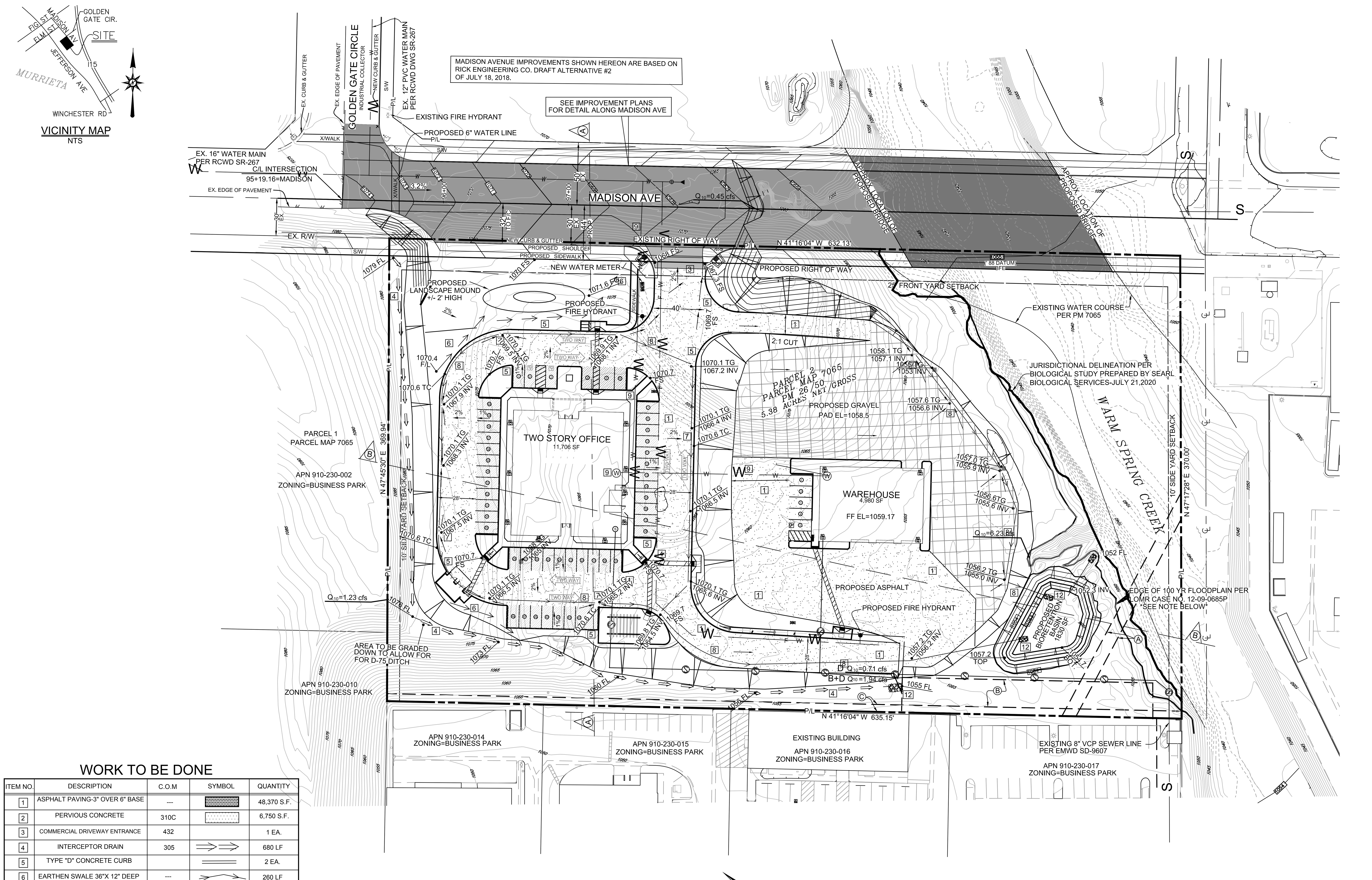
APPROVED
ROBERT K. MOEHLING __________ DATE ______
CITY ENGINEER RCE 63056

DWN BY:
CHKD BY:
FIELD BK:
PROJECT NO.
DRAWING NO.

JOB NO. 19-23

Appendix B

Grading Plan



GENERAL NOTES

APPLICANT
W.M. LYLES CO.
1210 WEST OLIVE AVENUE
FRESNO, CA 93728

LAND OWNER
W.M. LYLES CO.
1210 WEST OLIVE AVENUE
FRESNO, CA 93728

PREPARED BY
dk GREENE CONSULTING, INC.
P. O. BOX 143
BONSALL, CA 92003
KRISTIN GREENE, PE
(760) 310-9408

ASSESSOR PARCEL NUMBER
910-230-003, VACANT LAND

PROJECT DESCRIPTION
PROPOSED 12,000 SF TWO STORY OFFICE BUILDING AND 5,000 SINGLE STORY METAL WAREHOUSE

LEGAL DESCRIPTION
PARCEL 2 OF PARCEL MAP NO. 7065, COUNTY OF RIVERSIDE, STATE OF CALIFORNIA, CITY OF MURRIETA PER BOOK 26, PAGE 50

ZONING
EXISTING=BUSINESS PARK
PROPOSED=BUSINESS PARK
EXISTING USE=VACANT

EARTH WORK QUANTITIES
PROPOSED CUT=15,000 CY
PROPOSED FILL=15,000 CY

UTILITY PURVEYORS
WATER: RCWD METROPOLITAN MUNICIPAL WATER DISTRICT (ANNEXATION IN PROCESS)
SEWER: EASTERN MUNICIPAL WATER DISTRICT
GAS:
ELECTRICITY: SOUTHERN CALIFORNIA EDISON
TELEPHONE:
CABLE TV: N/A

EASEMENT LEGEND

A. A 10' EASEMENT AND RIGHT OF WAY GRANTED TO SOUTHERN CALIFORNIA EDISON COMPANY RECORDED APRIL 28, 1978 IN BOOK 1978 PAGE 84042

B. AN EASEMENT FOR INGRESS AND EGRESS RECORDED JUNE 22, 1990 AS DOCUMENT NO. 231465 OF OFFICIAL RECORDS.

C. A 6' EASEMENT AND RIGHT OF WAY GRANTED TO SOUTHERN CALIOFORNIA EDISON COMPANY RECORDED APRIL 28, 1978 BOOK1978 PAGE 84042

LEGEND

PROPERTY LINE	
EASEMENT	
EXISTING CONTOUR	180
PROPOSED CONTOUR	180
EXISTING SEWER LINE	S
EXISTING WATER LINE	W
DIRECTION OF FLOW	2%
FLOW LINE	
2:1 CUT SLOPE	2:1 CUT
2:1 FILL SLOPE	2:1 FILL
PAD ELEVATION	PAD EL=182.0
FINISH FLOOR ELEVATION	FF EL=183.5
DAYLITE LINE	
FLOWLINE GRADE	190.0 FL
FINISHED SURFACE	189.6 FS
NATURAL GROUND SURFACE	NG
FIRE - WATER SERVICE LINE	F W
TOP OF CURB	TC
TOP OF GRATE	TG
FINISH SURFACE	FS
PROPOSED PERVIOUS CONCRETE -SEE DETAIL ON SHEET 2-	
PROPOSED CONCRETE	
PROPOSED ASPHALT	
PROPOSED GRAVEL	
EDGE OF PAVEMENT	
6" PVC DRAIN PIPE	
18" NDS TRAFFIC RATED BROOKS BOXES	
PROPOSED 6" SEWER LATERAL	
PROPOSED 6" WATER SERVICE	
PROPOSED FIRE HYDRANT	
ROOF DRAIN LOCATIONS	RD

WORK TO BE DONE

ITEM NO.	DESCRIPTION	C.O.M	SYMBOL	QUANTITY
1	ASPHALT PAVING-3" OVER 6" BASE	---		48,370 S.F.
2	PERVIOUS CONCRETE	310C		6,750 S.F.
3	COMMERCIAL DRIVEWAY ENTRANCE	432		1 EA.
4	INTERCEPTOR DRAIN	305		680 LF
5	TYPE "D" CONCRETE CURB			2 EA.
6	EARTHEN SWALE 36"X 12" DEEP	---		260 LF
7	18" NDS TRAFFIC RATED BROOKS BOX	---		18 EA.
8	8" PVC DRAIN PIPE	---		1,210 LF
9	PRIVATE 2" WATER SERVICE	---	W	380 LF
10	6" WATER -FIRE	---	F W	520 LF
11	PRIVATE 6" SEWER LATERAL	---	S	688 LF
12	RIP RAP ENERGY DISSIPATOR	446A		1 EA.
13	GRAVEL SURFACE			23,215 SF
14	NOT USED			
15	NOT USED			
16	NOT USED			
17	NOT USED			
18	NOT USED			
19	NOT USED			

FEMA FLOODPLAIN NOTE
FEMA MAP NUMBER 06065C2720G PANEL 2720 OF 3805, EFF. DATE OF AUGUST 28, 2008 HAS BEEN REVISED PER CASE NO. 12-09-0685P EFF. DATE OF MAY 24, 2013. THIS LOMR PROVIDES THE BASE FLOOD ELEVATION (BFE) AND THE LIMITS OF THE 100 YR FLOOD PLAIN ON THE NAVD '88 VERTICAL DATUM. THIS CONCEPTUAL GRADING PLAN IS ON THE NGVD '29 VERTICAL DATUM. BY USING THE NGS VERTCON SOFTWARE THE APPROXIMATE VERTICAL SHIFT FROM '88 TO '29 IS APPROXIMATELY -2.4'. THE LIMITS OF SAID 100YR FLOODPLAIN SHOWN HEREON IS IS BASED ON THE BFE ELEVATION PER THE LOMR, ADJUSTED BY THE -2.4' AND FOLLOWING THE CONTOURS HEREON.

ADD 2.4' TO '29 DATUM TO OBTAIN '88 DATUM

0 40 80 120

REGISTERED PROFESSIONAL ENGINEER
KRISTIN LIPSKA GREENE
NO. C57860
CIVIL
STATE OF CALIFORNIA

Underground Service Alert
CALL BEFORE YOU DIG
Call: TOLL FREE
1-800
422-4133
TWO WORKING DAYS BEFORE YOU DIG

BENCH MARK
DESCRIPTON: 3 1/4" ALUM. DISC IN TOP OF CURB
LOCATION: E'LY CORNER OF JEFFERSON AVE AND DATE ST. 38' NE'LY OF JEFFERSON AVE AND 65' SEL'Y OF DATE ST.
RECORDED: BENCHMARK NO. "600-10-68"
ELEVATION: 1039.286 DATUM: NGVD '29

APPROVED FOR SIGNATURE
PLAN CHECK ENGR. NAME TYPED
PLAN CHECK FIRM
R.C.E. NO.

HORIZONTAL: AS NOTED
VERTICAL: AS NOTED

dk Greene Consulting, Inc.
P. O. BOX 143
BONSALL, CA 92003

PREPARED BY [signature] DATE 10/13/20
KRISTIN L. GREENE
PE NO. C 57860 EXP. 6/30/2022

