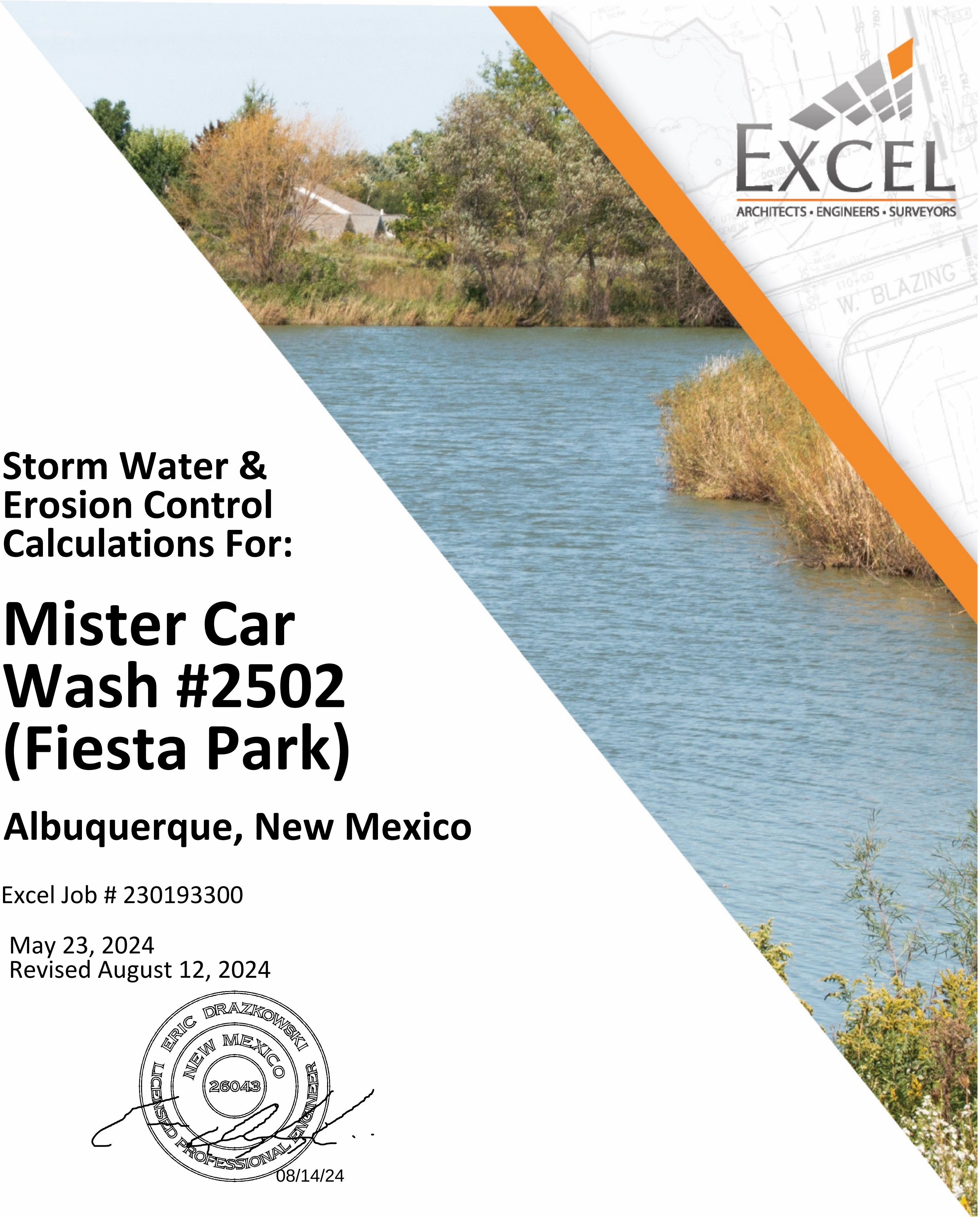

Storm Water & Erosion Control Calculations For:

Mister Car Wash #2502 (Fiesta Park)

Albuquerque, New Mexico

Excel Job # 230193300

May 23, 2024
Revised August 12, 2024

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Table of Contents

Appendices

0.0 Introduction

0.1 Existing Conditions

The proposed development is located on the north side of Alameda Boulevard NE in the city of Albuquerque, New Mexico. The project site is bound by Alameda Boulevard to the south, Columbine Avenue NE to the north, and commercial properties to the east and west. The existing site is currently vacant. The site currently drains to the northwest. The existing site can be seen in Appendix A.

- Property Area: 1.06 acres

0.2 Proposed Project Overview

The proposed project will include a proposed building with parking and vacuum stalls located to west and vehicle queuing to the east. The proposed development will drain to inlets that will drain stormwater north to a proposed detention pond. The stormwater management pond will reduce peak flows and treat stormwater to meet local and state requirements. The pond will overflow into the north driveway, draining north to the curb and gutter in Columbine Avenue. The proposed site can be seen in Appendix B.

- Disturbed Area: 1.07 acres

1.0 Design Criteria

1.1 Soils

Soil characteristics were determined using the web soil survey. See Table 1 for a summary of the soils and hydrologic ratings indicated by the web soil survey and Appendix E for web soil survey map.

Table 1: Web Soil Survey

MAP SYMBOL	SOIL TYPE	HYDROLOGIC RATING
EtC	Embudo-Tijeras complex	A

Soil borings were completed for the project site. The complete geotechnical investigation with boring logs can be seen in Appendix F.

1.2 Rainfall Data

NOAA Atlas 14, Albuquerque, New Mexico rainfall depths with a NOAA distribution was used for stormwater calculations.

Table 2: NOAA Atlas 14 24-hour Rainfall Depth

DESIGN STORM	RAINFALL DEPTH (INCHES)
100-YEAR	2.68

2.0 Stormwater Management Requirements

2.1 Peak Discharge

City of Albuquerque- Per communication with the city and based upon previously approved plans which conducted downstream capacity analysis, a conservative maximum allowable 100-yr discharge rate for the property is 3.21 cfs.

Additionally, City of Albuquerque Development Process Manual Article 6-2(A) Calculations were completed and are shown in Appendix C. These calculations show the existing peak discharge rate for the site as 3.24 cfs and the proposed site discharge as 4.30 cfs; however, these calculations do not take the proposed stormwater management pond into account.

A dry pond will be used to reduce peak flows to the conservative maximum allowable discharge rate. The proposed site will generate a peak flow of 3.36 cfs from Post Basins A and B. The site runoff must be reduced by 0.15 cfs to meet the 3.21 cfs max for the site. There is additional neighboring offsite flow to the pond which increases the total peak discharge to 5.07 cfs. The total discharge is at the same location in Columbine as per the original neighboring design.

Table 3: Runoff Summary

		POST DEVELOPMENT			
DESIGN STORM	**To Pond (cfs)**	**Offsite Undetained (cfs)**	**Pond discharge (cfs)**	**Offsite to Pond (cfs)**	Peak Discharge (Excluding offsite to pond) (cfs)
100YR-24HR	4.65	0.72	4.38	2.01	3.06

(**To Pond** includes Post Basin A, which is the proposed site area going to the Pond as well as Post Basin C, which is offsite flow that comes onto the site that will also go to the proposed pond)

(**Offsite Undetained** is post Basin B, which is site area that flows offsite from the site, without going to the pond)

(**Pond Discharge**) is the total pond discharge, including Post Basin A and Post Basin C flow to the pond)

(Offsite to Pond) is the amount of flow from Post Basin C to the pond

Peak Discharge (Excluding offsite to pond) is the modeled Peak Discharge from the pond and Offsite undetained area (taking time of concentration into account), subtracting the offsite to Pond.

Table 4: Pond Summary

DESIGN STORM	POND RELEASE RATE (CFS)	STORAGE VOLUME (C.F.)	MAXIMUM ELEVATION (FT)
100YR-24HR	4.38	2,690 c.f.	5114.86

Table 5: 100yr-24hr storm pond summary

POND	EMERGENCY SPILLWAY ELEVATION (FT)	CALCULATED POND ELEVATION (FT)	POND DISCHARGE IN (CFS)	DISCHARGE EXIT POINT
	5114.50	5114.86	4.38	Driveway

Table 6: Peak Discharge Release Summary

DESIGN STORM	MAX ALLOWABLE (CFS)	POST DEVELOPMENT (CFS)
100 YR- 24 HR	3.21	3.06

Table 6 shows that post development release rates will be less than maximum allowable discharge rate. See sheet C1.3 and C2.0 of the construction plans for pond design and Appendix D for peak discharge calculations.

Therefore, peak discharge requirements are met.

2.2 Stormwater Quality

City of Albuquerque – Per communication with the city of Albuquerque**,** the site is considered a new development project and will be required to apply best management practices to manage stormwater quality volume by management on-site, or payment-in-lieu, or private off-site mitigation. To calculate the required SWQV, multiply the impervious area draining to the BMP by 0.42 inches.

Proposed Impervious area=35,658 sf X (0.42/12)=1,248 cf required.

As seen within the peak discharge section of this report, as well as Appendix D, the provided pond storage in the 100 year storm is 2,690 c.f. which exceeds the minimum required 1,248 cf.

Therefore, stormwater quality requirements are met.

3.0 Storm Sewer Design

All storm sewer has been designed to convey the 100-year 24-hour post development storm.

See Appendix G, Appendix H, and Appendix I for pipe drainage areas and pipe sizing calculations.