DATE	INITIAL	REVISION DESCRIPTION	SHT. NO.	DATE	INITIAL
ENGINEER OF WORK				CITY APPROVAL	

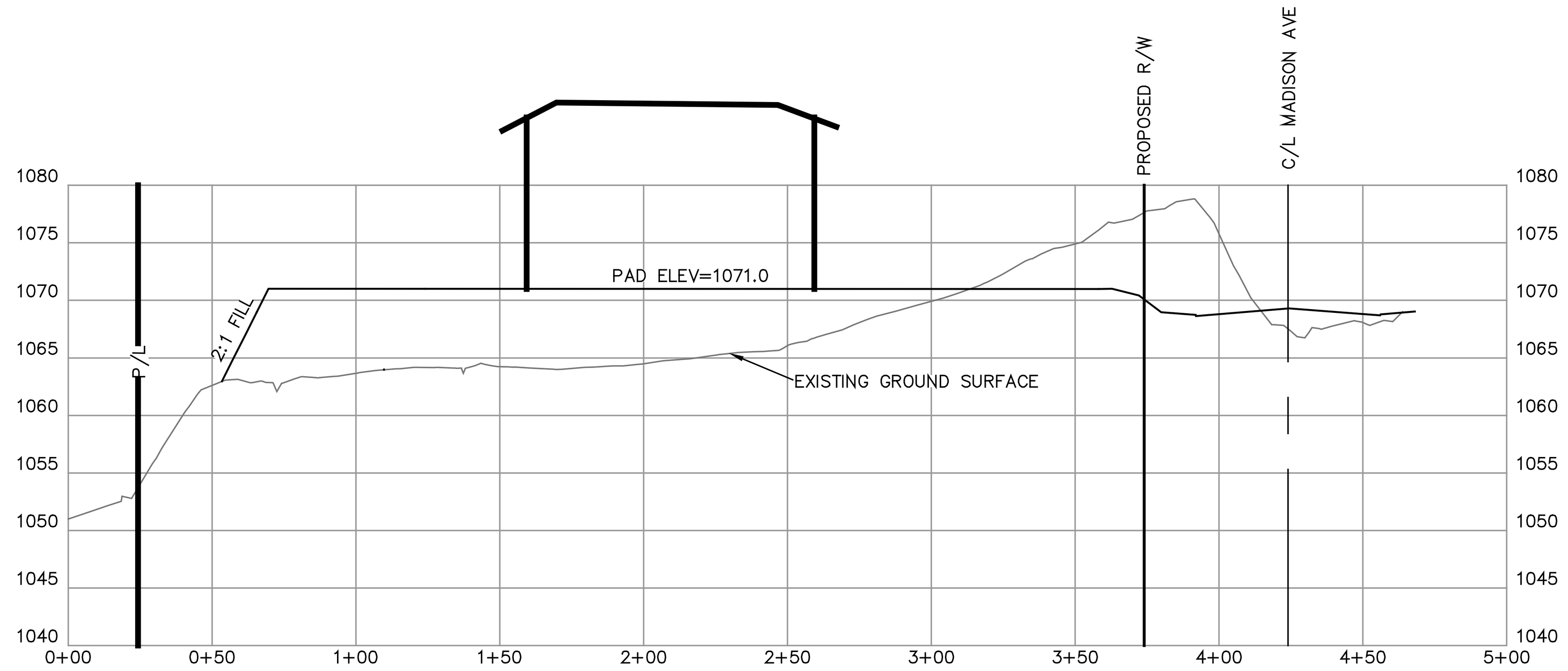
SHEET 1 — CITY OF MURRIETA, ENGINEERING DEPARTMENT — SHEETS 2

CONCEPTUAL GRADING
W.M. LYLES CO.
26501 MADISON AVE
PARCEL 2, P.M. 7065, BK 26, PG 50

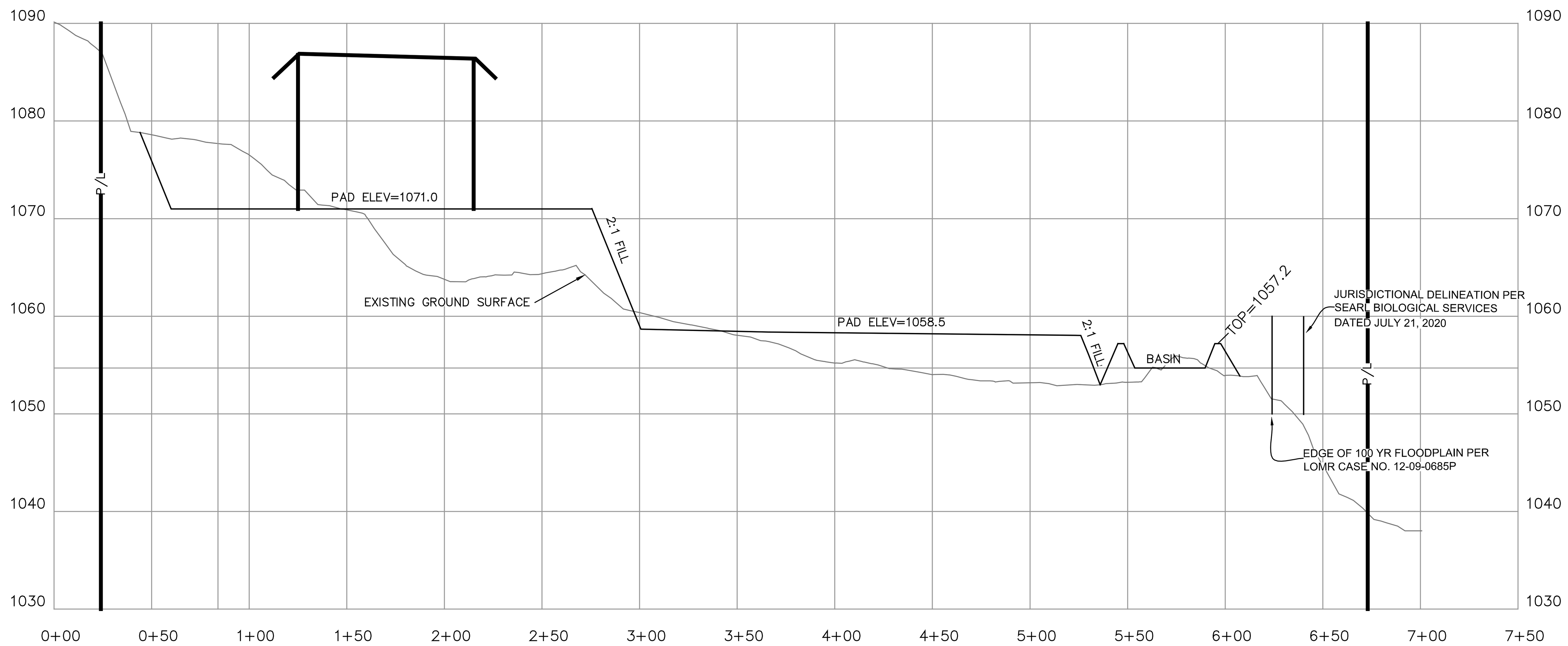
APPROVED
ROBERT K. MOEHLING ____ DATE ____
CITY ENGINEER

DWN BY:
CHKD BY:
FIELD BK:
DRAWING NO.

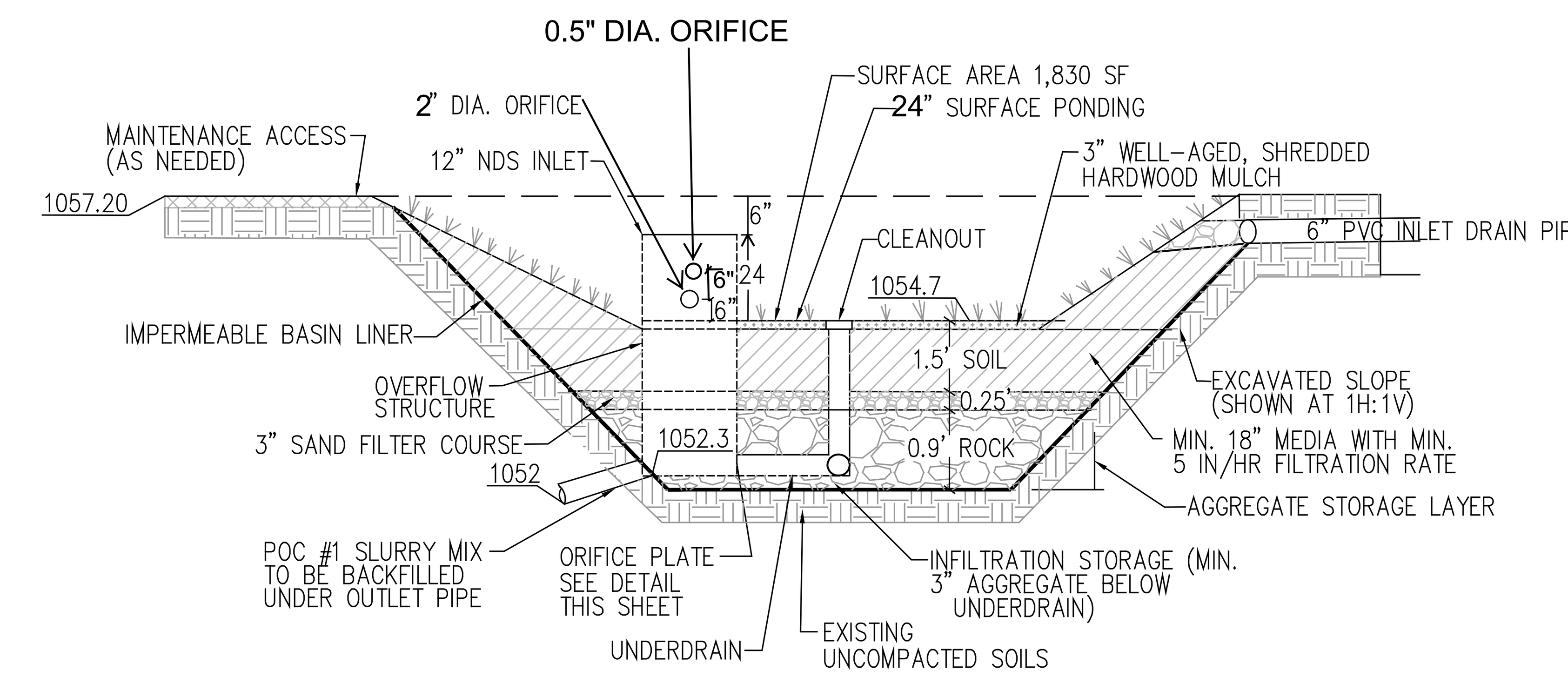
JOB NO. 2019-23 October 14, 2020



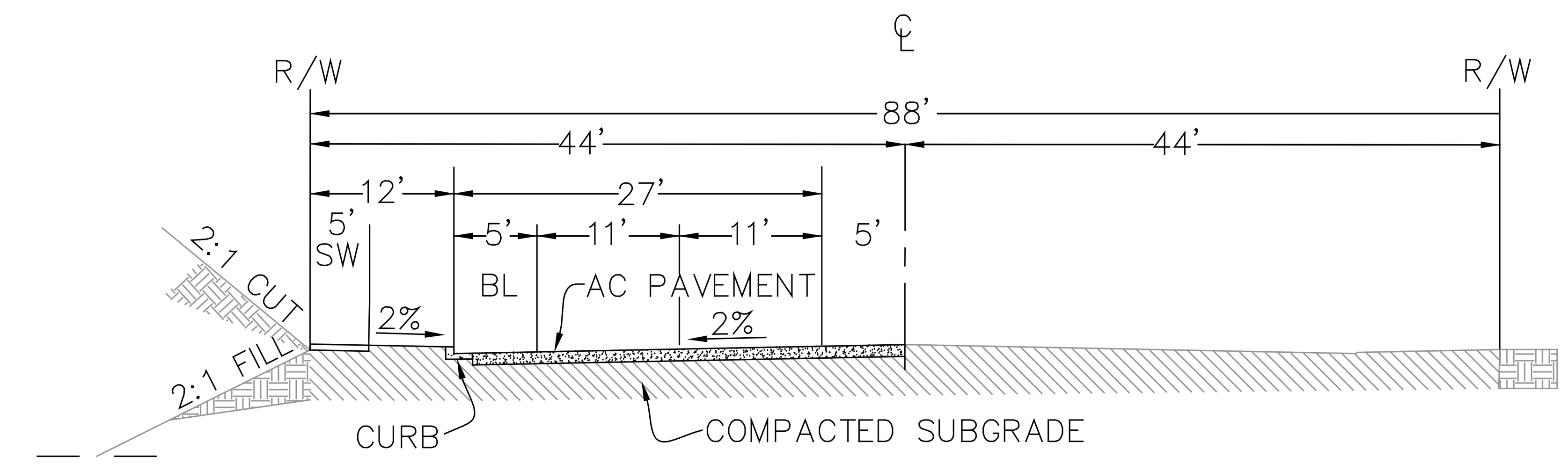
SECTION "A" – "A"
HORIZ. SCALE = 1" = 50'
VERT. SCALE = 1" = 10'



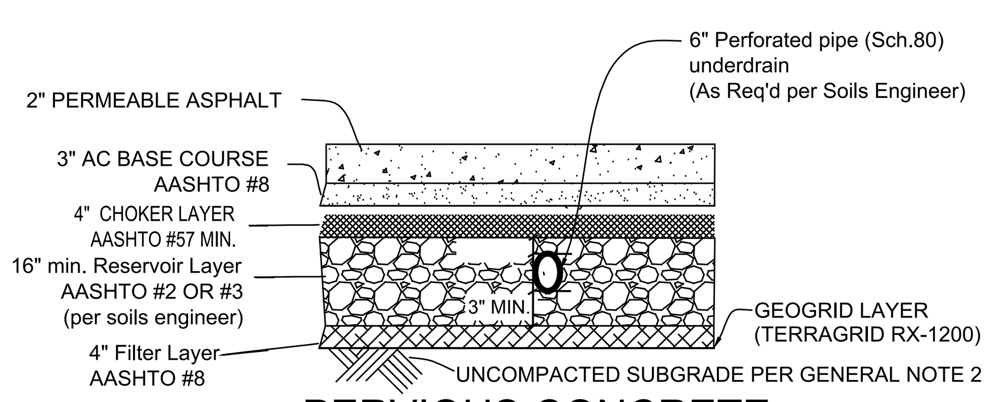
SECTION "B" – "B"
HORIZ. SCALE = 1" = 50'
VERT. SCALE = 1" = 10'



PCBMP #1
BIOFILTRATION BASIN
NOT TO SCALE



TYPICAL SECTION: MADISON AVENUE
SECONDARY HIGHWAY (STD 104)



PERVIOUS CONCRETE
Per Modified Drawing GS-5.2a
N.T.S.

NOTES:
REGULAR MAINTENANCE AND UPKEEP OF PERVIOUS SERVICES WILL BE REQUIRED TO ENSURE THAT SURFACE REMAINS PERVIOUS.

DO NOT SEAL PERVIOUS SURFACE AFTER INSTALLATION.
PERVIOUS SURFACE SLOPE NOT TO EXCEED 5%.
DEPTH OF RESERVOIR LAYER SHALL BE CALCULATED ON THE GRADING PLANS AND WILL BE SIZED TO ADDRESS STORMWATER CONVEYANCE REQUIREMENTS AND PAVEMENT STRUCTURAL DESIGN.