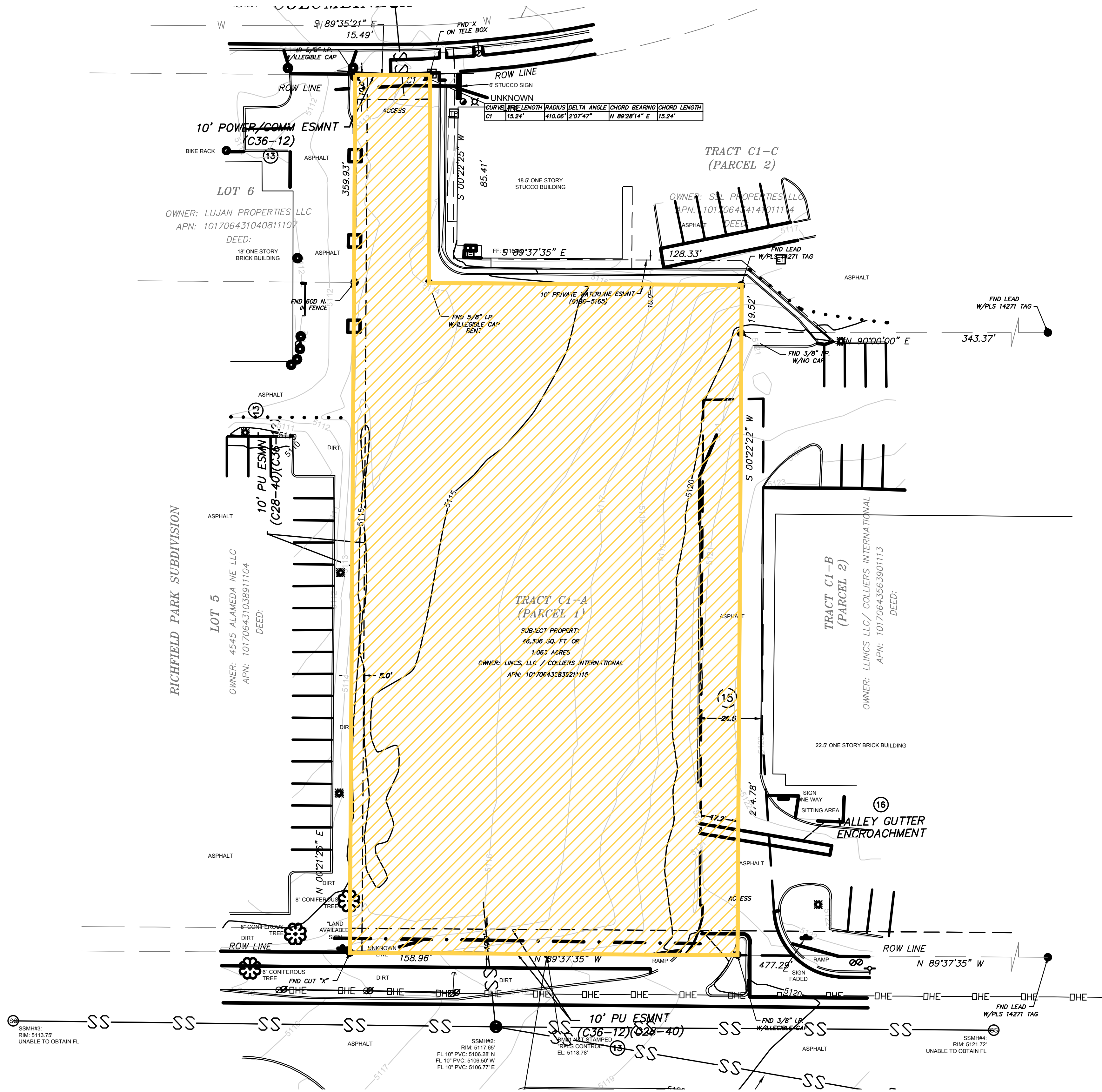
3.1 Emergency Overflow Route

The emergency overflow route is to the north, through the existing driveway. Maximum ponding onsite will be 6” in drive aisles and parking stalls.

4.0 Erosion Control

The erosion control specifications, construction sequence, site stabilization notes, seeding notes, dewatering notes, and post construction and maintenance plan will be included on sheet C0.1 of the construction plan set.

Appendix A: Pre-Development Basin Area(s)



BENCHMARKS/UTILITIES:

CP#1 SET 1/2 REBAR
W/ "RPLS CONTROL" CAP
OPUS CORRECTED
EL: 5113.99' NAVD88
NM CENTRAL ZONE NAD83

BM#1 N&T STAMPED
"RPLS CONTROL"
EL: 5118.78'

BM#2 TOP NUT
OF HYDRANT
EL: 5114.40'

BM#3
FND 1/2" STEEL ROD (NGS G442)
NAVD88 EL: 5112.86'

SSMH#1:
RIM: 5112.10'
FL 10" PVC: 5104.92' S
FL 10" PVC: 5104.86' N
FL 10" PVC: 5104.68' W

SSMH#2:
RIM: 5117.65'
FL 10" PVC: 5106.28' N
FL 10" PVC: 5106.50' W
FL 10" PVC: 5106.77' E

SSMH#3:
RIM: 5113.75'
UNABLE TO OBTAIN FL

SSMH#4:
RIM: 5121.72'
UNABLE TO OBTAIN FL

IG#1:
GRATE: 5115.47'
24" RCP, EXTREME ANGLE
BOTTOM OF BOX: 5110.74'

IG#2:
GRATE: 5124.27'
24" RCP, EXTREME ANGLE
BOTTOM OF BOX: 5116.60'

PRE-DEVELOPMENT BASIN AREA(S)

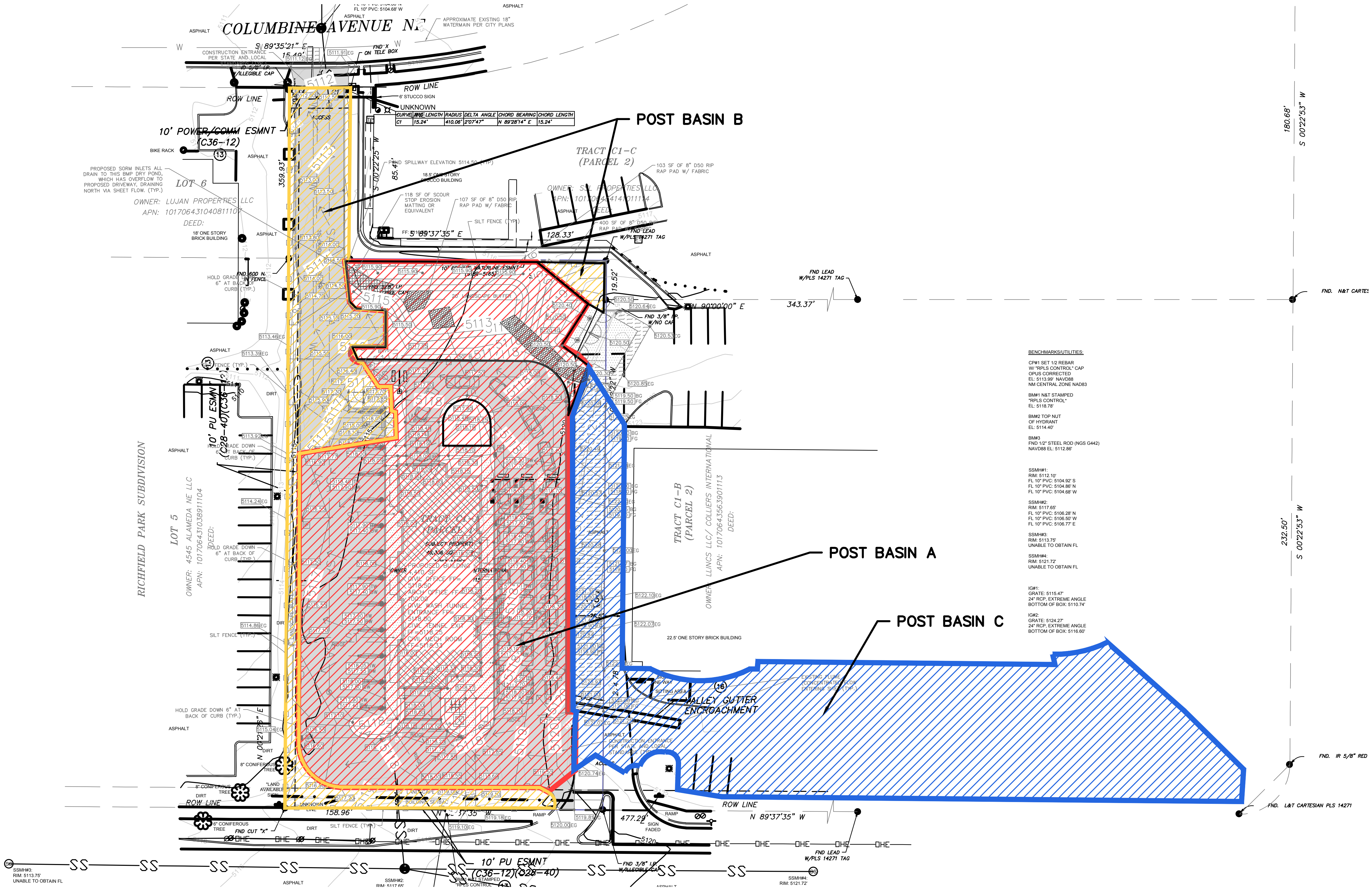
NORTH

1"= 50'

50' 0 50' 100'

SCALE 1"=50' (11X17) FEET

Appendix B: Post Development Basin Area(s)



POST-DEVELOPMENT BASIN AREA(S)

POST BASIN	TOTAL (SF)	TOTAL (AC)	BLDG (SF)	BLDG (AC)	PAVEMENT (SF)	PAVEMENT (AC)	OPEN (SF)	OPEN (AC)
A	32,107	0.74	4,410	0.10	21,902	0.50	5,795	0.13
B	9,590	0.22	0	0.00	5,537	0.13	4,053	0.09
C	23,470	0.54	0	0.00	23,470	0.54	0	0.00

NORTH

1"= 50'

50' 0 50' 100'

SCALE 1"=50' (11X17) FEET

Appendix C: City of Albuquerque Development Process Manual Article 6-2(A) Calculations

CITY OF ALBUQUERQUE DEVELOPMENT PROCESS MANUAL ARTICLE6-2(A) CALCULATIONS

EXISTING CONDITION

	A (sf)	E	(ZONE 2)
A		0.62	
B		0.8	
C	46306	1.03	
D	0	2.33	
TOTAL	46306		

ON-SITE WEIGHTED EXCESS PRECIPITATION (100 YEAR, 6 Hour Storm)

$$\textbf{Weighted E} = \frac{E_A A_A + E_B A_B + E_C A_C + E_D A_D}{A_A + A_B + A_C + A_D}$$

Weighted E 1.03 in.

$$\mathbf{V_{360}\ (as\ volume)} = \text{weighted } E^* (A_A + A_B + A_C + A_D)$$

On site volume of runoff: V360= 3975

On site Peak discharge Rate

Qpa	1.71	cfs
Qpb	2.36	cfs
Qpc	3.05	cfs
Qpd	4.34	cfs
Qp=	3.24	cfs

PROPOSED CONDITON

	A (sf)	E	(ZONE 2)
A		0.62	
B		0.8	
C	10648	1.03	
D	35658	2.33	
TOTAL	46306		

ON-SITE WEIGHTED EXCESS PRECIPITATION (100 YEAR, 6 Hour Storm)

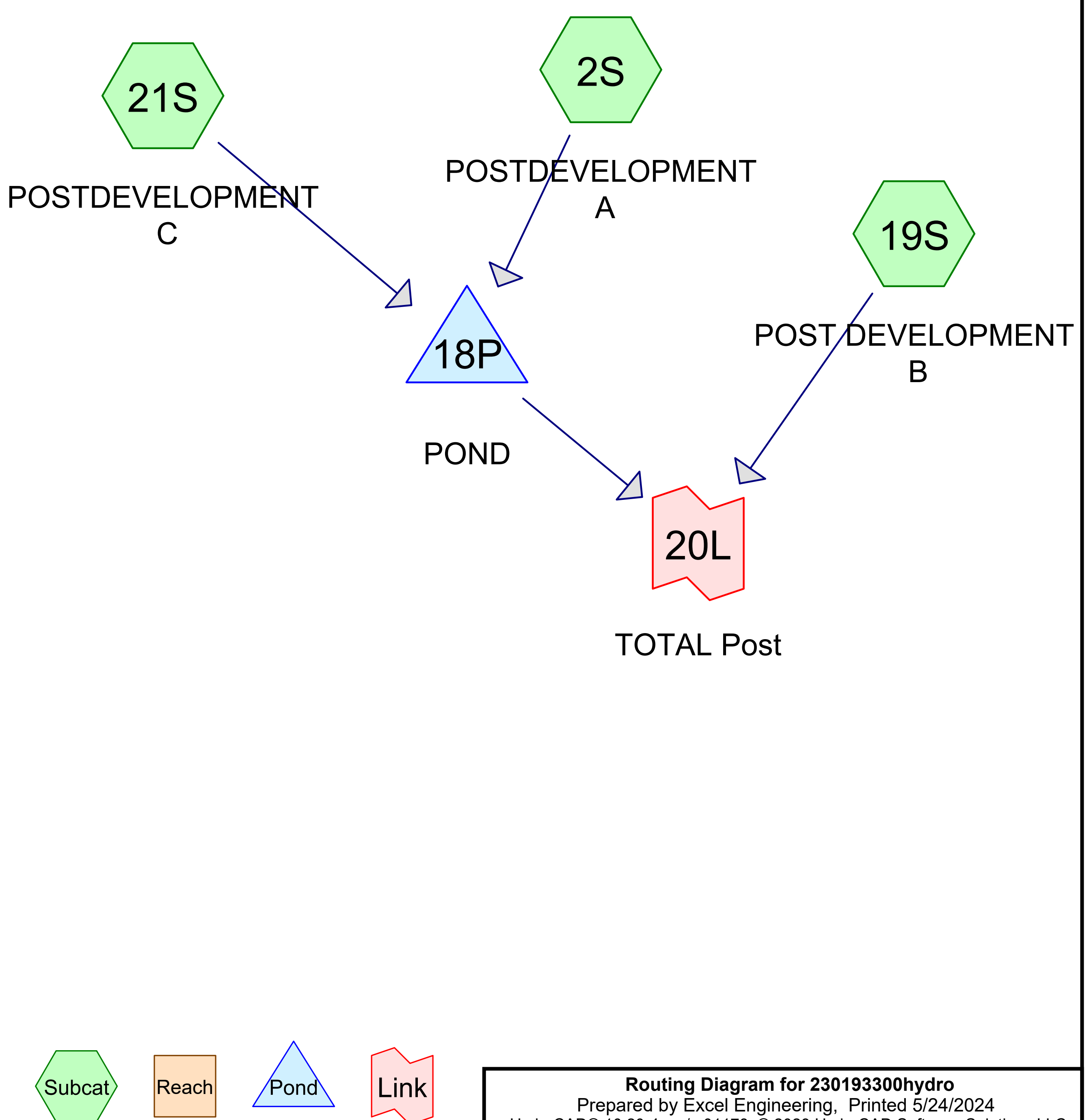
Weighted E 2.03 in.

On site volume of runoff: V360= 7838 cf

On site Peak discharge Rate

Qpa	1.71	cfs
Qpb	2.36	cfs
Qpc	3.05	cfs
Qpd	4.34	cfs
Qp=	4.30	cfs

Appendix D: Peak Discharge Calculations



Routing Diagram for 230193300hydro
Prepared by Excel Engineering, Printed 5/24/2024
HydroCAD® 10.20-4a s/n 01178 © 2023 HydroCAD Software Solutions LLC

Time span=1.00-20.00 hrs, dt=0.01 hrs, 1901 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: POSTDEVELOPMENTA Runoff Area=32,107 sf 81.95% Impervious Runoff Depth>2.15"
Tc=6.0 min CN=96 Runoff=2.64 cfs 0.132 af

Subcatchment19S: POST DEVELOPMENT Runoff Area=9,590 sf 57.74% Impervious Runoff Depth>1.87"
Tc=6.0 min CN=93 Runoff=0.72 cfs 0.034 af

Subcatchment21S: Runoff Area=23,470 sf 100.00% Impervious Runoff Depth>2.36"
Tc=6.0 min CN=98 Runoff=2.01 cfs 0.106 af

Pond 18P: POND Peak Elev=5,114.86' Storage=2,690 cf Inflow=4.65 cfs 0.238 af
Discarded=0.00 cfs 0.003 af Primary=4.38 cfs 0.188 af Outflow=4.38 cfs 0.191 af

Link 20L: TOTAL Post Inflow=5.07 cfs 0.222 af
Primary=5.07 cfs 0.222 af

Summary for Subcatchment 2S: POSTDEVELOPMENT A

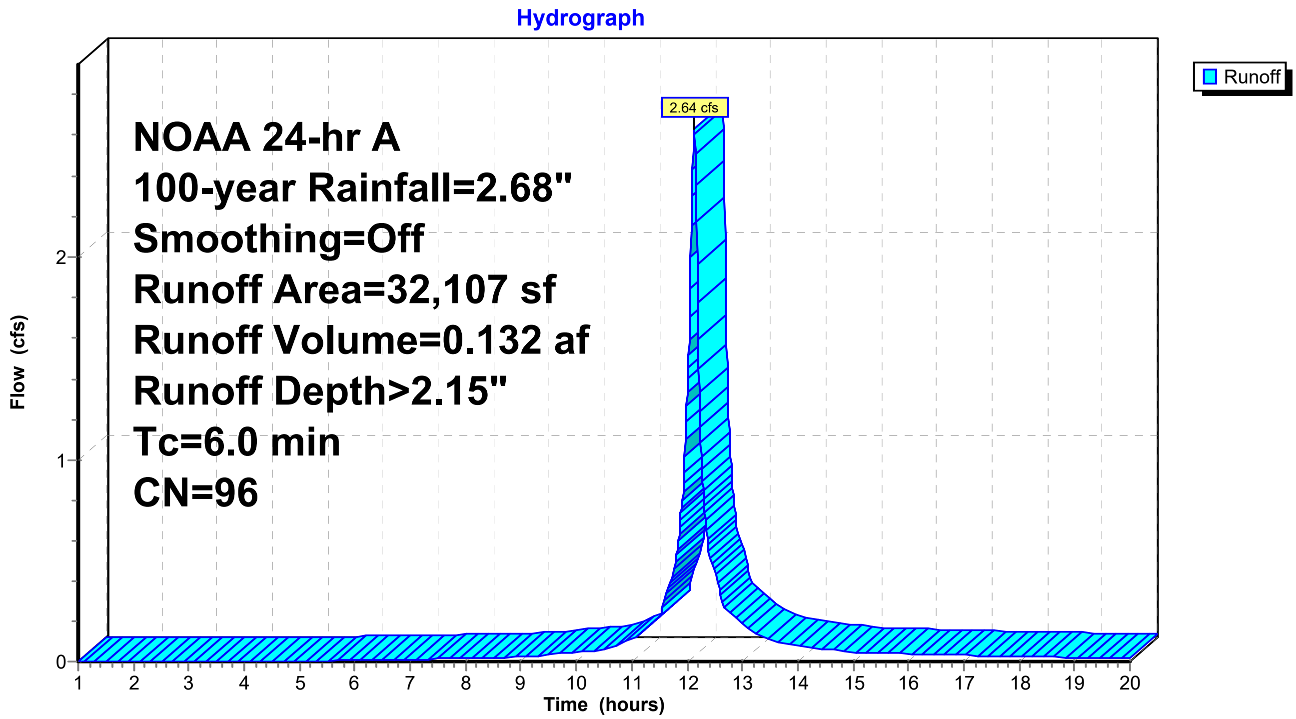
Runoff = 2.64 cfs @ 12.13 hrs, Volume= 0.132 af, Depth> 2.15"
Routed to Pond 18P : POND

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 1.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	4,410	98	
*	21,902	98	
*	5,795	86	
	32,107	96	Weighted Average
	5,795		18.05% Pervious Area
	26,312		81.95% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 2S: POSTDEVELOPMENT A



Summary for Subcatchment 19S: POST DEVELOPMENT B

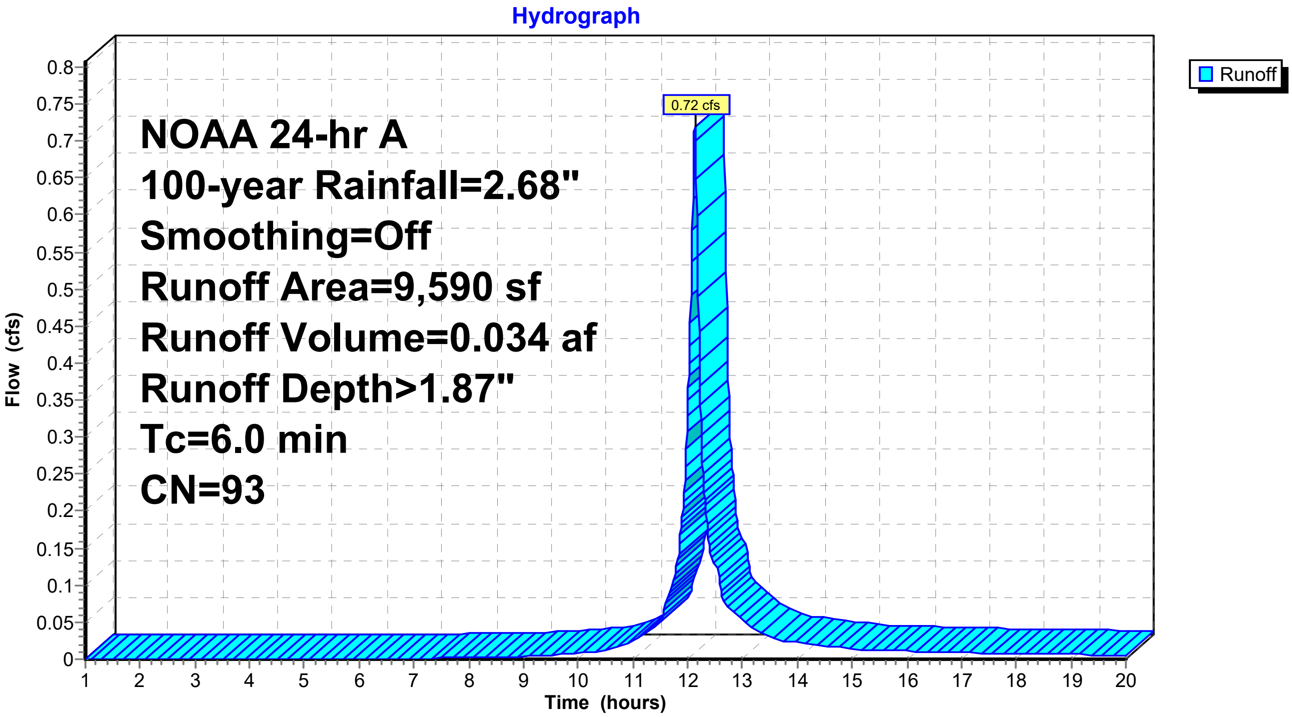
Runoff = 0.72 cfs @ 12.13 hrs, Volume= 0.034 af, Depth> 1.87"
Routed to Link 20L : TOTAL Post