REGISTERED PROFESSIONAL ENGINEER KRISTIN LIPSKA GREENE NO. C57860 CIVIL STATE OF CALIFORNIA

dk Greene Consulting, Inc.
P. O. BOX 143
BONSALL, CA 92003

Underground Service Alert
Call: TOLL FREE 1-800 422-4133
TWO WORKING DAYS BEFORE YOU DIG

BENCH MARK
DESCRIPTON:
LOCATION:
RECORDED:
ELEVATION: DATUM:

APPROVED FOR SIGNATURE
PLAN CHECK ENGR. NAME TYPED
PLAN CHECK FIRM
R.C.E. NO.

HORIZONTAL: AS NOTED
VERTICAL: AS NOTED

PREPARED BY DATE 10/13/20
KRISTIN L. GREENE
PE NO. C 57860 EXP. 6/30/2022

DATE	INITIAL	REVISION DESCRIPTION	SHT. NO.	DATE	INITIAL
ENGINEER OF WORK				CITY APPROVAL	

SHEET 2
CITY OF MURRIETA
ENGINEERING DEPARTMENT
SHEETS 2

CONCEPTUAL GRADING
W.M. LYLES CO.
26501 MADISON AVE.
PARCEL 2, P.M. 7065, BK 26, PG 50

APPROVED
ROBERT K. MOEHLING DATE
CITY ENGINEER

DWN BY:
CHKD BY:
FIELD BK:
DRAWING NO.

JOB NO. 2019-23 October 14, 2020

Appendix C

Soils Map

Soils Report

Soils Map from NRCS Web Soil Survey

This property was determined to be predominantly Hydrologic **Soil Group C** per the U.S. Dept. of Agriculture Natural Resources Conservation Service (NRCS) Web Soil Survey online tool.

Report — Hydrologic Soil Group and Surface Runoff

Absence of an entry indicates that the data were not estimated. The dash indicates no documented presence.

Western Riverside Area, California

Map symbol and soil name	Pct. of map unit	Surface Runoff	Hydrologic Soil Group
AtC2—Arlington and Greenfield fine sandy loams, 2 to 8 percent slopes , eroded			
Arlington	45	High	C
Greenfield	40	Low	A
AtD2—Arlington and Greenfield fine sandy loams, 8 to 15 percent slopes, eroded			
Arlington	35	High	C
Greenfield	35	Low	A
GtA—Grangeville fine sandy loam, drained, 0 to 2 percent sl opes			
Grangeville	85	Very low	A/D
RsC—Riverwash			
Riverwash	100	Very low	—

LGC GEO-ENVIRONMENTAL, INC.

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT, PROPOSED OFFICE BUILDING AND WORKSHOP, 26501 MADISON AVENUE, CITY OF MURRIETA, RIVERSIDE COUNTY, CALIFORNIA, APN: 910-230-003.

Dated: April 25, 2019
Project No. G19-1706-10

Prepared For:

Mr. Todd R. Sheller
Lyles Diversified, Inc.
1210 West Olive Avenue
Fresno, California 93728

27570 COMMERCE CENTER DRIVE, SUITE. 128, TEMECULA, CALIFORNIA 92590
PHONE: 951.297.2450 I FAX: 951.719.2998 I WWW.LGCGEOENV.COM

April 25, 2019

Project No. G19-1706-10

Mr. Todd R. Sheller,
Assistant Vice President
Lyles Diversified, Inc.
1210 West Olive Avenue
Fresno, California 93728

Subject: ***Preliminary Geotechnical Investigation Report, Proposed Office Building and Workshop, 26501 Madison Avenue, City of Murrieta, Riverside County, California, APN: 910-230-003.***

LGC Geo-Environmental, Inc. (LGC) is pleased to submit herewith our preliminary geotechnical investigation report regarding proposed office building and workshop development of the subject property (the site), which is located at 26501 Madison Avenue, City of Murrieta, Riverside County, California. The site is identified as Assessor's Parcel Number 910-230-003.

This report presents the results of our research of published geologic/geotechnical reports and maps, geologic mapping and review of aerial imagery, field exploration and laboratory testing; in addition to our geotechnical and geologic judgment, opinions, conclusions and preliminary recommendations associated with the proposed office building and workshop development.

Based on the results of our field exploration, geologic mapping, field and laboratory testing, geologic and geotechnical engineering evaluations, along with our review of published literature and the referenced Site Plan it is our opinion that the subject site is suitable for the proposed office building and workshop development provided that the recommendations presented herein are utilized during the design, grading, and construction. LGC should review any grading plans, as well as any foundation/structural plans when they become available, and revise the recommendations presented herein, if necessary.

It has been a pleasure to be of service to you during the design stages of this project. If you have any questions regarding the contents of this report or should you require additional information, please do not hesitate to contact us.

Respectfully submitted,

LGC GEO-ENVIRONMENTAL, INC.

Duncan Walker, CEG 1395
Certified Engineering Geologist

PROFESSIONAL GEOLOGIST
DUNCAN WALKER
No. EG 1395
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
LARRY DWAIN COOLEY
No. C54037
Exp. 12/31/19
CIVIL
STATE OF CALIFORNIA

Larry D. Cooley, RCE 54037
Project Engineer

DW/RLG/LDC

Distribution: (4) Addressee

TABLE OF CONTENTS

LISTS OF TABLES, FIGURES & PLATES, AND APPENDICES

Tables

Figures & Plates

Appendices

1.0 INTRODUCTION

1.1 Purpose

This report presents the results of LGC Geo-Environmental, Inc.'s (LGC's) preliminary geotechnical investigation regarding proposed office building and workshop development of the subject property (the site), which is located at 26501 Madison Avenue, City of Murrieta, Riverside County, California. The site is identified as Assessor's Parcel Number (APN) 910-230-003.

In February 2019, LGC conducted a phase I environmental site assessment for the subject site, the results of which are documented in the referenced *Phase I Environmental Site Assessment* by LGC dated February 28, 2019.

The purpose of this preliminary geotechnical investigation is to determine the nature of surface and subsurface soil conditions, evaluate their characteristics, and provide geotechnical recommendations with respect to grading, construction, foundation design and other aspects relative to the proposed office building and workshop development of the subject site. The referenced 40-scale Site Plan by dk Greene Consulting, Inc. (undated), which depicts the site, was utilized as the base map for our Geotechnical Map for the site (Plate 1).

1.2 Scope of Services

Our scope of services included the following:

- Review of previous preliminary geotechnical and geologic reports for the site, as well as readily available published geologic maps, recent aerial imagery, and pertinent documents regarding the anticipated geologic and geotechnical conditions at the site (Appendix A).
- Geologic observations and mapping of the existing surface conditions on the site.
- Field exploration consisting of excavating nine exploratory trenches (TR-1 through TR-9) to determine existing subsurface geological conditions using a wheeled backhoe.
- Laboratory testing of selected representative samples of soil for characterization of the engineering properties of onsite soil.
- Geotechnical engineering and geologic analysis of the data with respect to the proposed office building and workshop development.
- Preparation of this report presenting our findings, conclusions, and preliminary geotechnical design recommendations for the proposed office building and workshop development.

1.3 Site Description and Topography

Located along the southwest side of Madison Avenue at its intersection with Golden Gate Circle, the subject site is approximately rectangular and comprises approximately 5.83 acres (Site Location Map, Figure 1). The site is vacant and unfenced. In the northwest there is an inactive water well which was installed in 2017; the well has a steel standpipe with a welded cap. During the 1980s and early 1990s, there was apparently a single-family residence (SFR) and another structure on the northeast part of the site along Madison Avenue. The former SFR was probably served by an onsite wastewater treatment system (OWTS). If there is or was an OWTS on the site, its location is unknown.

The regional surface slope for the site and surrounding area is generally toward the southwest. Ground surface elevations on the site range from approximately 1,088 feet above mean sea level (msl) along the northwest property line to approximately 1,040 feet above msl in the channel of Warm Springs Creek near the south property corner, based on the referenced 30-scale *Non-Specific Rough Grading Plan* by Saxon Engineering Services, Inc. (Saxon). An existing 2:1 (h:v) cut slope up to approximately 20 feet high descends southwest from the northwest part of the site toward an offsite parking area. There is an elevated L-shaped area in the northwest and northeast, which is partially underlain by undocumented artificial fill. The northwest portion is a bench; a cut slope ascends northwest from the bench toward higher ground offsite. The bench and a small adjoining pad, together with the access road from Madison Avenue in the northeast, were graded in 2017 for equipment access to drill and install the onsite water well. The northeast portion consists of an arcuate pad which includes the site of the former SFR; graded slopes descend southwest, southeast and northeast from the pad. The south portion of the site is apparently ungraded natural ground, including the steeply-sloped, incised channel of Warm Springs Creek. Most onsite stormwater, together with tributary runoff from the elevated offsite area to the northwest, apparently flows into Warm Springs Creek.

Map distances are approximate
Map base reproduced from Google Earth photo (08/13/2018).

FIGURE 1
SITE LOCATION MAP

Project Name	Lyles Diversified – Murrieta
Project No.	G19-1706-10
Geol./Eng.	DW/LDC
Scale	1" = 100'
Date	April 2019

1.4 ***Previous Preliminary Geotechnical Investigation***

In 2017, a previous preliminary geotechnical investigation was conducted on the subject site, the results of which are documented in *Geotechnical Investigation, Proposed Covered Outdoor Storage Facility, 26501 Madison Avenue, Murrieta, California* by Global Geo-Engineering, Inc. (Global), dated November 17, 2017 (Appendix B). Nine exploratory borings were drilled, logged and sampled to depths ranging from approximately 8.0 feet to 18.5 feet below ground surface (bgs). Groundwater was not encountered in any of the nine borings. Limited soil testing was conducted using soil samples from the borings. Global placed a perforated pipe for future percolation testing in its boring P-1 in the south part of the site (Figure 1). Global reportedly did not perform percolation testing in boring P-1, but the pipe remains.

1.5 ***Proposed Development and Grading***

The referenced 30-scale *Non-Specific Rough Grading Plan* by Saxon indicates that the following grading is proposed for the site. Most of the site will consist of a proposed cut/fill pad that will slope gently toward the south at approximately 2.7 percent grade. At the perimeters of the pad, proposed 2:1 (h:v) cut and fill slopes, as well as the existing 2:1 cut slope in the northwest part of the site, will transition from the proposed pad to adjoining offsite and onsite grade. Surface water flow will be directed toward a proposed infiltration device which will be located in the southwest area of the site. The proposed development will consist of an office building with an asphalt-paved parking area in the northwest and a workshop building with a gravel parking area in the southeast, together with two driveways extending from Madison Avenue, landscaped areas and hardscape areas. It is anticipated that the proposed structures will be constructed of wood and/or steel framing, with concrete footings and floor slabs constructed on-grade. The currently unimproved portion of Madison Avenue, which adjoins the site to the northeast, will be improved/paved extending northwest to the existing end of pavement.

1.6 ***Historical Aerial Photograph and Topographic Map Evaluation***

Historical aerial photographs of the site dating back to 1938, as well as historical topographic maps dating back to 1901, were reviewed as part of LGC's prior Phase I ESA. In addition, Google Earth Pro imagery (from 1994 to 2018) for the site and surrounding area was evaluated. Information from these sources, as it pertains to the geologic and geotechnical issues of the proposed development, is included herein.

2.0 ***FIELD EXPLORATION***

2.1 ***Surface Reconnaissance***

Surface reconnaissance of the subject site and accessible surrounding areas was accomplished by an LGC geologist during February and March 2019 to document existing surface geological conditions using the referenced Site Plan for plotting geologic units. This information has been plotted on the enclosed Geotechnical Map (Plate 1)

2.2 ***Field Exploration***

Prior to subsurface work, underground utilities clearance was obtained from Underground Service Alert of Southern California. Subsurface exploration at the subject site was performed on March 15, 2019 and involved excavating nine exploratory trenches (TR-1 through TR-9) to depths ranging from approximately 4.5 feet to 10.5 feet bgs using the backhoe.

Earth materials encountered within the exploratory trenches were classified and logged by an LGC geologist in accordance with the visual-manual procedures of the Unified Soil Classification System (USCS). At the conclusion of the subsurface exploration, all trenches were backfilled with excavated soil, using minor compactive effort. Minor settlement of the backfill soil may occur over time. The approximate locations of the exploratory trenches are shown on the Geotechnical Map (Plate 1).

2.3 ***Laboratory Testing***

During our subsurface exploration, representative samples of earth materials were collected for laboratory testing. Laboratory testing was performed on selected representative samples of onsite earth materials and included in-situ and maximum density and optimum moisture content, chloride content, sulfate content, minimum resistivity and pH, expansion index, atterburg limits, consolidation, direct shear, and R-value. Laboratory test data are presented in Appendix D, together with brief descriptions of the test criteria.

3.0 FINDINGS

3.1 Regional Geologic Setting

Regionally, the site is within the Peninsular Ranges Geomorphic Province of California. The Peninsular Ranges are characterized by steep, elongated valleys and mountain ranges that trend west and northwest. The mountainous areas are underlain by Pre-Cretaceous metasedimentary and metavolcanic rocks and Cretaceous plutonic rocks of the Southern California Batholith. The valleys are underlain by young alluvial deposits followed by Quaternary and Tertiary bedrock units (sandstones, mudstones and conglomerates, as well as volcanics). The site and surrounding area are primarily underlain by sandstone bedrock of Pauba formation (Pleistocene). Young alluvial fan deposits (Holocene and late Pleistocene) overlie Pauba formation bedrock in the southwest and south parts of the site including in Warm Springs Creek (U.S. Geologic Survey (USGS), 2003). Regional geology is presented on the Regional Geologic Map (Figure 2).

The northwest-southeast trending topography for the area is controlled by the Elsinore fault zone (EFZ), which extends northwesterly approximately 190 miles from San Diego County through Riverside County to southeastern Los Angeles County. The EFZ separates the Perris Block on the northeast, which includes the site, from the Santa Ana Mountains Block on the southwest. The subject site is not underlain by active faults. A short trace of the Wildomar fault, which is not designated an active fault, is located approximately 0.10 mile southwest of the site. The nearest active fault is the Wildomar fault, which is part of the EFZ and is located approximately 0.19 mile southwest of site. A narrow portion of the site along the southwest property line is within the County Fault Zone, which has been established by Riverside County regarding the Wildomar fault (California Geologic Survey (CGS), 2018b and Riverside County, 2018).

3.2 Local Geology and Soil Conditions

Based on our review of available geological and geotechnical literature, together with field mapping and LGC's nine exploratory backhoe trenches, the subject site is primarily underlain by topsoil and bedrock of the Pauba formation (Sandstone member). In Warm Springs Creek and the southwest-center area, young alluvial-fan deposits (Holocene and late Pleistocene) overlie Pauba formation bedrock. The subsurface geological contacts are described in greater detail below and presented in the logs of the exploratory trenches (Appendix C). The observed geologic units and contacts are depicted on the Geotechnical Map (Plate 1).

- **Artificial Fill (Undocumented) (Afu):** There are apparently areas of undocumented artificial fill on downslope portions of the former SFR site and the bench/pad for the water well. The undocumented fill was encountered in several of LGC's exploratory trenches and ranges up to an estimated 8.0 feet thick. The undocumented fill is generally composed of silty to clayey sand, which are various shades of brown, damp to moist, loose to medium dense, very fine- to medium-grained, with roots and roothairs.
- **Topsoil:** Topsoil was encountered in LGC's exploratory trenches and ranges from approximately 0.5 foot to 1.0 foot thick. The topsoil is generally composed of silty to clayey sand and sandy clay, which are various shades of brown, damp to very moist, loose, fine- to medium-grained, with pores, roots and roothairs.
- **Young Alluvial Fan Deposits (Qyf):** Holocene and late Pleistocene age young alluvial fan deposits (Qyf) overlie Pauba formation bedrock in the southwest and south parts of the site including in Warm Springs Creek and in an onsite drainage that trends approximately north across the site. The young alluvial fan deposits were encountered in LGC's exploratory trenches generally and range from approximately 2.5 feet to 9.0 feet thick. The young alluvial fan deposits are generally composed of silty to clayey sand and sandy silt and clay, which are various shades of brown, damp to wet, loose to dense, very fine- to coarse-grained, with pores.
- **Pauba Formation (Qpfs):** Pleistocene age bedrock of the Pauba formation (Sandstone member) was encountered underlying the undocumented artificial fill, topsoil and young alluvial fan deposits to the maximum depth of approximately 10.5 feet bgs in LGC's exploratory trenches on the subject site. Approximately the upper 1.0 foot to 2.0 feet are generally weathered to clayey sand, sandy silt and poorly-graded sand. The Pauba formation is generally composed of sandstone (very fine- to coarse-grained and friable) and siltstone, which are various shades of brown, dry to moist, moderately hard to very hard.

0 ft. 1,000 ft. 2,000 ft. 4,000 ft.

Map distances are approximate

N

Legend

Approximate Site boundary

Map base reproduced from *Preliminary Geologic Map of the Murrieta 7.5' Quadrangle* (USGS, 2003).

FIGURE 2
REGIONAL GEOLOGIC MAP

Project Name	Lyles Diversified – Murrieta
Project No.	G19-1706-10
Geol./Eng.	DW/LDC
Scale	1" = 2,000'
Date	April 2019

3.3 Groundwater

Groundwater was not encountered to a maximum depth of approximately 10.0 feet bgs in the nine exploratory trenches on the subject site during this preliminary geotechnical investigation. Groundwater was also not encountered to depths of approximately 8.0 feet to 18.5 feet bgs in any of the nine borings on the site during the previous preliminary geotechnical investigation by Global Geo-Engineering, Inc. in 2017. The California Department of Water Resources (DWR) *Water Data Library* website was reviewed regarding historical groundwater depths in wells near the subject site. The *Water Data Library* indicates State Well Number 335381N1171759W001 is the nearest well that is located on same side of Warm Springs Creek as the site. This well is located approximately 0.21 mile northeast of the site, and the only groundwater depth was recorded at 34 feet below ground surface (bgs) in 1968. In July and August 2017, a public water supply well was drilled and installed onsite in the northwest. This well is inactive (capped), and the recorded groundwater depth was 380 feet bgs on August 2, 2017 (Eric Haley dba Heritage Well Service, 2017).

3.4 Caving

Caving was not encountered within the nine exploratory trenches on the subject site during this investigation. Localized minor caving may occur within low-density portions of undocumented artificial fill and/or topsoil.

3.5 Surface Water

Based on our review of the referenced Site Plan, proposed onsite surface water flow from the proposed office building and adjoining paved parking area will be directed toward a proposed infiltration device which will be located in the southwest area of the site. Onsite surface water flow from the proposed workshop building and adjoining gravel parking area will be directed toward Warm Springs Creek. Surface water runoff relative to project design is the purview of the project civil engineer and should be designed to direct surface water runoff away from the proposed structures and walls. The southeast part of the site is within a 100-year flood zone associated with Warm Springs Creek; the zone extends approximately to the top of the west streambank.

3.6 Faulting

The geologic structure of the Southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. Faults such as the Newport-Inglewood, Whittier, Elsinore, San Jacinto and San Andreas, are major faults in this system and are known to be active and may produce moderate to strong ground shaking during an earthquake. In addition, the San Andreas, Elsinore and San Jacinto faults are known to have ruptured the ground surface in historic times.

The subject site is **not** underlain by active faults. A short trace of the Wildomar fault, which is not designated an active fault, is located approximately 0.10 mile southwest of the site (CGS, 2018b). The nearest active fault is the Wildomar fault, which is part of the EFZ and is located approximately 0.19 mile southwest of site. A narrow portion of the site along the southwest property line is within the County Fault Zone, which has been established by Riverside County regarding the Wildomar fault (CGS, 2018b and Riverside County, 2018).

Table 1 is a list of the significant faults located within 20 miles of the site (site coordinates of 33.5346°N, -117.1768°W). We have also included the Maximum Earthquake Magnitude predicted for each of these faults.

TABLE 1
SIGNIFICANT FAULTS IN PROXIMITY OF THE SITE

FAULT NAME	*APPROXIMATE DISTANCE (mi)*	*MAXIMUM EARTHQUAKE MAGNITUDE (Mw)*
Elsinore - Temecula (Wildomar)	0.2	6.8
Elsinore – Glen Ivy	12.6	6.8
Elsinore - Julian	14.5	7.1

Sources: EQFAULT for Windows Version 3.00b and Riverside County *Map My County* GIS Website

3.7 Secondary Seismic Effects

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include soil liquefaction and dynamic settlement. Other secondary seismic effects include shallow ground rupture, lateral spreading, seiches and tsunamis. In general, these secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault, and the onsite geology. An evaluation of these secondary seismic effects is included herein.

3.8 *Liquefaction*

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential.

The site is located within a Riverside County designated liquefaction hazard zone. Groundwater was not encountered in the nine exploratory trenches to a maximum depth of approximately 10.5 feet bgs during this preliminary geotechnical investigation on the subject site. Groundwater was also not encountered in the nine borings to a maximum depth of approximately 18.5 feet bgs during the previous preliminary geotechnical investigation by Global in 2017.

From the exploratory trenches and borings on the subject site, and review of the historic high groundwater data in the area (see section 3.3), a groundwater depth of 34 feet bgs was used for the liquefaction analyses. The analyses of proposed post-graded conditions did not indicate potentially liquefiable soils other than young alluvial fan deposits which extend to a maximum depth of approximately 9.0 feet bgs in the proposed development area. The Pauba formation bedrock that underlies the young alluvial fan deposits are not considered to be potentially liquefiable. Therefore, liquefaction does not present itself as a possible constraint for the proposed development.

3.9 *Subsidence*

The site is located within a Riverside County designated active subsidence zone. Unfavorable ground subsidence is not anticipated due to: recommended overexcavation associated with proposed structures and improvements and subsurface earth material types including Pauba formation bedrock.

3.10 *Landsliding*

Landslides or surface failures were not observed at or directly adjacent to the site. As a result, the possibility of the site being affected by land sliding is not anticipated.

3.11 *Shallow Ground Rupture*

The potential for shallow ground rupture is considered moderate at the site, due to potentially active faults near the site. Cracking because of shaking from nearby or distant seismic events is not considered a significant hazard, although it is a possibility at any site.

3.12 *Lateral Spreading*

Lateral spreading is the outward and downward movement of soil adjacent to a descending slope that occurs during a seismic event and is usually associated with liquefaction of underlying soils. This typically occurs adjacent to drainage channels as the affected soil moves laterally into the open channel area. The potential for lateral spreading is not considered to be a concern, due to the relatively hard nature of Pauba formation bedrock.

3.13 *Tsunamis and Seiches*

Based on the elevation and location of the site with respect to sea level and its distance from large open bodies of water, the potential of seiches and/or tsunamis is considered to be a nil possibility.

4.0 *CONCLUSIONS AND RECOMMENDATIONS*

Based on the results of our geotechnical investigation, it is our opinion that the proposed office building and workshop development as indicated on the referenced Site Plan and *Non-Specific Rough Grading Plan*, is feasible from a geotechnical and geologic standpoint provided that the following recommendations are incorporated into the design criteria and project specifications. When grading and foundation/structural plans for the proposed development are available, a comprehensive plan review should be performed by LGC. Depending on the results, additional recommendations may be necessary for geotechnical design parameters for both earthwork and foundations. Grading should be conducted in accordance with local and state codes, including the 2016 edition of the California Building Code (CBC), the recommendations within this report, and future geotechnical reports. It is also our opinion that the proposed grading and construction will not adversely impact the geologic stability of adjoining properties.

The following is a summary of the primary geotechnical factors, as determined from our geotechnical evaluation of the data, published/unpublished literature, and geotechnical reports:

- Based on our subsurface exploration, the site is underlain by topsoil, young alluvial fan deposits, and Pauba formation bedrock, as well as localized undocumented artificial fill associated with former structures and previous grading.
- Groundwater is not considered a constraint for the proposed development.
- Active or potentially active faults are not known to exist on the site.
- There are no known landslides impacting the site.
- Laboratory test results of the upper soil on the site indicate a **VERY LOW** to **LOW** expansion potential. For the site, earth materials are considered to have a **LOW** expansion potential.
- Laboratory test results of the upper soil indicate a **MEDIUM** plasticity index and liquid limit.
- Laboratory test results of the upper soil indicate a **negligible** potential for soluble sulfate attack on normal concrete and **negligible** chloride effects on reinforcing steel.
- Laboratory test results of the upper soil encountered indicated a **moderate** corrosion potential to buried metals.
- The site is underlain by approximately 3 feet to 9 feet of potentially-compressible topsoil, young alluvial fan deposits and weathered Pauba formation bedrock, as well as localized undocumented artificial fill, which may be prone to potential intolerable post-grading settlement and/or hydroconsolidation, under the surcharge of the future proposed structural loads and/or fill loads. These materials should be overexcavated to underlying competent bedrock and/or young alluvial fan deposits.
- From a geotechnical perspective, the existing onsite soil appears to be suitable material for use as fill, provided that the onsite soil is relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and organic material. It is anticipated that the onsite soil and bedrock may be excavated with conventional heavy-duty construction equipment.

5.0 SEISMIC DESIGN CONSIDERATIONS

5.1 Ground Motion

The site will probably experience ground shaking from moderate- to large-size earthquakes during the life of the proposed development. Furthermore, it should be recognized that the Southern California region is an area of high seismic risk, and that it is not considered feasible to make structures totally resistant to seismic-related hazards.

Proposed structures on the site should be designed and constructed to resist the effects of seismic ground motions as provided in the 2016 CBC Sections 1613 and 1616, and 2010 ASCE 7. The method of design is dependent on the seismic zoning, site characterizations, occupancy category, building configuration, type of structural system, and building height.

Table 2 presents the seismic design parameters, which were developed based on the CBC 2016 and should be used for the proposed structures. Site coordinates of 33.5346°N, -117.1768°W were used to derive the seismic parameters in Table 2.