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 1.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	5,537	98	
*	4,053	86	
	9,590	93	Weighted Average
	4,053		42.26% Pervious Area
	5,537		57.74% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 19S: POST DEVELOPMENT B



Summary for Subcatchment 21S: POSTDEVELOPMENT C

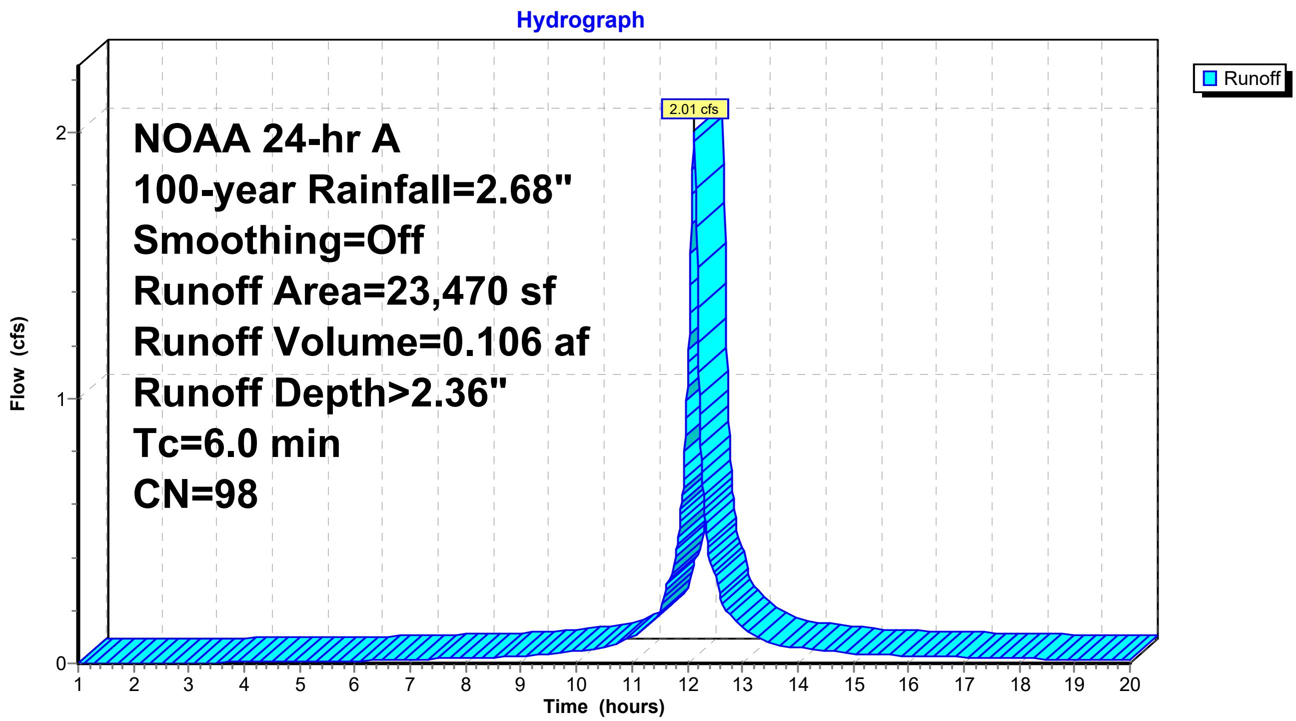
Runoff = 2.01 cfs @ 12.13 hrs, Volume= 0.106 af, Depth> 2.36"
Routed to Pond 18P : POND

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 1.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	23,470	98	
	23,470		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 21S: POSTDEVELOPMENT C



Summary for Pond 18P: POND

Inflow Area = 1.276 ac, 89.57% Impervious, Inflow Depth > 2.24" for 100-year event
Inflow = 4.65 cfs @ 12.13 hrs, Volume= 0.238 af
Outflow = 4.38 cfs @ 12.15 hrs, Volume= 0.191 af, Atten= 6%, Lag= 1.3 min
Discarded = 0.00 cfs @ 12.15 hrs, Volume= 0.003 af
Primary = 4.38 cfs @ 12.15 hrs, Volume= 0.188 af
Routed to Link 20L : TOTAL Post

Routing by Stor-Ind method, Time Span= 1.00-20.00 hrs, dt= 0.01 hrs
Peak Elev= 5,114.86' @ 12.15 hrs Surf.Area= 1,969 sf Storage= 2,690 cf

Plug-Flow detention time= 77.7 min calculated for 0.191 af (80% of inflow)
Center-of-Mass det. time= 35.5 min (778.7 - 743.2)

Volume	Invert	Avail.Storage	Storage Description
#1	5,113.00'	4,072 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

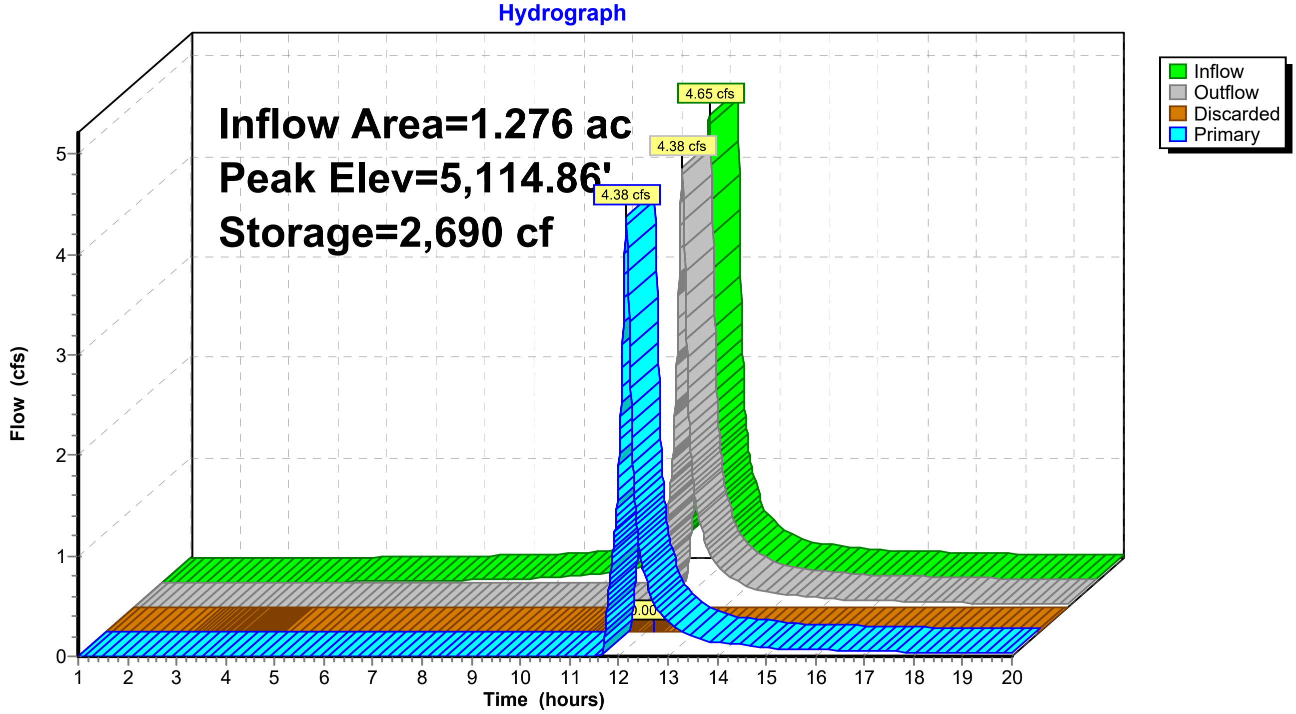
Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
5,113.00	955	0	0
5,114.00	1,466	1,211	1,211
5,115.00	2,050	1,758	2,969
5,115.50	2,365	1,104	4,072

Device	Routing	Invert	Outlet Devices
#1	Primary	5,114.50'	**8.0' long x 8.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 1.80 2.00 2.50 3.00 3.50 4.00 4.50 5.00 5.50 Coef. (English) 2.43 2.54 2.70 2.69 2.68 2.68 2.66 2.64 2.64 2.64 2.65 2.65 2.66 2.66 2.68 2.70 2.74
#2	Discarded	5,113.00'	**0.070 in/hr Exfiltration over Surface area** Conductivity to Groundwater Elevation = 4,000.00'

Discarded OutFlow Max=0.00 cfs @ 12.15 hrs HW=5,114.86' (Free Discharge)
└─**2=Exfiltration** (Controls 0.00 cfs)

Primary OutFlow Max=4.37 cfs @ 12.15 hrs HW=5,114.86' (Free Discharge)
└─**1=Broad-Crested Rectangular Weir** (Weir Controls 4.37 cfs @ 1.51 fps)