TABLE 2
SEISMIC DESIGN SOIL PARAMETERS

SEISMIC DESIGN SOIL PARAMETERS (2016 CBC Section 1613 and 2010 ASCE 7)	
Site Class Definition (ASCE 7; Chapter 20)	C
Mapped Spectral Response Acceleration Parameter S_s (for 0.2 second)	1.58
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second)	0.59
Site Coefficient F_a (0.2-second period)	1.20
Site Coefficient F_v (1-second period)	1.41
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter S_{MS} (0.2-second period)	1.89
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter S_{M1} (1-second period)	0.83
Design Spectral Response Acceleration Parameter, S_{DS} (0.2-second period)	1.26
Design Spectral Response Acceleration Parameter, S_{D1} (1-second period)	0.55
Mean Peak Ground Acceleration, PGA_m	0.84

Source: *ATC (Applied Technology Council) Hazards by Location* Website (Structural Engineers Association of California)

6.0 GEOTECHNICAL DESIGN PARAMETERS

6.1 Shrinkage/Bulking and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite soils are replaced as properly compacted fill. Table 3 contains an estimate of the shrinkage and bulking factors for the various geologic units present onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction that will be achieved during grading.

TABLE 3
ESTIMATED SHRINKAGE/BULKING

GEOLOGIC UNIT	SHRINKAGE/BULKING
Undocumented Artificial Fill	10% to 15% (Shrinkage)
Topsoil	5% to 10% (Shrinkage)
Young Alluvial Fan Deposits (Qyf)	5% to 10% (Shrinkage)
Pauba Formation Bedrock (Qpfs)	2% to 7% (Shrinkage)

Subsidence due to recompaction of exposed overexcavation bottom prior to fill placement, and placement of proposed fills, is estimated to be about 0.15 foot to 0.20 foot.

The above estimates of shrinkage and subsidence are intended as an aid for project engineers in determining earthwork quantities. These are preliminary rough estimates which may vary with depth of removal, stripping losses, field conditions at the time of grading, etc. However, these estimates should be used with some caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during the grading operations.

6.2 Cut/Fill Transition and Fill Differentials

To mitigate distress to structures related to the potential adverse effects of excessive differential settlement, cut/fill transitions should be eliminated from all building areas where the depth of fill placed within the "fill" portion exceeds proposed footing depths. The entire structure should be founded on a uniform bearing material. This should be accomplished by overexcavating the "cut" portion and replacing the excavated materials as properly compacted fill, so that all footings for structures and walls are founded into engineered fill with a minimum of 2 feet of fill below footings for proposed structures and 2 feet below footings for proposed walls. Recommended depths of overexcavation are provided in the following table:

TABLE 4
CUT/FILL TRANSITION

DEPTH OF FILL ("fill" portion)	DEPTH OF OVEREXCAVATION ("cut" portion)
Up to 4 feet	Equal Depth
4 to 12 feet	4 feet
Greater than 12 feet	One-third the maximum thickness of fill placed on the "fill" portion (20 feet maximum)

Overexcavation of the "cut" portion should extend beyond the perimeter building lines to a horizontal distance equal to the depth of overexcavation or to a minimum distance of 5 feet, whichever is greater.

6.3 Excavation Characteristics

It is anticipated that the onsite soil may be excavated with conventional heavy-duty construction equipment, based on our subsurface exploration and experience with these materials in the area.

6.4 Compressible/Collapsible Soils

The results of laboratory testing, together with field observations, indicate that the upper 3 feet to 9 feet of surficial materials are susceptible to varying degrees of intolerable settlement and/or hydro-consolidation (collapse) when a load is applied, or the soil is saturated. Consequently, these materials should be overexcavated to underlying competent Pauba formation bedrock and replaced as engineered fill.

7.0 SITE EARTHWORK

7.1 General Earthwork and Grading Specifications

Earthwork and grading should be performed in accordance with applicable requirements of the grading code of the County of Riverside, and in accordance with the following recommendations prepared by this firm. Grading should also be performed in accordance with the applicable provisions of the attached "General Earthwork and Grading Specifications for Rough Grading" (Appendix E) prepared by LGC, unless specifically revised or amended herein.

7.2 Geotechnical Observations and Testing

Prior to the start of grading, a meeting should be held at the site with the owner, developer, grading contractor, civil engineer and LGC to discuss the work schedule and geotechnical aspects of the grading. Rough grading, which includes clearing, overexcavation, scarification/processing and fill placement, should be accomplished under the full-time observation and testing of LGC. Fills should not be placed without prior approval from the geotechnical consultant.

A representative of LGC should also be present onsite during grading operations to document proper placement and compaction of fills, as well as to document excavations and compliance with the other recommendations presented herein.

7.3 Clearing and Grubbing

Weeds and grass in areas to be graded should be stripped and hauled offsite. Trees to be removed should be grubbed so that their stumps and major-root systems are also removed, and the organic materials hauled offsite. During site grading, laborers should clear from fills, roots and other deleterious materials missed during clearing and grubbing operations.

LGC or a qualified representative should be notified at the appropriate times to provide observation and testing services during clearing and grubbing operations to observe and document compliance with the above recommendations. In addition, buried structures, and any unusual or adverse soil conditions encountered that are not described or anticipated herein, should be brought to the immediate attention of LGC.

7.4 Onsite Wastewater Treatment System Abandonment

There is no information available regarding the former SFR that was located on the northeast part the site, but it was probably served by an OWTS. If there is or was an OWTS on the site, its location is unknown. If an OWTS is encountered during future grading and development onsite, then it should be removed and/or properly abandoned under permit from the Riverside County Department of Environmental Health (RCDEH).

7.5 Water-Supply Well Abandonment

An inactive (capped) water well was observed on the northwest part of the site (Figure 1). If the well is not intended to be used in the future, then it should be properly abandoned (destroyed) under permit from the RCDEH.

7.6 Overexcavation and Ground Preparation

The site is underlain by up to approximately 3 feet to 9 feet of potentially compressible topsoil and weathered bedrock, as well as localized undocumented artificial fill. These potentially compressible materials are considered unsuitable for support of proposed fills, structures, and/or improvements and should be overexcavated to expose underlying competent Pauba formation bedrock. Within the shallow fill or cut areas of the proposed building pads, overexcavations should also be 4 feet below proposed grade or a minimum of 2 feet below the proposed footings in the building pad areas, whichever is deeper. The overexcavation should also extend at least 5 feet outside the proposed building footprints (or a 1:1 projection away from the footing to the approved removal bottom, whichever is greater). Groundwater is not anticipated to be encountered during site grading. Actual depths of overexcavation should be evaluated upon review of final grading and foundation plans on the basis of observations and testing during grading by LGC.

Prior to placing engineered fill, exposed bottom surfaces in each overexcavated area should first be scarified to a depth of approximately 6 inches, watered or air-dried as necessary to achieve a uniform moisture content of optimum or higher and then compacted in place to a relative compaction of 90 percent or more (based on American Society for Testing and Materials (ASTM) Test Method D1557).

The estimated locations, extent and approximate depths for overexcavation of unsuitable materials are indicated on the enclosed Geotechnical Map (Plate 1). LGC should be provided with appropriate survey staking during grading to document that depths and/or locations of recommended overexcavation are adequate.

Sidewalls for overexcavations greater than 5 feet in height should be no steeper than 1:1 (h:v) and should be periodically slope-boarded during their excavation to remove loose surficial debris and facilitate mapping. Flatter excavations may be necessary for stability.

The grading contractor will need to consider appropriate measures necessary to excavate adjacent existing improvements adjacent to the site without endangering them due to caving or sloughing.

7.7 Fill Suitability

Earth materials excavated during grading are generally considered suitable for use as compacted fill provided they do not contain significant amounts of trash, vegetation, construction debris and oversize material. It will be necessary to blend the excavated soil to mitigate the high expansion potential of some of the upper soil.

7.8 Oversized Material

Oversized material that may be encountered during grading, greater than 8 inches, should be reduced in size or removed from the site.

7.9 Benching

Where compacted fills are to be placed on natural slope surfaces inclining at 5:1 (h:v) or greater, the ground should be excavated to create a series of level benches, which are at least a minimum height of 4 feet, excavated into competent bedrock.

7.10 Fill Placement

Fills should be placed in uncompacted lifts having a maximum 8-inch thickness, watered or air-dried as necessary to achieve a uniform moisture content of at least optimum moisture content, and then compacted in place to relative compaction of 90 percent or more. Fills should be maintained in a relatively level condition. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with ASTM Test Method D1557.

7.11 *Inclement Weather*

Inclement weather may cause rapid erosion during mass grading and/or construction. Proper erosion and drainage control measures should be taken during periods of inclement weather in accordance with County of Riverside and California State requirements.

8.0 SLOPE CONSTRUCTION

8.1 *Slope Stability*

The full scope of proposed grading is not known at this time. The referenced *Non-Specific Rough Grading Plan* indicates that the following grading is proposed for the site, including the adjoining northeast site. Most of site (approximately 4 acres) will consist of a cut/fill pad at elevations ranging from approximately 1,058 feet to 1,073 feet above msl. At the perimeters of the pad, proposed 2:1 (h:v) cut and fill slopes up to approximately 15 feet high, as well as the existing 2:1 cut slope in the northwest, will transition from the proposed pad to adjoining offsite and onsite grade. The proposed and existing 2:1 cut and fill slopes should be grossly and surficially stable.

8.2 *Fill Slopes*

Following overexcavation of unsuitable soils, a 15-foot wide fill key excavated into competent bedrock should be provided at the toe of fill and fill over cut slopes. The bottom of the fill keys should be tilted at 2 percent back into the slope.

8.3 *Cut Slopes*

Proposed cut slopes may expose low-density, dry and/or cohesionless soils, which will likely require stabilization by overexcavation and replacement with compacted fill.

8.4 *Temporary Excavations*

Based on the physical properties of the onsite soils, temporary excavations exceeding 5 feet in height should be cut back at a ratio of 1:1 (h:v) or flatter, for the duration of the overexcavation and recompaction of unsuitable soil material. Temporary slopes excavated at the above slope configurations are expected to remain stable during grading operations. However, the temporary excavations should be observed by a representative of LGC for any evidence of potential instability. Depending on the results of these observations, revised slope configurations may be necessary.

Other factors which should be considered with respect to the stability of the temporary slopes include construction traffic and storage of materials on or near the tops of the slopes; construction scheduling; presence of nearby walls or structures on adjacent properties; drainage; and weather conditions at the time of construction. Applicable requirements of the California Construction and General Industry Safety Orders; the Occupational Safety and Health Act of 1970; and the Construction Safety Act should also be followed.

9.0 POST-GRADING CONSIDERATIONS

9.1 *Control of Surface Water and Drainage Control*

Positive-drainage device, such as sloping sidewalks, graded-swales and/or area drains, should be provided to collect and direct water away from the structure and slopes. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations. Roof gutters and downspouts should be provided on the sides of structures. Drainage should be directed to adjacent driveways, adjacent streets or storm-drain facilities. The ground surface adjacent to the structures should be sloped at a gradient of at least 5 percent for a distance of at least 10 feet, and further maintained by a swale or drainage path at a gradient of at least 2 percent. Where necessary, drainage paths may be shortened by use of area drains and collector pipes. The civil engineer is responsible for designing drain control devices on the site.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Over watering must be avoided.

9.2 ***Utility Trenches***

Utility-trench backfill within roadways, utility easements, under walls, sidewalks, driveways, floor slabs and any other structures or improvements should be compacted. The onsite soils should generally be suitable as trench backfill provided they are screened of rocks and other material over 3 inches in diameter and organic matter. Trench backfill should be compacted in uniform lifts (generally not exceeding 6 inches to 8 inches in uncompacted thickness) by mechanical means to at least 90 percent relative density (per ASTM Test Method D1557).

Where onsite soils are utilized as backfill, mechanical compaction should be used. Density testing, along with probing, should be performed by LGC or its representative, to document proper compaction.

If trenches are shallow and the use of conventional equipment may result in damage to the utilities; clean sand, having sand equivalent (SE) of 30 or greater, should be used to bed and shade the utilities. Sand backfill should be densified. The densification may be accomplished by jetting or flooding and then tamping to ensure adequate compaction. A representative from LGC should observe, probe, and test the backfill to verify compliance with the project specifications.

Utility-trench sidewalls deeper than 5 feet should be laid back at a ratio of 1:1 (h:v) or flatter or braced. A trench box may be used in lieu of shoring. If shoring is anticipated, LGC should be contacted to provide design parameters.

To avoid point-loads and subsequent distress to clay, cement or plastic pipe, imported sand bedding should be placed 1 foot or more above pipe in areas where excavated trench materials contain significant cobbles. Sand-bedding materials should be compacted and tested prior to placement of backfill.

Where utility trenches are proposed parallel to building footings (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the adjacent footing.

10.0 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

10.1 ***General***

Provided that site grading is performed in accordance with the recommendations of this report, conventional shallow foundations are still considered feasible for support of the proposed structures. Tentative foundation recommendations are provided herein. However, these recommendations may require modification depending on as-graded conditions within the building pad areas upon completion of grading.