Pond 18P: POND

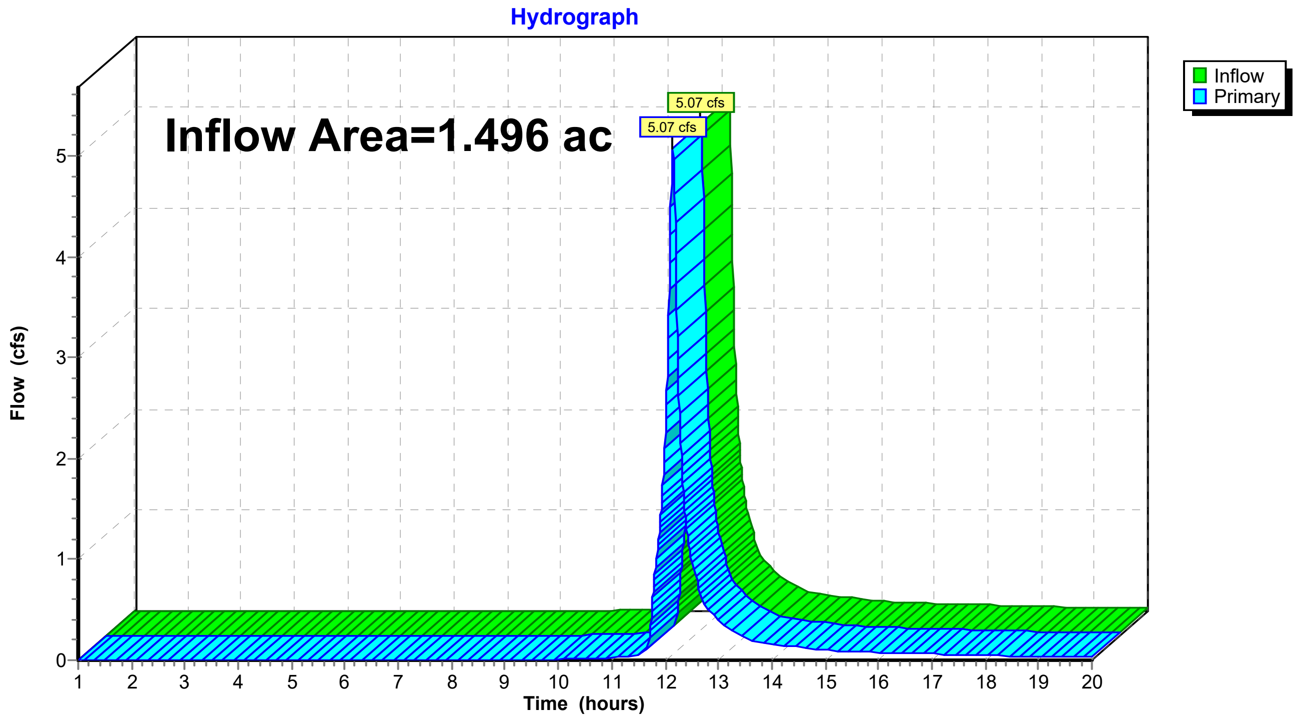


Summary for Link 20L: TOTAL Post

Inflow Area =	1.496 ac, 84.89% Impervious, Inflow Depth > 1.78"		for 100-year event
Inflow =	5.07 cfs @ 12.15 hrs, Volume=	0.222 af	
Primary =	5.07 cfs @ 12.15 hrs, Volume=	0.222 af, Atten= 0%, Lag= 0.0 min	

Primary outflow = Inflow, Time Span= 1.00-20.00 hrs, dt= 0.01 hrs

Link 20L: TOTAL Post



Appendix E: Web Soil Survey Map

United States Department of Agriculture

NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for Bernalillo County and Parts of Sandoval and Valencia Counties, New Mexico

May 16, 2024

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

The U.S. Department of Agriculture (USDA) prohibits discrimination in all its programs and activities on the basis of race, color, national origin, age, disability, and where applicable, sex, marital status, familial status, parental status, religion, sexual orientation, genetic information, political beliefs, reprisal, or because all or a part of an individual's income is derived from any public assistance program. (Not all prohibited bases apply to all programs.) Persons with disabilities who require alternative means for communication of program information (Braille, large print, audiotape, etc.) should contact USDA's TARGET Center at (202) 720-2600 (voice and TDD). To file a complaint of discrimination, write to USDA, Director, Office of Civil Rights, 1400 Independence Avenue, S.W., Washington, D.C. 20250-9410 or call (800) 795-3272 (voice) or (202) 720-6382 (TDD). USDA is an equal opportunity provider and employer.

Contents

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Soil Map

Map Scale: 1:663 if printed on A portrait (8.5" x 11") sheet.

Meters: 0 5 10 20 30

Feet: 0 30 60 120 180

N

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 13N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Bernalillo County and Parts of Sandoval and Valencia Counties, New Mexico
Survey Area Data: Version 18, Sep 7, 2023

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Nov 22, 2020—Jan 1, 2021

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background

MAP LEGEND	MAP INFORMATION
	imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
EtC	Embudo-Tijeras complex, 0 to 9 percent slopes	0.9	100.0%
Totals for Area of Interest		**0.9**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Bernalillo County and Parts of Sandoval and Valencia Counties, New Mexico

EtC—Embudo-Tijeras complex, 0 to 9 percent slopes

Map Unit Setting

National map unit symbol: 1vwt
Elevation: 2,700 to 7,000 feet
Mean annual precipitation: 5 to 16 inches
Mean annual air temperature: 48 to 70 degrees F
Frost-free period: 130 to 250 days
Farmland classification: Not prime farmland

Map Unit Composition

Embudo and similar soils: 50 percent
Tijeras and similar soils: 35 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Embudo

Setting

Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Concave
Across-slope shape: Linear
Parent material: Alluvium derived from igneous and sedimentary rock

Typical profile

H1 - 0 to 4 inches: gravelly fine sandy loam
H2 - 4 to 20 inches: gravelly sandy loam
H3 - 20 to 60 inches: stratified gravelly loamy coarse sand to very gravelly loamy sand

Properties and qualities

Slope: 0 to 5 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Runoff class: Very low
Capacity of the most limiting layer to transmit water (Ksat): High (2.00 to 6.00 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: Rare
Frequency of ponding: None
Calcium carbonate, maximum content: 7 percent
Maximum salinity: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum: 2.0
Available water supply, 0 to 60 inches: Low (about 3.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7e
Hydrologic Soil Group: A
Ecological site: R042BE051NM - Sandy, Cool Desert Grassland
Hydric soil rating: No

Description of Tijeras

Setting

Landform: Fan remnants
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Alluvium derived from igneous and sedimentary rock

Typical profile

H1 - 0 to 4 inches: gravelly fine sandy loam
H2 - 4 to 14 inches: sandy clay loam
H3 - 14 to 19 inches: gravelly sandy loam
H4 - 19 to 60 inches: stratified very gravelly sand to very gravelly sandy loam

Properties and qualities

Slope: 1 to 9 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Runoff class: Medium
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.60 to 2.00 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum content: 5 percent
Maximum salinity: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum: 2.0
Available water supply, 0 to 60 inches: Low (about 5.2 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7c
Hydrologic Soil Group: B
Ecological site: R042BE051NM - Sandy, Cool Desert Grassland
Hydric soil rating: No

Minor Components

Wink

Percent of map unit: 5 percent
Ecological site: R042BE052NM - Loamy, Cool Desert Grassland
Hydric soil rating: No

Millett

Percent of map unit: 5 percent
Ecological site: R035XG114NM - Gravelly
Hydric soil rating: No

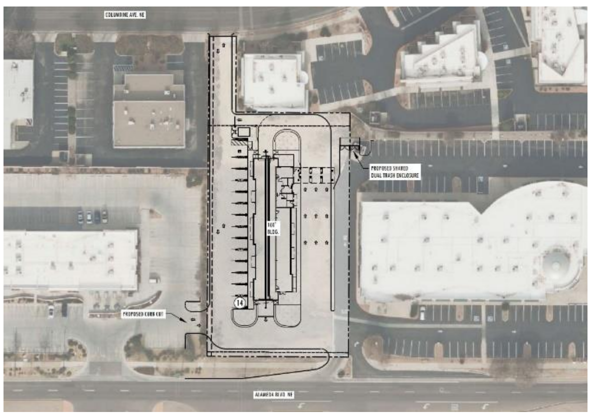
Tesajo

Percent of map unit: 5 percent
Ecological site: R035XG114NM - Gravelly
Hydric soil rating: No

Appendix F: Geotechnical Report & Stormwater Soil Evaluation

PARTNER
Engineering and Science, Inc.

More Than Just Assessments.
Solutions.



GEOTECHNICAL REPORT

Mister Car Wash – NM 2502 Fiesta Park

4703 Alameda Boulevard NE
Albuquerque, New Mexico 87113

Report Date:
July 11, 2023

Partner Project No.
23-412531.1

Prepared for:
Mister Car Wash
222 E. 5th Street
Tucson, Arizona 85705

Building Science

Environmental Consulting

Construction & Development

Energy & Sustainability

PARTNER

July 11, 2023

Mister Car Wash
Prabhs Matharoo
222 E. 5th Street
Tucson, Arizona 85705

Subject: **Geotechnical Report**
Mister Car Wash – NM 2502 Fiesta Park
4703 Alameda Boulevard NE
Albuquerque, New Mexico 87113
Partner Project No. 23-412531.1

Dear Prabhs Matharoo:

Partner Assessment Corporation (Partner) presents the following general opinion regarding the geotechnical conditions at the subject site, based on the information contained within this geotechnical report and our general experience with construction practices and geotechnical conditions on other sites. This statement does not constitute an engineering recommendation.

- *The geotechnical conditions on the site related to the planned construction are expected to be similar to less favorable in comparison with other similar sites*; given challenges associated with loose, potentially hydro-collapsible soil in the near surface.*

The descriptions and findings of our geotechnical report are presented for your use in this electronic format, for your use as shown in the hyperlinked outline below. To return to this page after clicking a hyperlink, hold "alt" and press the "left arrow key" on your keyboard.

We appreciate the opportunity to be of service during this phase of the work.

Sincerely,

MATTHEW WERNER MARCUS
NEW MEXICO
25227
PROFESSIONAL ENGINEER

Matthew Marcus, PE, PG
Principal Geotechnical Engineer/Geologist

Andrew J. Atry, PE*
Senior Engineer

Joey Masters
Joey Masters, GIT
Project Geologist

*Registered in Other States (CA, AZ, NV and Others) "

similar sites" refers to sites with similar planned and current use, where we have recently performed similar work, and is a general statement not based on statistical analysis.

1. GEOTECHNICAL EXECUTIVE SUMMARY

The executive summary is meant to consolidate information provided in more detail in the body of this report. This summary in no way replaces or overrides the detailed sections of the report.

Geologic Zones and Site Hazards

The site is located in the City of Albuquerque within the Basin and Range physiographic province of the state of New Mexico. According to the United States Geological Survey (USGS), surficial geology at the site can be described as Younger stream-valley alluvium. Stream-valley deposits generally consists of sand, muddy sand, and gravel. Site grades are relatively flat, gently sloping down towards the north. The site is currently undeveloped land. Based on review of historic aerials and topographic maps, the site has previously been undeveloped. As such, the site may be impacted by buried root balls and light vegetation. This portion of the state has a moderate seismic risk per the USGS 2014 Hazard Risk Map. According to the Federal Emergency Management Agency (FEMA) map, the subject property is located within an area of minimal flood hazard (Zone X). Near surface soils may be susceptible to hydro-collapse. No other hazards are known or suspected on the site.

Excavation Conditions

We anticipate excavations on the site to depths of up to 4 feet for building foundations and/or slabs on grade, and 5 feet for utility lines. Based on boring data, conventional construction equipment in good working condition should be able to perform the planned excavations. As previously mentioned, native loose sandy soil may be present on the site and could cave or be difficult to remove and require additional planning and equipment. Groundwater was not encountered on the site in our borings at the time of drilling. However, groundwater levels fluctuate over time and may be different at the time of construction and during the project life.

Foundation/Slab Support

We anticipate that the new building may be supported on conventional strip foundations and/or slabs on grade bearing on re-worked on site soils. Given the loose sandy nature of the on-site material, we recommend that the upper 5 feet of site material be over-excavated, moisture conditioned and recompacted below buildings and/or foundations, to create a uniform fill pad. This can be accomplished by overexciting to a depth of 4 feet below the existing grades then scarifying to a depth of 12-inches, moisture conditioning, and recompacting in place prior to adding in new compacted fill material. The new building foundations and slabs can be supported within this pad. Over excavation for the fill pad should extend a minimum of 5 feet beyond the limits of the proposed building. In structural areas, prior to the placement of new fills or pavements, we recommend the subgrade be proofrolled or otherwise evaluated and repaired under the direction of the engineer and should then be scarified to a depth of 12 inches or more, moisture-conditioned, and compacted in-place prior to the placement of fills or slabs on grade.

Soil Reuse

Based on our borings site soils will generally be usable as structural fill provided it is free of organic material and other debris. The addition of water to the onste sandy soils should be anticipated given the low moisture content. Moisture conditioning requirements should be evaluated at the time of construction. We recommend engineered fill for the site be moisture conditioned and compacted to at least 95% of the maximum dry density in accordance with ASTM D1557 and Appendix C of this report.

Pavement Design

Roadway Type	Subgrade Preparation	Pavement Section
Parking Area Light Duty	Proofrolled/Compacted Subgrade	6 in. Concrete/ 4 in. Aggregate Base
Parking Area Heavy Duty	Proofrolled/Compacted Subgrade	6 in. Concrete / 5 in. Aggregate Base

2. REPORT OVERVIEW & LIMITATIONS

2.1 Report Overview

To develop this report, Partner accessed existing information and obtained site specific data from our exploration program. Partner also used standard industry practices and our experience on previous projects to perform engineering analysis and provide recommendations for construction along with construction considerations to guide the methods of site development. The opinions on the cover letter of this report do not constitute engineering recommendations, and are only general, based on our recent anecdotal experiences and not statistical analysis. Section 1.0, Executive Geotechnical Summary, compiles data from each of the report sections, while each of sections in the report presents a detailed description of our work. The detailed descriptions in Section 5.0 and Appendix C constitute our engineering recommendations for the project, and they supersede the Executive Geotechnical Summary.

The report overview, including a description of the planned construction and a list of references, as well as an explanation of the report limitations is provided in Section 2.0. The findings of Partner's geologic review are included in Section 3.0 Geologic Conditions and Hazards. The descriptions of our methods of exploration and testing, as well as our findings are included in Section 4.0 Geotechnical Exploration and Laboratory Results. In addition, logs of our exploration excavations are included in Appendix A of the report, and laboratory testing is included in Appendix B of the report. Site Location and Site Plan maps are included as Figures in the report.

2.2 Assumed Construction

Partner's understanding of the planned construction was based on information provided by the project team. The proposed site plan is included as Figure 3 to this report. Partner's assumptions regarding the new construction are presented in the below table.

Property Data	
Property Use	Car Wash
Building Footprint/Height	Approximately 6,300 sf/ single-story at grade with car wash structure(s)
Land Acreage (Ac)	Approximately 1.06 acres
Number of Buildings	1
Expected Cuts and Fills	Up to 5 feet of cut for foundation and utility installation
Type of Construction	Assumed slab-on-grade with pre-engineered metal or masonry units
Foundations Type	Assumed conventional spread foundations and/or slab-on-grade
Anticipated Loads	Unknown, assumed 20-kip column loads and 4 kips/ft wall loads
Traffic Loading	Primarily vehicular traffic with occasional heavy truck traffic
Site Information Sources	Google Earth Pro and Client Provided Site Plan

2.3 References

The following references were used to generate this report:

Federal Emergency Management Agency, FEMA Flood Map Service Center, accessed 6/21/2023

Google Earth Pro (Online), accessed 6/21/2023

Historic Aerials by NETR Online, accessed 6/21/2023

New Mexico Geological Survey, The Geological Map of New Mexico (Online), accessed 6/21/2023

United States Geologic Survey, Earthquake Hazards Program (Online), accessed 6/21/2023

United States Geological Survey, Lower 48 States 2014 Seismic Hazard Map, accessed online 6/21/2023

United States Geologic Survey, Earthquake Hazards Tool (Online), accessed 6/21/2023

United States Geologic Survey, USGS US Topo 7.5-minute map for Alameda Quadrangle, New Mexico 2020: USGS -National Geospatial Operations Center (NGTOC)

2.4 Limitations

The conclusions, recommendations, and opinions in this report are based upon soil samples and data obtained in widely spaced locations that were accessible at the time of exploration and collected based on project information available at that time. Our findings are subject to field confirmation that the samples we obtained were representative of site conditions. If conditions on the site are different than what was encountered in our borings, the report recommendations should be reviewed by our office, and new recommendations should be provided based on the new information and possible additional exploration if needed. It should be noted that geotechnical subsurface evaluations are not capable of predicting all subsurface conditions, and that our evaluation was performed to industry standards at the time of the study, no other warranty or guarantee is made.

Likewise, our document review and geologic research study made a good-faith effort to review readily available documents that we could access and were aware of at the time, as listed in this letter. We are not able to guarantee that we have discovered, observed, and reviewed all relevant site documents and conditions. If new documents or studies are available following the completion of the report, the recommendations herein should be reviewed by our office, and new recommendations should be provided based on the new information and possible additional exploration if needed.

This report is intended for the use of the client in its entirety for the proposed project as described in the text. Information from this report is not to be used for other projects or for other sites. All the report must be reviewed and applied to the project or else the report recommendations may no longer apply. If pertinent changes are made in the project plans or conditions are encountered during construction that appear to be different than indicated by this report, please contact this office for review. Significant variations may necessitate a re-evaluation of the recommendations presented in this report. The findings in this report are valid for one year from the date of the report. This report has been completed under specific Terms and Conditions relating to scope, relying parties, limitations of liability, indemnification, dispute resolution, and other factors relevant to any reliance on this report.

If parties other than Partner are engaged to provide construction geotechnical special inspection services, they will also be required to assume construction geotechnical engineer of record (GEOR) services as well. To confirm this, they should issue a letter concurring with the findings and recommendations in this geotechnical design report or providing alternate recommendations prior to the start of construction. The GEOR should be directly involved in the construction process, provide engineering review the special inspection reports on a daily basis, and sign off at the end of the project that the construction was done per the geotechnical design report. If Partner is not the GEOR, we should be contacted as the design geotechnical engineer in the case of changed conditions or changes to the planned construction. Interpretation of the design geotechnical report during construction, response to project RFI's, and oversight of special inspectors and quality control testing is to be handled by the GEOR. Partner can provide a proposal for special inspection and GEOR services upon request.

3. GEOLOGIC CONDITIONS & HAZARDS

This section presents the results of a geologic review performed by Partner, for the proposed new construction on site. The general location of the project is shown on Figure 1.

3.1 Site Location and Project Information

The proposed construction will consist of a 6,300 sf, single-story building on an approximately 1.06-acre parcel of undeveloped land within a mixed-commercial and residential area of Albuquerque, New Mexico. The site is currently undeveloped land with light vegetation. The project site is bordered by a commercial building to the north, Alameda Boulevard NE followed by a commercial building to the south, a commercial building to the east, and a commercial building to the west. Figure 3 presents the project site and the locations of our explorations.

Based on our review of available documents, the site has had the following previous uses:

Historical Use Information		
Period/Date	**Source**	**Description/Use**
1951 – Present	Aerial Photographs, Topographic Maps, Onsite Observations	Undeveloped Land

3.2 Geologic Setting

The site is located in the City of Albuquerque within the Basin and Range physiographic province of the state of New Mexico. According to the United States Geological Survey (USGS), surficial geology at the site can be described as Younger stream-valley alluvium. Stream-valley deposits generally consists of sand, muddy sand, and gravel. Site grades are relatively flat, gently sloping down towards the north. The site is currently undeveloped land. Based on review of historic aerials and topographic maps, the site has previously been undeveloped. As such, the site may be impacted by buried root balls and light vegetation. This portion of the state has a moderate seismic risk per the USGS 2014 Hazard Risk Map. According to the Federal Emergency Management Agency (FEMA) map, the subject property is located within an area of minimal flood hazard (Zone X). Near surface soils may be susceptible to hydro-collapse. No other hazards are known or suspected on the site.

Based on information obtained from the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) *Web Soil Survey*, the subject property is mapped as Embudo-Tijeras complex. The Embudo series consists of deep, well drained, high permeable soils that formed from alluvium derived from igneous and sedimentary rock. Slopes range from 0 to 9 percent.

A general summary of the geologic data compiled for this project is provided in the below table.

Geologic Data		
Parameter	**Value**	**Source**
Geomorphic Region	Basin and Range	USGS
Ground Elevation	5117 - 5126 feet above MSL	USGS, Google Earth Pro
Flood Elevation	Zone X (0.2% Flood Hazard)	FEMA
Seismic Hazard Zone	Moderate	USGS
Geologic Hazards	Hydro-Collapsible Soils	USGS, CGS

Geologic Data		
Parameter	**Value**	**Source**
Surface Cover	Light Vegetation	Partner Borings
Surficial Geology	Younger Stream-Valley Alluvial	USGS
Depth to Bedrock	Not Encountered	Partner Borings
Groundwater Depth	Not Encountered	Partner Borings

3.3 Geologic Hazards

This region of New Mexico is susceptible to low to moderate ground shaking, liquefiable, and hydro-collapse susceptible soil. Based on review of the soil borings, laboratory data, and available hazard maps, the site is mapped in an area known for moderately hydro-collapsible and highly liquefiable susceptible soil. Laboratory analyses indicate that possible hydro-collapsible soils were encountered during our investigation. No other hazards are known or suspected on the site. We anticipate the new buildings will be designed using the current ICC code which is based on the American Society of Civil Engineers publication ASCE 7-16, "Minimum Design Loads and Associated Criteria for Buildings and Other Structures."

The seismic design parameters based on the USGS Design Maps Detailed Report for ASCE 7-16 Standard Method are presented below. State, County, City, and other jurisdictions in seismically active areas update seismic standards on a regular basis. The design team should carefully evaluate all of the building requirements for the project

Seismic Item	Value	Seismic Item	Value
Site Classification	E	Seismic Design Category	D
F_a	1.93	F_v	4.2
S_s	0.418g	S_1	0.136g
S_{MS}	0.807g	S_{M1}	0.572g
S_{DS}	0.538g	S_{D1}	0.382g
MCE_G PGA	0.187g	Design PGA (2/3 of MCE_G)	0.123g

4. GEOTECHNICAL EXPLORATION & LABORATORY RESULTS

Our evaluation of soils on the site included field exploration and laboratory testing. The field exploration and laboratory testing programs are briefly described below. Data reports from the field exploration and laboratory testing are provided in Appendix A and Appendix B, respectively.