10.2 ***Allowable-Bearing Values***

An allowable-bearing value of 1,500 pounds per square foot (psf) may be used for 24-inch square pad footings and 12-inch or more wide continuous footings founded in compacted fill or competent native soil/material at a depth of 12 inches or more below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of width and depth, to a value no greater than 1,800 psf.

10.3 ***Settlement***

Based on the general settlement characteristics of compacted fill, as well as the aforementioned overexcavation recommendations and anticipated loading, it is estimated that the total settlement of conventional footings will be approximately 0.50 inch. Differential settlement is expected to be 0.25-inch over 30 feet. It is anticipated that the majority of the static settlement will occur during construction or shortly thereafter as building loads are applied.

The above settlement estimates are based on the assumption that the grading will be performed in accordance with the grading recommendations presented in this report and that LGC will observe or test the soil conditions in the footing excavations.

10.4 Lateral Resistance

A passive earth pressure of 250 psf per foot of depth, to a maximum value of 450 psf may be used to determine lateral-bearing resistance for footings. The passive earth pressure incorporates a minimum factor of safety of 1.5. Where structures are planned in or near descending slopes, the passive earth pressure should be reduced to 150 psf per foot of depth to a maximum value of 300 psf. In addition, a coefficient of friction of 0.35 times the dead-load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. When combining passive and friction for lateral resistance, the passive component should be reduced by one third.

The above values are based on footings placed directly against engineered compacted fill. In the case where footing sides are formed, backfill placed against the footings should be compacted to 90 percent or more of maximum dry density as determined by ASTM D1557.

10.5 Footing Setbacks from Descending Slopes

Where structures are proposed near the tops of descending graded or natural slopes, the footing setbacks from the slope face should conform to the 2016 CBC, Figure 1808.7.1. The required setback is H/3 (one-third the slope height) measured along a horizontal line projected from the lower outside face of the footing to the slope face. The footing setbacks should be 5 feet where the slope height is 15 feet or less and up to a maximum of 40 feet where the slope height exceeds 15 feet.

10.6 Building Clearances from Ascending Slopes

Building setbacks from ascending graded or natural slopes should conform with the 2016 CBC, Figure 1808.7.1, which requires a building clearance of H/2 (one-half the slope height) varying from 5 to 15 feet. The building clearance is measured along a horizontal line projected from the toe of the slope to the face of the building. A retaining wall may be constructed at the base of the slope to achieve the required building clearance.

10.7 Footing Observations

Footing excavations should be observed by LGC to document that they have been excavated into competent bearing soils. The foundation excavations should be observed prior to the placement of forms, reinforcement or concrete. The excavations should be trimmed neat, level and square. Loose, sloughed or moisture-softened soil should be removed prior to concrete placement.

Excavated materials from footing excavations should not be placed in slab-on-ground areas unless the soils are compacted to 90 percent or more of maximum dry density as determined by ASTM D1557.

10.8 Expansive Soil Considerations

The results of laboratory testing indicate that onsite earth materials exhibit an overall expansion potential of **LOW** in accordance with 2016 CBC, Chapter 18. However, expansive soil conditions should be evaluated for the building pads during and at the completion of rough grading to observe and document the actual as-graded conditions. It will be necessary to blend the excavated soil to mitigate the high expansion potential of some of the upper soil. The design and construction details presented herein are intended to provide recommendations for the levels of expansion potential which may be evident at the completion of rough grading. Furthermore, it should be noted that additional slab thickness, footing sizes and/or reinforcement more stringent than the recommendations that follow should be provided as recommended by the project architect or structural engineer.

10.9 Footings/Floor Slabs – Low Expansion Potential

The following are our recommendations where foundation soils exhibit **LOW** expansion potential as classified in accordance with 2016 CBC. However, expansive soil conditions should be evaluated for the building pads during and at the completion of rough grading to observe and document the actual as-graded conditions. For this condition, it is recommended that footings and floors be constructed and reinforced in accordance with the following criteria. However, additional slab thickness, footing sizes and/or reinforcement may be required by the project architect or structural engineer. We recommend using a Plasticity Index of 14 per our Atterberg limits test results (Appendix D).

Footings

- Exterior continuous footings should be founded into compacted engineered fill below the lowest adjacent final grade at minimum depths of 12 inches and 18 inches deep for one-story and two-story construction, respectively. Interior continuous footings may be founded at a depth of 12 inches or greater into compacted engineered fill below the lowest adjacent final grade. Continuous footings should have a minimum width of 12 inches for one-story and 15 inches for two-story structures.
- Continuous footings should be reinforced with two (2) No. 4 bars, one near top and one at bottom.
- Interior isolated pad footings should be 24 inches or more square and founded at a depth of 12 inches or more below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer's recommendation.
- Exterior pad footings should be 24 inches square or greater and founded at a depth of 18 inches or more below the lowest adjacent grade; and if isolated, interconnected and connected to the main foundation by in-grade beams. Exterior footings should be reinforced in accordance with the structural engineer's recommendations.

Floor Slabs

- Concrete foundation floor slabs should be 4 inches or more thick and reinforced with No. 3 bars spaced 24 inches or less on-centers, both ways. Slab reinforcement should be supported on concrete chairs so that the desired placement is properly placed per the design engineer.
- Concrete floors should be underlain with a moisture-vapor retarder consisting of a 15-mil thick vapor barrier. Laps within the membrane should be sealed and overlapped 12 inches. Two inches or more of clean sand should be placed above and below the membrane. These recommendations must be confirmed (and/or modified) by the foundation engineer with our concurrence, based upon the performance expectations of the foundation. It is the responsibility of the contractor to ensure that the moisture/vapor barrier systems are placed in accordance with the project plans and specifications, and that the moisture/vapor retarder materials are free of tears and punctures prior to concrete placement. Additional moisture reduction and/or prevention measures may be needed, depending on the performance requirements of future interior floor coverings.
- Garage area floor slabs should be 4 inches thick and should be reinforced in a similar manner as concrete floor slabs. Garage area floor slabs should also be placed separately from adjacent wall footings with a positive separation maintained with 3/8-inch minimum felt expansion joint materials and quartered with weakened-plane joints. A 12-inch wide grade beam founded at the same depth as adjacent footings should be provided across garage entrances. The grade beam should be reinforced with a minimum of two No. 4 bars, one top and one bottom.
- Prior to placing concrete, the subgrade soils below all floor slabs should be pre-watered to achieve a moisture content that is equal to 120 percent of the optimum moisture content of the subgrade soils. The moisture content should penetrate to a minimum depth of 18 inches. This will promote uniform curing of the concrete and minimize the development of shrinkage cracks.

10.10 Nonstructural Concrete Flatwork

Concrete flatwork (such as walkways, driveways, patios, bicycle trails, etc.) has a high potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 5. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 5
NONSTRUCTURAL CONCRETE FLATWORK FOR LOW EXPANSIVE SOILS

	Private Sidewalks	***Private Drives***	***Patios/Entryways***	***City Sidewalk Curb and Gutters***
Minimum Thickness (in.)	4 (nominal)	4 (full)	4 (full)	City/Agency Standard
Presaturation	Presoak to 18 inches	Presoak to 18 inches	Presoak to 18 inches	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on center	No. 3 at 24 inches on center	City/Agency Standard
Thickened Edge	—	8″ x 8″	8″ x 8″	City/Agency Standard
Crack Control	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard

11.0 SOIL CORROSIVITY

The National Association of Corrosion Engineers (NACE) defines corrosion as "a deterioration of a substance or its properties because of a reaction with its environment". From a geotechnical viewpoint, the "environment" is the prevailing foundation soils and the "substances" are the reinforced concrete foundations or various buried metallic elements such as rebar, piles, pipes, etc., which are in direct contact with or within close vicinity of the foundation soil.

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates. ACI 318R-05, Table 4.3.1 provides specific guidelines for the concrete mix design based on different amount of soluble sulfate content. The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover, or plain steel substructures such as steel pipes or piles, is 500 ppm per California Test 532 and ACI 318R-05, Table 4.4.1.

The corrosion potential of the onsite materials was evaluated for its effect on steel and concrete. The corrosion potential was evaluated using the results of laboratory tests performed on representative samples obtained during the subsurface exploration. Laboratory testing was performed to evaluate pH, resistivity, chloride content, and soluble sulfate content. Based on the laboratory testing performed, the onsite soils are classified as having a **negligible** sulfate exposure condition in accordance with ACI 318R-05, Table 4.3.1, and **negligible** chloride exposure condition in accordance with ACI 318R-05, Table 4.4.1. Based on laboratory testing of onsite soil, it is also our opinion that onsite soil should be considered to have a **moderate** corrosion risk to buried metals due to the moderate resistivity. Metal piping should be corrosion-protected or consideration should be given to using plastic piping instead of metal or plastic sleeves around the pipe.

Despite the minimum recommendation above, LGC is not a corrosion-engineering firm. Therefore, we recommend that you consult with a competent corrosion engineer and conduct additional testing (if required) to evaluate the actual corrosion potential of the site and to provide recommendations to reduce the corrosion potential with respect to the proposed improvements. The recommendations of the corrosion engineer may supersede the above recommendations.

These recommendations are based on representative samples of the near-surface engineered fill soils. The initiation of grading at the site could blend various soil types and import soils may be used locally. These changes made to the foundation soils could alter sulfate-content levels. Accordingly, it is recommended that additional testing may be performed at the completion of grading.

12.0 RETAINING WALLS

12.1 Lateral Earth Pressures and Retaining Wall Design Parameters

Conventional foundations for retaining walls within properly compacted fill within competent bedrock should be embedded at least 18 inches below lowest adjacent grade. At this depth, an allowable bearing capacity of 1,500 psf may be assumed for retaining walls founded in competent compacted fill.

The following lateral earth pressures are recommended for retaining walls that may be proposed. The recommended lateral pressures for approved onsite soils or import material (with an expansion index of **20** or less and phi angle of internal friction of at least **30** degrees), for level or sloping backfill are presented in Table 6. **Onsite fill soil with an expansion index of greater than 20 should not be used as backfill due to the expansive nature.** Onsite soil should be screened of rocks and other material over 3 inches in diameter.

TABLE 6
LATERAL EARTH PRESSURES

CONDITIONS	***EQUIVALENT FLUID WEIGHT (pcf)***			
	Level Backfill (up to 6 feet)	***Level Backfill Dynamic (>6 feet to10 feet)***	***2:1 Backfill Ascending (up to 6 feet)***	***2:1 Backfill Ascending-Dynamic (>6 feet to 10 feet)***
Active	45	45	80	55
At-Rest	70	70	100	95
Seismic	0	45	0	95
Passive	250	250	120	120

Notes:
1. Applicable to retaining walls only.
2. Active force applied a 1/3 wall height.
3. Seismic force applied to at 1/2 to 3/5 wall height.
4. Lateral pressure acts normally to vertical stem.

For sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations.

Restrained structural walls should include design for at rest conditions, if applicable. The magnitude of those pressures depends on the amount of deformation that the wall can yield under load. If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the retained soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at-rest" conditions.

The equivalent fluid pressure values assume free-draining conditions and a soil expansion index of 20 or less. If conditions other than those assumed above are anticipated, revised equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineers.

12.2 Footing Embedments

The base of retaining wall footings constructed on level ground may be founded at a depth of 12 inches or more below the lowest adjacent final grade. Where retaining walls are proposed on or within 15 feet from the top of an adjacent descending fill slope, the footings should be deepened such that a minimum horizontal clearance of H/3 (one-third the slope height) is maintained between the outside bottom edges of the footings and the face of the slope but not to exceed 15 feet or be less than 5 feet. The above recommended footing setbacks are preliminary and may be revised based on site-specific soil conditions. Footing or pier excavations should be observed by the project geotechnical representative to document that the footing trenches have been excavated into competent bearing soils and to the embedments recommended above. These observations should be performed prior to placing forms or reinforcing steel.

12.3 Drainage

All retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. The outlet pipe should be sloped to drain to a suitable outlet. It should be noted that that recommended subdrains does not provide protection against seepage through the face of the wall and/or efflorescence. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

Weep holes or open vertical masonry joints should be provided in retaining walls 3 feet or less in height to reduce the likelihood of entrapment of water in the backfill. Weep holes, if used, should be 3 inches or more in diameter and provided at intervals of 6 feet or less along the wall. Open vertical masonry joints, if used, should be provided at 32-inch or less intervals. A continuous gravel fill, 12 inches by 12 inches, should be placed behind the weep holes or open masonry joints. The gravel should be wrapped in filter fabric to reduce infiltration of fines and subsequent clogging of the gravel. Filter fabric may consist of Mirafi 140N or equivalent.

In lieu of weep holes or open joints, for retaining walls less than 3 feet, a perforated pipe and gravel subdrain may be used. Perforated pipe should consist of 4-inch or more diameter PVC Schedule 40 or ABS SDR-35, with the perforations laid down. The pipe should be embedded in 1.5 cubic feet per foot of 0.75 or 1.5-inch open graded gravel wrapped in filter fabric. Filter fabric may consist of Mirafi 140N equivalent.

Retaining walls greater than 3 feet high should be provided with a continuous backdrain for the full height of the wall. This drain could consist of geosynthetic drainage composite, such as Miradrain 6000 or equivalent, or a permeable drain material, placed against the entire backside of the wall. If a permeable drain material is used, the backdrain should be 1 or more feet thick. Caltrans Class II permeable material or open graded gravel or crushed stone (described above) may be used as permeable drain material. If gravel or crushed stone is used, it should have less than 5 percent material passing the No. 200 sieve. The drain should be separated from the backfill with a geofabric. The upper 1 foot of the backdrain should be covered with compacted fill. A drainage pipe consisting of 4-inch diameter perforated pipe (described above) surrounded by 1 cubic foot per foot of gravel or crushed rock wrapped in a filter fabric should be provided along the back of the wall. The pipe should be placed with perforations down, sloped at 2 percent or more and discharge to an appropriate outlet through a solid pipe. The pipe should outlet away from structures and slopes. The outside portions of retaining walls supporting backfill should be coated with an approved waterproofing compound to inhibit infiltration of moisture through the walls.

12.4 Temporary Excavations

Retaining walls, if any are proposed, should be constructed and backfilled as soon as possible after backcut excavations are constructed. Prolonged exposure of backcut slopes may result in some localized slope instability. To facilitate retaining wall construction, the lower 5 feet of temporary slopes may be cut vertical and the upper portions exceeding a height of 5 feet should be cut back at a gradient of 1:1 (h:v) or flatter for the duration of construction. However, temporary slopes should be observed by LGC for evidence of potential instability. Depending on the results of these observations, flatter slopes may be necessary. The potential effects of various parameters such as weather, heavy equipment travel, storage near the tops of the temporary excavations and construction scheduling should also be considered in the stability of temporary slopes. Water should not be permitted to drain away from the slope. Surcharges, due to equipment, spoil piles, etc., should not be allowed within 10 feet of the top of the slope.

All excavations should be made in accordance with Cal/OSHA. Excavation safety is the sole responsibility of the contractor.

12.5 Retaining Wall Backfill

Any retaining wall backfill soils (with an expansion index of 20 or less) should be placed in 6-inch to 8-inch loose lifts, watered or air-dried as necessary to achieve near optimum moisture conditions and compacted to at least 90 percent relative density (based on ASTM Test Methods D2922 and D3017).

13.0 PRELIMINARY PAVEMENT DESIGN

Structural pavement section design recommendations presented herein are based on a soil sample from our preliminary geotechnical investigation, as well as a soil sample from our previous preliminary geotechnical investigation for the adjoining northeast site. However, it should be understood that the soil material exposed during grading may differ from the materials sampled and tested during this investigation. Therefore, these preliminary pavement recommendations are subject to verification and possible revision based on any revised Traffic Indices (TI's), as well as sampling and testing of subgrade soils that exist after rough grading.

For planning and design purposes, we have prepared the following preliminary pavement sections based on R-value testing results. The R-value is 68 for a soil sample collected on the site, which has been used in Table 7 below for preliminary pavement section recommendations. Table 7 presents recommended preliminary pavement designs for a TI of 5.0 for Driveways & Parking Lots (Local Roads) and a TI of 6.0 for Residential Collectors, based on the design R-value of 68 and City of Murrieta pavement sections.

TABLE 7
PRELIMINARY PAVEMENT DESIGN

AREA	***ASSUMED TRAFFIC INDEX***	***DESIGN (AVERAGE) R-VALUE***	***ASPHALTIC CONCRETE (AC) (inches)***	***AGGREGATE BASE (AB) (inches)***
Driveways & Parking Lots (Local Roads)	5.0	68	3.0	6.0
Residential Collectors	6.0	68	4.0	6.0

Subgrade soil immediately below the aggregate base (base) should be compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D1557 to a minimum depth of 12 inches. Final subgrade compaction should be performed prior to placing base or asphaltic concrete and after all utility trench backfills have been compacted and tested.

Base materials should consist of crushed aggregate base conforming to Section 200-2 of Greenbook. The upper 12 inches of all aggregate base materials should be compacted to at least 95 percent of the laboratory maximum dry density determined in accordance with ASTM D1557.

Our preliminary pavement recommendations should be considered as minimum, per City of Murrieta requirements.

14.0 PLAN REVIEWS AND CONSTRUCTION SERVICES

This report has been prepared for the exclusive use of **Lyles Diversified, Inc.** to assist the project engineer and architect in the design of the proposed office building and workshop development. It is recommended that LGC be engaged to review the rough grading plans, storm-drain/storm water mitigation plans, structural plans and the final design drawings and specifications prior to construction. This is to document that the recommendations contained in this report have been properly interpreted are incorporated into the project specifications. LGC's review of the rough grading plan may indicate that additional subsurface exploration, laboratory testing and analysis should be performed to address areas of concern. If LGC is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that LGC be retained to provide geotechnical engineering services during both the rough grading and construction phases of the work. This is to document compliance with the design, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

If the project plans change significantly (e.g., building loads or type of structures), we should be retained to review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appears to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

15.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The subsurface observations and information contained herein are believed representative of the entire project; however, soil and geologic conditions revealed by excavation may be different than our preliminary findings. If this occurs, the changed conditions must be evaluated by the project geotechnical engineer and engineering geologist and design(s) adjusted as required or alternate design(s) recommended.

The findings of this report may be modified upon performing future geotechnical/geologic evaluations. However, changes in the conditions of a property can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our professional judgment. The findings, conclusions and recommendations contained in this report are to be considered tentative only and subject to confirmation by LGC during the construction process. Without this confirmation, this report is to be considered incomplete and LGC will not assume any responsibility for its use.

The conclusions and opinions contained in this report are valid up to a period of 1 year from the date of this report or adopted changes within the California Building Code, whichever occurs first. Changes in the conditions of a site can and do occur with the passage of time, whether those be because of natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate codes or standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside LGC's control. Therefore, if any of the above-mentioned situations occur, an update of this report must be completed.

This report has not been prepared for use by parties or projects other than those named or designed above. It may not contain sufficient information for other parties or other purposes.

The opportunity to be of service is appreciated. Should you have any questions regarding the content of this report, or should you require additional information, please do not hesitate to contact this office at your earliest convenience.