4.1 Soil Borings

Subsurface materials and conditions at the site were investigated on June 13, 2023. Six (6) borings designated B-1 through B-6 and three (3) infiltration tests designated I-1 through I-3 were advanced by the use of a truck-mounted CME-55 drill rig using hollow-stem auger drilling techniques. The borings were advanced to depths of 6.5 and 21.5 feet below existing site grades. The approximate locations of the explorations are shown on Figure 3.

Logs of subsurface conditions encountered in the borings were prepared in the field by a representative of Partner Engineering. Soil samples consisting of Standard Penetration Tests (SPT) samples were collected at approximately 2.5 and 5-foot depth intervals and were returned to the laboratory for testing. The SPTs were performed in general accordance with ASTM D 1586. Typed boring logs were prepared from the field logs and are presented in Appendix A. A summary table description is provided below:

Surficial Geology		
Strata	**Depth to Bottom of Layer (bgs*)**	**Description**
Surface Cover	Thickness Varies	Light Vegetation
Native Stratum 1	Approx. 5 to 7 feet	Silty SAND (SM), Clayey SAND (SC), and Sandy CLAY (CL)
Native Stratum 2	Approx. 21.5 + feet	Silty SAND (SM)
Groundwater	Not Encountered	Partner Borings
Bedrock	Not Encountered	Partner Borings

bgs – below ground surface

4.2 Groundwater

Groundwater was not encountered at the time of drilling. However, groundwater levels fluctuate over time and may be different at the time of construction and during the life of the project.

4.3 Laboratory Evaluation

Selected samples collected during drilling activities were tested in the laboratory to assist in evaluating engineering properties of subsurface materials at the site. The results of laboratory analyses are presented in Appendix B.

4.4 Infiltration Testing

Three (3) infiltration tests were performed, as shown on Figure 3. The tests were performed at a depth of 4 feet. The testing was performed using the standard borehole percolation test method. The measured infiltration rates were calculated using the standard Bernalillo County methods and are reported below and are unfactored. The civil engineer should apply the proper reduction factors or factors of safety based on the type of system used. Data is shown in Appendix A, and is summarized below:

Test Number	I-1	I-2	I-3
Location	See Figure 3	See Figure 3	See Figure 3
Test Depth	4 ft	4 ft	4 ft
Final Water Drop	0.8 in.	0.3 in.	0.3in.
Un-factored Infiltration Rate	0.07 in./hr	0.04 in./hr	0.07 in./hr

5. GEOTECHNICAL RECOMMENDATIONS & PARAMETERS

The following discussion of findings for the site is based on the assumed construction, geologic review, results of the field exploration, and laboratory testing programs. The recommendations of this report are contingent upon adherence to Appendix C of this report, General Geotechnical Design and Construction Considerations. For additional details on the below recommendations, please see Appendix C.

5.1 Geotechnical Recommendations

The proposed construction is generally feasible from a geotechnical perspective provided the recommendations and assumptions of this report are followed.

Geologic/General Site Considerations

- The site is located in the City of Albuquerque within the Basin and Range physiographic province of the state of New Mexico. According to the United States Geological Survey (USGS), surficial geology at the site can be described as Younger stream-valley alluvium. Stream-valley deposits generally consists of sand, muddy sand, and gravel. Site grades are relatively flat, gently sloping down towards the north. The site is currently undeveloped land. Based on review of historic aerials and topographic maps, the site has previously been undeveloped. As such, the site may be impacted by buried root balls and light vegetation. This portion of the state has a moderate seismic risk per the USGS 2014 Hazard Risk Map. According to the Federal Emergency Management Agency (FEMA) map, the subject property is located within an area of minimal flood hazard (Zone X). Near surface soils may be susceptible to hydro-collapse. No other hazards are known or suspected on the site.
- Given the presence of the site in a moderately seismically active area, ground shaking during earthquakes should be anticipate during the project life. State, County, City, and other jurisdictions in seismically active areas update seismic standards on a regular basis. The design team should carefully evaluate all of the building requirements for the project.

Excavation Considerations

- We anticipate excavations on the site to depths of up to 4 feet for building foundations and/or slabs on grade, and 5 feet for utility lines. Based on boring data, conventional construction equipment in good working condition should be able to perform the planned excavations. As previously mentioned, native loose sandy soil may be present on the site and could cave or be difficult to remove and require additional planning and equipment.
- Groundwater was not encountered at the time of drilling. However, groundwater levels fluctuate over time and may be different at the time of construction and during the life of the project.
- Excavations should be sloped and/or shored to protect worker safety and adjacent properties, per OSHA and local guidelines and the presence of existing utilities should be thoroughly and carefully checked prior to digging. Appendix C further discusses excavation recommendations in the following sections, which can be accessed by clicking hyperlinks: Earthwork, Underground Pipeline, Excavation De-Watering.

Building Foundations

- We anticipate that the new building may be supported on conventional strip foundations and/or slabs on grade bearing on re-worked on site soils. Given the loose sandy nature of the on-site material, we recommend that the upper 5 feet of site material be over-excavated, moisture conditioned and recompacted below buildings and/or foundations, to create a uniform fill pad. This can be accomplished by overexciting to a depth of 4 feet below the existing grades then scarifying to a depth of 12-inches, moisture conditioning, and recompacting in place prior to adding in new compacted fill material. The new building foundations and slabs can be supported within this pad. Over excavation for the fill pad should extend a minimum of 5 feet beyond the limits of the proposed building. In structural areas, prior to the placement of new fills or pavements, we recommend the subgrade be proofrolled or otherwise evaluated and repaired under the direction of the engineer and should then be scarified to a depth of 12 inches or more, moisture-conditioned, and compacted in-place prior to the placement of fills or slabs on grade.
- Section 5.2 of this report provides a table outlining the embedment depth, bearing capacity, settlement and other parameters for foundation design and construction.

On-Grade Construction Considerations

- In new fill, structural, and pavement areas, cleaned subgrade should be proofrolled and evaluated by the engineer with a loaded water truck (4,000 gallon) or equivalent rubber-tired equipment. In locations where proofrolling is not feasible, probing, dynamic cone penetration testing, or other methods may be employed. Soft or unstable areas should be repaired per the direction of the engineer. Once approved, the subgrade soil should be scarified to a depth of 12 inches, moisture conditioned, and compacted as engineered fill. Improvements in these areas should extend laterally beyond the new structure limits 2 feet or a distance equal to or greater than the layer thickness, whichever is greater. This zone should extend vertically from the bearing grade elevation to the base of the fill. The thicknesses of the layer, settlement estimates, and modulus values are provided on the design tables in the next section.
- Based on our borings, we anticipate that some over-excavation may result from proofrolling operations. In areas where unsuitable subgrade conditions are encountered, we recommend an engineer be called to perform an evaluation of the issue and to propose a resolution. Such resolutions may include but are not limited to the use of geotextiles, chemical treatments (soil cement, hydrated lime, etc.) thickened slabs or pavements sections, lime-treated aggregate base, or others. Pavement sections provided in Section 5.2 are based on approved, compacted in-place soils being used in the subgrade. If subgrade conditions in the upper 3 feet of pavement areas vary or are improved, the pavement sections may be modified.
- Appendix C provides additional recommendations for earthwork/on-grade construction in the following sections: Cast-in-place Concrete, Foundations, Earthwork, Paving, Subgrade Preparation which can be accessed by clicking the hyperlinks.

Soil Reuse Considerations

- Based on our borings site soils will generally be usable as structural fill provided it is free of organic material and other debris. The addition of water to the onste sandy soils should be anticipated given the low moisture content. Moisture conditioning requirements should be evaluated at the time of construction. We recommend engineered fill for the site be moisture conditioned and compacted to at least 95% of the maximum dry density in accordance with ASTM D698 and Appendix C of this report.
- Appendix C provides additional recommendations for foundations in the following sections: EARTHWORK, SUBGRADE PREPARATION which can be accessed by clicking the hyperlinks.

Geotechnical Concrete and Steel Construction Considerations

- Soil/rock may be corrosive to concrete. We recommend using corrosion resistant concrete (*e.g.* Type II/V Portland Cement, a fly ash mixture of 25 percent cement replacement, and a water/cement ratio of 0.45 or less) as directed by the producer, engineer or other qualified party based on their knowledge of the materials and site conditions. Concrete exposed to freezing weather should be air entrained. Mix designs should be well-established and reviewed by the project engineers prior to placement, to verify the design is appropriate to meet the project needs and parameters provided in this report. Quality control testing should be performed to verify appropriate mixes are used and are properly handled and placed. Please refer to Appendix C, Cast In-Place Concrete for more details. USDA maps indicate site soils have a low corrosion potential for concrete.
- Concrete exposed to freezing weather should be air entrained. Mix designs should be well-established and reviewed by the project engineers prior to placement, to verify the design is appropriate to meet the project needs and parameters provided in this report. Quality control testing should be performed to verify appropriate mixes are used and are properly handled and placed. Please refer to Appendix C, Cast In-Place Concrete for more details.
- Soil/rock may be corrosive to un-protected metallic elements such as pipes, poles, rebar, etc. We recommend the use of coatings and/or cathodic protection for metals in contact with the ground, as directed by the product manufacturer, engineer or other qualified party based on their knowledge of the materials to be used and site soil conditions. USDA maps indicate site soils have a high corrosion potential for steel.

Site Storm Water Considerations

- Surface drainage and landscaping design should be carefully planned to protect the new structures from erosion/undermining, and to maintain the site earthwork and structure subgrades in a relatively consistent moisture condition. Water should not flow towards or pond near to new structures, and high water-demand plants should not be planned near to structures. Appendix C provides additional recommendations for foundations in the following sections: SITE GRADING AND DRAINAGE, WATER PROOFING which can be accessed by clicking the hyperlinks.
- We recommend consulting with the landscape designer and civil engineer regarding management of site storm water and irrigation water, as changes in moisture content below the site after construction will lead to soil movement and potential distress to the building.

5.2 Geotechnical Parameters

Based on the findings of our field and laboratory testing, we recommend that design and construction proceed per industry accepted practices and procedures, as described in Appendix C, General Geotechnical Design and Construction Considerations (Considerations).

Prepared Subgrade Parameters – (hyperlink to Construction Considerations)

Prepared Subgrade Parameters				
Structure	**Design Values**	**Cover Depth**	**Bearing Surface [a]**	**Static Settlement [d]**
Slab on Grade (Reinforced with #3 bars spaced 18 inches O.C. or equivalent)	k=100 pci [b] q_{all} = 125 psf[c] μ = 0.35	N/A	Within fill pad as discussed in section 5.1	<1 inch
Isolated Spread Foundations Max Load – 20 kips	q_{all} = 3.5 ksf[c] μ = 0.35	18-inches	Within fill pad as discussed in section 5.1	<1 inch
Continuous Spread Foundations Max Load 4 kips/ft	q_{all} = 3.5 ksf[c] μ = 0.35	18-inches	Within fill pad as discussed in section 5.1	<1 inch

[a] Repairs in bearing surface areas should be structural fill per the recommendation of the Earthwork section of Appendix C that is moisture conditioned to within 3 percent below to optimum moisture content and compacted to 95 percent or more of the soil maximum dry density per ASTM D698. Expansive material should not be located within the upper 3 feet of the soil subgrade.