Appendix D

Modified Rational Method Analysis
Using CivilD

```
              Riverside County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software,(c) 1989 - 2018 Version 9.0
      Rational Hydrology Study        Date: 10/14/20  File:1.out
------------------------------------------------------------------------
Madison Ave
Existing Basin A
------------------------------------------------------------------------
 *********   Hydrology Study Control Information **********

 English (in-lb) Units used in input data file
------------------------------------------------------------------------
Program License Serial Number 6463
------------------------------------------------------------------------
Rational Method Hydrology Program based on
Riverside County Flood Control & Water Conservation District
1978 hydrology manual

Storm event (year) =   10.00 Antecedent Moisture Condition = 3

Standard intensity-duration curves data (Plate D-4.1)
For the [ Murrieta,Tmc,Rnch CaNorco ] area used.
10 year storm 10 minute intensity =  2.360(In/Hr)
10 year storm 60 minute intensity =  0.880(In/Hr)
100 year storm 10 minute intensity =  3.480(In/Hr)
100 year storm 60 minute intensity =  1.300(In/Hr)

Storm event year =  10.0
Calculated rainfall intensity data:
1 hour intensity =  0.880(In/Hr)
Slope of intensity duration curve = 0.5500

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station  0.000(Ft.) to Point/Station  309.000(Ft.)
**** INITIAL AREA EVALUATION ****
______________________________________________________________________
Initial area flow distance =   309.000(Ft.)
Top (of initial area) elevation =  1083.000(Ft.)
Bottom (of initial area) elevation =  1057.000(Ft.)
Difference in elevation =    26.000(Ft.)
Slope =    0.08414  s(percent)=       8.41
TC = k(0.530)*[(length^3)/(elevation change)]^0.2
Initial area time of concentration =    8.615 min.
Rainfall intensity =      2.559(In/Hr) for a    10.0 year storm
UNDEVELOPED (poor cover) subarea
Runoff Coefficient = 0.873
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
```

```
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  94.40
Pervious area fraction =  1.000; Impervious fraction =  0.000
Initial subarea runoff =      0.447(CFS)
Total initial stream area =        0.200(Ac.)
Pervious area fraction = 1.000
End of computations, total study area =            0.20 (Ac.)
The following figures may
be used for a unit hydrograph study of the same area.

Area averaged pervious area fraction(Ap) = 1.000
Area averaged RI index number =  86.0
```

```
             Riverside County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software,(c) 1989 - 2018 Version 9.0
      Rational Hydrology Study        Date: 10/14/20  File:2ExBasinB.out
------------------------------------------------------------------------
Madison Avenue
Existing Basin B and D
------------------------------------------------------------------------
 *********   Hydrology Study Control Information **********

 English (in-lb) Units used in input data file
------------------------------------------------------------------------
Program License Serial Number 6463
------------------------------------------------------------------------
Rational Method Hydrology Program based on
Riverside County Flood Control & Water Conservation District
1978 hydrology manual

Storm event (year) =   10.00 Antecedent Moisture Condition = 3

Standard intensity-duration curves data (Plate D-4.1)
For the [ Murrieta,Tmc,Rnch CaNorco ] area used.
10 year storm 10 minute intensity =  2.360(In/Hr)
10 year storm 60 minute intensity =  0.880(In/Hr)
100 year storm 10 minute intensity =  3.480(In/Hr)
100 year storm 60 minute intensity =  1.300(In/Hr)

Storm event year =  10.0
Calculated rainfall intensity data:
1 hour intensity =  0.880(In/Hr)
Slope of intensity duration curve = 0.5500

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station    0.000(Ft.) to Point/Station  242.000(Ft.)
**** INITIAL AREA EVALUATION ****
______________________________________________________________________
Initial area flow distance =   242.000(Ft.)
Top (of initial area) elevation =  1098.000(Ft.)
Bottom (of initial area) elevation =  1089.000(Ft.)
Difference in elevation =     9.000(Ft.)
Slope =    0.03719  s(percent)=       3.72
TC = k(0.530)*[(length^3)/(elevation change)]^0.2
Initial area time of concentration =    9.198 min.
Rainfall intensity =      2.468(In/Hr) for a    10.0 year storm
UNDEVELOPED (poor cover) subarea
Runoff Coefficient = 0.872
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  94.40
Pervious area fraction =  1.000; Impervious fraction =  0.000
```

```
Initial subarea runoff =      1.227(CFS)
Total initial stream area =        0.570(Ac.)
Pervious area fraction = 1.000

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station  242.000(Ft.) to Point/Station  690.000(Ft.)
**** SUBAREA FLOW ADDITION ****
______________________________________________________________________
UNDEVELOPED (poor cover) subarea
Runoff Coefficient = 0.872
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  94.40
Pervious area fraction =  1.000; Impervious fraction =  0.000
Time of concentration =     9.20 min.
Rainfall intensity =      2.468(In/Hr) for a    10.0 year storm
Subarea runoff =      0.710(CFS) for      0.330(Ac.)
Total runoff =      1.937(CFS)   Total area =       0.900(Ac.)
End of computations, total study area =           0.90 (Ac.)
The following figures may
be used for a unit hydrograph study of the same area.

Area averaged pervious area fraction(Ap) = 1.000
Area averaged RI index number =  86.0
```

```
              Riverside County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software,(c) 1989 - 2018 Version 9.0
      Rational Hydrology Study        Date: 10/14/20  File:2ExBasinC.out
------------------------------------------------------------------------
Madison Avenue
Existing Basin C
------------------------------------------------------------------------
 *********   Hydrology Study Control Information **********

 English (in-lb) Units used in input data file
------------------------------------------------------------------------
Program License Serial Number 6463
------------------------------------------------------------------------
Rational Method Hydrology Program based on
Riverside County Flood Control & Water Conservation District
1978 hydrology manual

Storm event (year) =   10.00 Antecedent Moisture Condition = 3

Standard intensity-duration curves data (Plate D-4.1)
For the [ Murrieta,Tmc,Rnch CaNorco ] area used.
10 year storm 10 minute intensity =  2.360(In/Hr)
10 year storm 60 minute intensity =  0.880(In/Hr)
100 year storm 10 minute intensity =  3.480(In/Hr)
100 year storm 60 minute intensity =  1.300(In/Hr)

Storm event year =  10.0
Calculated rainfall intensity data:
1 hour intensity =  0.880(In/Hr)
Slope of intensity duration curve = 0.5500

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station   0.000(Ft.) to Point/Station  639.000(Ft.)
**** INITIAL AREA EVALUATION ****
______________________________________________________________________
Initial area flow distance =   639.000(Ft.)
Top (of initial area) elevation =  1079.000(Ft.)
Bottom (of initial area) elevation =  1052.000(Ft.)
Difference in elevation =    27.000(Ft.)
Slope =    0.04225  s(percent)=       4.23
TC = k(0.530)*[(length^3)/(elevation change)]^0.2
Initial area time of concentration =   13.222 min.
Rainfall intensity =      2.022(In/Hr) for a    10.0 year storm
UNDEVELOPED (poor cover) subarea
Runoff Coefficient = 0.866
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  94.40
```

```
Pervious area fraction =  1.000; Impervious fraction =  0.000
Initial subarea runoff =      6.234(CFS)
Total initial stream area =        3.560(Ac.)
Pervious area fraction = 1.000
End of computations, total study area =           3.56 (Ac.)
The following figures may
be used for a unit hydrograph study of the same area.

Area averaged pervious area fraction(Ap) = 1.000
Area averaged RI index number =  86.0
```

```
              Riverside County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software,(c) 1989 - 2018 Version 9.0
       Rational Hydrology Study        Date: 10/14/20  File:2PropBasinA.out
------------------------------------------------------------------------
Madison Avenue
Proposed Basin A
------------------------------------------------------------------------
 *********   Hydrology Study Control Information **********

 English (in-lb) Units used in input data file
------------------------------------------------------------------------
Program License Serial Number 6463
------------------------------------------------------------------------
Rational Method Hydrology Program based on
Riverside County Flood Control & Water Conservation District
1978 hydrology manual

Storm event (year) =   10.00 Antecedent Moisture Condition = 3

Standard intensity-duration curves data (Plate D-4.1)
For the [ Murrieta,Tmc,Rnch CaNorco ] area used.
10 year storm 10 minute intensity =  2.360(In/Hr)
10 year storm 60 minute intensity =  0.880(In/Hr)
100 year storm 10 minute intensity =  3.480(In/Hr)
100 year storm 60 minute intensity =  1.300(In/Hr)

Storm event year =  10.0
Calculated rainfall intensity data:
1 hour intensity =  0.880(In/Hr)
Slope of intensity duration curve = 0.5500

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station   0.000(Ft.) to Point/Station  295.000(Ft.)
**** INITIAL AREA EVALUATION ****
______________________________________________________________________
Initial area flow distance =   295.000(Ft.)
Top (of initial area) elevation =  1078.000(Ft.)
Bottom (of initial area) elevation =  1067.000(Ft.)
Difference in elevation =    11.000(Ft.)
Slope =    0.03729  s(percent)=       3.73
TC = k(0.300)*[(length^3)/(elevation change)]^0.2
Initial area time of concentration =    5.633 min.
Rainfall intensity =      3.233(In/Hr) for a    10.0 year storm
COMMERCIAL subarea type
Runoff Coefficient = 0.894
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  84.40
Pervious area fraction =  0.100; Impervious fraction =  0.900
```

```
Initial subarea runoff =      0.578(CFS)
Total initial stream area =        0.200(Ac.)
Pervious area fraction = 0.100
End of computations, total study area =            0.20 (Ac.)
The following figures may
be used for a unit hydrograph study of the same area.

Area averaged pervious area fraction(Ap) = 0.100
Area averaged RI index number =  69.0
```

```
              Riverside County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software,(c) 1989 - 2018 Version 9.0
      Rational Hydrology Study        Date: 10/14/20  File:2PropBasinBD.out
------------------------------------------------------------------------
Madison Avenue
Proposed Basin B and D
------------------------------------------------------------------------
 *********   Hydrology Study Control Information **********

 English (in-lb) Units used in input data file
------------------------------------------------------------------------
Program License Serial Number 6463
------------------------------------------------------------------------
Rational Method Hydrology Program based on
Riverside County Flood Control & Water Conservation District
1978 hydrology manual

Storm event (year) =   10.00 Antecedent Moisture Condition = 3

Standard intensity-duration curves data (Plate D-4.1)
For the [ Murrieta,Tmc,Rnch CaNorco ] area used.
10 year storm 10 minute intensity =  2.360(In/Hr)
10 year storm 60 minute intensity =  0.880(In/Hr)
100 year storm 10 minute intensity =  3.480(In/Hr)
100 year storm 60 minute intensity =  1.300(In/Hr)

Storm event year =  10.0
Calculated rainfall intensity data:
1 hour intensity =  0.880(In/Hr)
Slope of intensity duration curve = 0.5500

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station   0.000(Ft.) to Point/Station  351.000(Ft.)
**** INITIAL AREA EVALUATION ****
______________________________________________________________________
Initial area flow distance =   351.000(Ft.)
Top (of initial area) elevation =  1094.000(Ft.)
Bottom (of initial area) elevation =  1076.000(Ft.)
Difference in elevation =    18.000(Ft.)
Slope =    0.05128  s(percent)=       5.13
TC = k(0.530)*[(length^3)/(elevation change)]^0.2
Initial area time of concentration =   10.009 min.
Rainfall intensity =      2.356(In/Hr) for a    10.0 year storm
UNDEVELOPED (poor cover) subarea
Runoff Coefficient = 0.871
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  94.40
```

```
Pervious area fraction =  1.000; Impervious fraction =  0.000
Initial subarea runoff =      1.170(CFS)
Total initial stream area =        0.570(Ac.)
Pervious area fraction = 1.000

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station  351.000(Ft.) to Point/Station  781.000(Ft.)
**** SUBAREA FLOW ADDITION ****
______________________________________________________________________
UNDEVELOPED (poor cover) subarea
Runoff Coefficient = 0.871
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  94.40
Pervious area fraction =  1.000; Impervious fraction =  0.000
Time of concentration =    10.01 min.
Rainfall intensity =      2.356(In/Hr) for a    10.0 year storm
Subarea runoff =      0.677(CFS) for      0.330(Ac.)
Total runoff =      1.847(CFS)   Total area =       0.900(Ac.)
End of computations, total study area =            0.90 (Ac.)
The following figures may
be used for a unit hydrograph study of the same area.

Area averaged pervious area fraction(Ap) = 1.000
Area averaged RI index number =  86.0
```

```
            Riverside County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software,(c) 1989 - 2018 Version 9.0
      Rational Hydrology Study        Date: 10/14/20  File:2proBasinC1.out
------------------------------------------------------------------------
Madison Avenue
Proposed Basin C1
------------------------------------------------------------------------
 *********   Hydrology Study Control Information **********

 English (in-lb) Units used in input data file
------------------------------------------------------------------------
Program License Serial Number 6463
------------------------------------------------------------------------
Rational Method Hydrology Program based on
Riverside County Flood Control & Water Conservation District
1978 hydrology manual

Storm event (year) =   10.00 Antecedent Moisture Condition = 3

Standard intensity-duration curves data (Plate D-4.1)
For the [ Murrieta,Tmc,Rnch CaNorco ] area used.
10 year storm 10 minute intensity =  2.360(In/Hr)
10 year storm 60 minute intensity =  0.880(In/Hr)
100 year storm 10 minute intensity =  3.480(In/Hr)
100 year storm 60 minute intensity =  1.300(In/Hr)

Storm event year =  10.0
Calculated rainfall intensity data:
1 hour intensity =  0.880(In/Hr)
Slope of intensity duration curve = 0.5500

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station   0.000(Ft.) to Point/Station  701.000(Ft.)
**** INITIAL AREA EVALUATION ****
______________________________________________________________________
Initial area flow distance =   701.000(Ft.)
Top (of initial area) elevation =  1070.000(Ft.)
Bottom (of initial area) elevation =  1057.200(Ft.)
Difference in elevation =    12.800(Ft.)
Slope =    0.01826  s(percent)=       1.83
TC = k(0.300)*[(length^3)/(elevation change)]^0.2
Initial area time of concentration =    9.186 min.
Rainfall intensity =      2.470(In/Hr) for a    10.0 year storm
COMMERCIAL subarea type
Runoff Coefficient = 0.892
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  84.40
```

```
Pervious area fraction =  0.100; Impervious fraction =  0.900
Initial subarea runoff =      4.208(CFS)
Total initial stream area =        1.910(Ac.)
Pervious area fraction = 0.100
End of computations, total study area =           1.91 (Ac.)
The following figures may
be used for a unit hydrograph study of the same area.

Area averaged pervious area fraction(Ap) = 0.100
Area averaged RI index number =  69.0
```

```
              Riverside County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software,(c) 1989 - 2018 Version 9.0
       Rational Hydrology Study        Date: 10/14/20  File:2PropBasinC2.out
------------------------------------------------------------------------
Madison Avenue
Proposed Basin C2
------------------------------------------------------------------------
 *********   Hydrology Study Control Information **********

 English (in-lb) Units used in input data file
------------------------------------------------------------------------
Program License Serial Number 6463
------------------------------------------------------------------------
Rational Method Hydrology Program based on
Riverside County Flood Control & Water Conservation District
1978 hydrology manual

Storm event (year) =   10.00 Antecedent Moisture Condition = 3

Standard intensity-duration curves data (Plate D-4.1)
For the [ Murrieta,Tmc,Rnch CaNorco ] area used.
10 year storm 10 minute intensity =  2.360(In/Hr)
10 year storm 60 minute intensity =  0.880(In/Hr)
100 year storm 10 minute intensity =  3.480(In/Hr)
100 year storm 60 minute intensity =  1.300(In/Hr)

Storm event year =  10.0
Calculated rainfall intensity data:
1 hour intensity =  0.880(In/Hr)
Slope of intensity duration curve = 0.5500

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Process from Point/Station   0.000(Ft.) to Point/Station  401.000(Ft.)
**** INITIAL AREA EVALUATION ****
______________________________________________________________________
Initial area flow distance =   401.000(Ft.)
Top (of initial area) elevation =  1069.700(Ft.)
Bottom (of initial area) elevation =  1055.000(Ft.)
Difference in elevation =    14.700(Ft.)
Slope =    0.03666  s(percent)=       3.67
TC = k(0.300)*[(length^3)/(elevation change)]^0.2
Initial area time of concentration =    6.391 min.
Rainfall intensity =      3.016(In/Hr) for a    10.0 year storm
COMMERCIAL subarea type
Runoff Coefficient = 0.893
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 1.000
Decimal fraction soil group D = 0.000
RI index for soil(AMC 3)  =  84.40
```

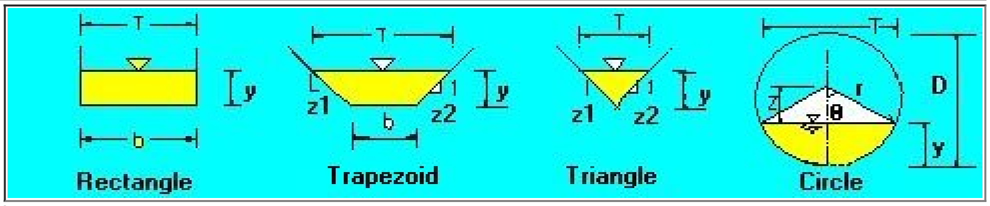
```
Pervious area fraction =  0.100; Impervious fraction =  0.900
Initial subarea runoff =      4.445(CFS)
Total initial stream area =        1.650(Ac.)
Pervious area fraction = 0.100
End of computations, total study area =            1.65 (Ac.)
The following figures may
be used for a unit hydrograph study of the same area.

Area averaged pervious area fraction(Ap) = 0.100
Area averaged RI index number =  69.0
```

Appendix E

Hydraulic Calculations
Using Open Channel Flow Calculator

The open channel flow calculator

Select Channel Type: Circle

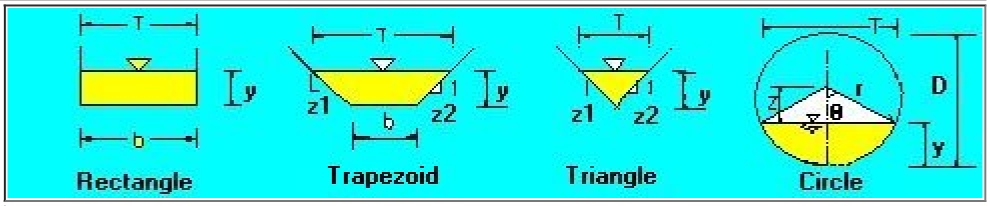


Depth from Q | Select unit system: Feet(ft)

Channel slope: .02 ft/ft	Water depth(y): 0.7 ft	Radius (r) .5 ft
Flow velocity 7.185 ft/s	LeftSlope (Z1): to 1 (H:V)	RightSlope (Z2): to 1 (H:V)
Flow discharge 4.2 ft^3/s	Input n value .013 or select n	
Calculate!	Status: Calculation finished	Reset
Wetted perimeter 1.98 ft	Flow area 0.59 ft^2	Top width(T) 0.92 ft
Specific energy 1.5 ft	Froude number 1.58	Flow status Supercritical flow
Critical depth 0.87 ft	Critical slope 0.0127 ft/ft	Velocity head 0.8 ft

Copyright 2000 Dr. Xing Fang, Department of Civil Engineering, Lamar University.

The open channel flow calculator

Select Channel Type: Circle	
Depth from Q	Select unit system: Feet(ft)

Channel slope: .02 ft/ft	Water depth(y): 0.74 ft	Radius (r) .5 ft
Flow velocity 7.25 ft/s	LeftSlope (Z1): to 1 (H:V)	RightSlope (Z2): to 1 (H:V)
Flow discharge 4.45 ft^3/s	Input n value .013 or select n	
Calculate!	Status: Calculation finished	Reset
Wetted perimeter 2.06 ft	Flow area 0.62 ft^2	Top width(T) 0.88 ft
Specific energy 1.55 ft	Froude number 1.53	Flow status Supercritical flow
Critical depth 0.89 ft	Critical slope 0.0139 ft/ft	Velocity head 0.82 ft

Copyright 2000 Dr. Xing Fang, Department of Civil Engineering, Lamar University.