[b] Subgrade modulus value "k", assuming the grade slab is supported by aggregate layer roughly equal to slab thickness (minimum 4 inches), as required for capillary break.

[c] Can be increased by 1/3 for temporary loading such as seismic and wind, allowable parameters, estimated FS of 2.5.

[d] Differential settlement is expected to be half to ¾ of total settlement.

Pavement Design and Construction Recommendations

- In our experience we recommend that multiple different pavement sections be considered for the project for economic and performance reasons. For loading docks and trash enclosures we recommend that thickened reinforced concrete pavement be utilized. For heavily used and ADA parking spaces, etc., we recommend the use of thinner reinforced concrete pavement. For the main drives of the parking lot, we recommend a heavy-duty asphalt pavement section, and thinner sections can be used in the parking field if any. We recommend concrete pavements consist of local DOT, or otherwise jurisdictionally approved mixes, and that paving cross slopes, curbs, and other features conform to the applicable local standard specifications and details.
- Depending on the planned changes to site grading, and the availability of clean granular soil, different pavement sections would be appropriate. These can also be adjusted using treatment using soil cement. The following sections are provided for native soil subgrade conditions. If imported fill is used, the section may need to be adjusted. This information assumes that construction will proceed per the provided Construction Considerations, presented in Appendix C.

Paving Structural Sections – (hyperlink to Construction Considerations)

Pavement Sections		
Roadway Type	**Subgrade Preparation [a]**	**Pavement Section [b]**
Parking Area Light Duty	Proofrolled/Compacted Subgrade	6 in. Concrete/ 4 in. Aggregate Base
Parking Area Heavy Duty	Proofrolled/Compacted Subgrade	6 in. Concrete / 5 in. Aggregate Base
Trash enclosure/Dumpster Pad	Proofrolled/Compacted Subgrade	7 in. Concrete / 4 in. Aggregate Base

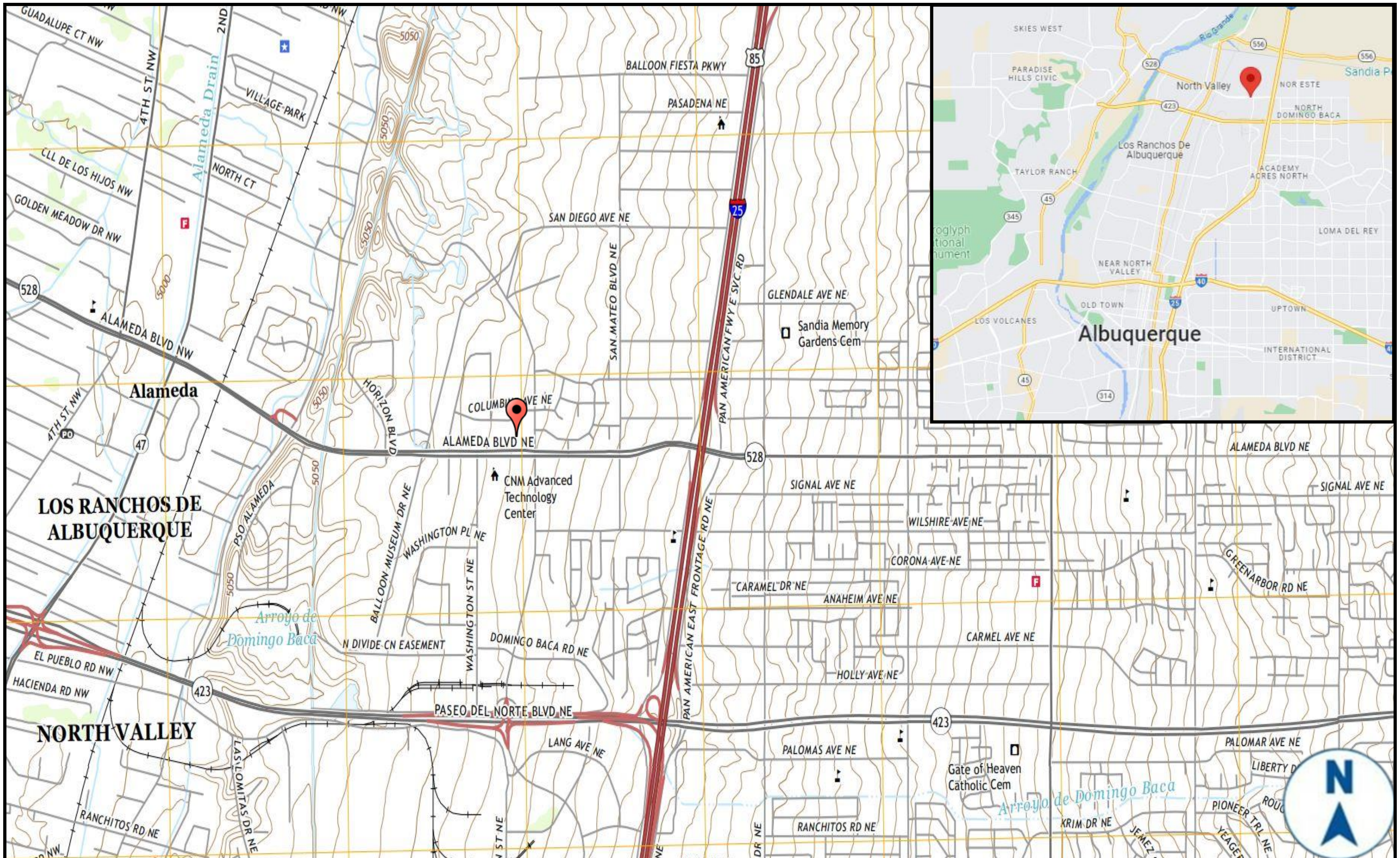
[a] Repairs in proof rolled areas should be structural fill per the recommendation of the Earthwork (hyperlink to Construction Considerations) that is moisture conditioned to within 3 percent above to optimum moisture content and compacted to 95 percent or more of the soil maximum dry density per ASTM D1557.

[b] 1 inch of pavement may be reduced if 6-in of lime or cement-treated soil is used with a 500 psi 28-day compressive strength. Soils with Plasticity Index of 10 or more are generally candidates for lime treatment, other soils are candidates for cement treatment, if any.

FIGURES

- Site Vicinity Plan
- Approximate Site Limits
- Boring Location Plan
- Geologic Map

PARTNER

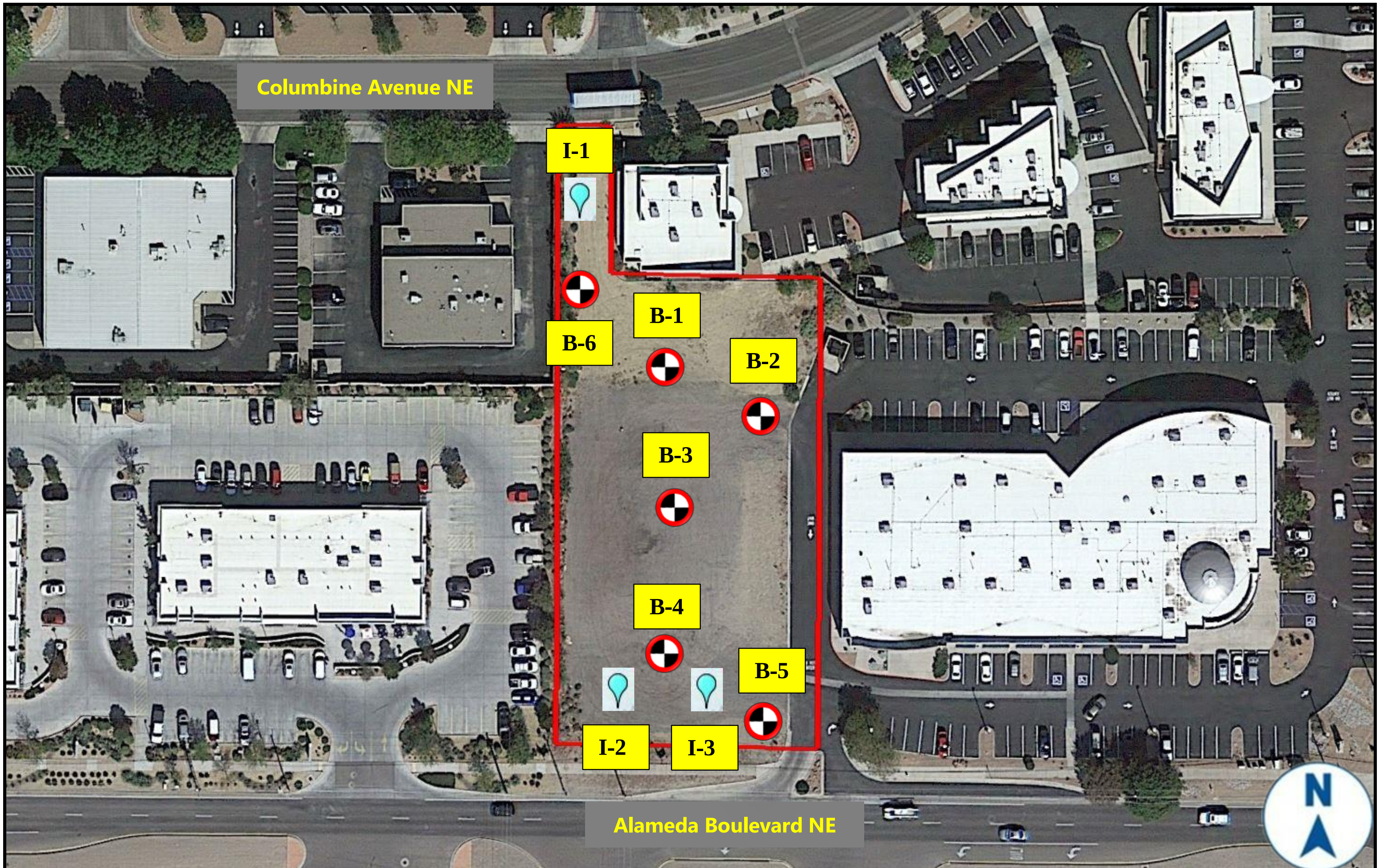


Source: U.S. Geological Survey, USGS US Topo 7.5-minute map for Alameda Quadrangle, New Mexico-Sandoval County. 2020: USGS - National Geospatial Technical Operations Center (NGTOC)

FIGURE 1 – SITE VICINITY PLAN

KEY

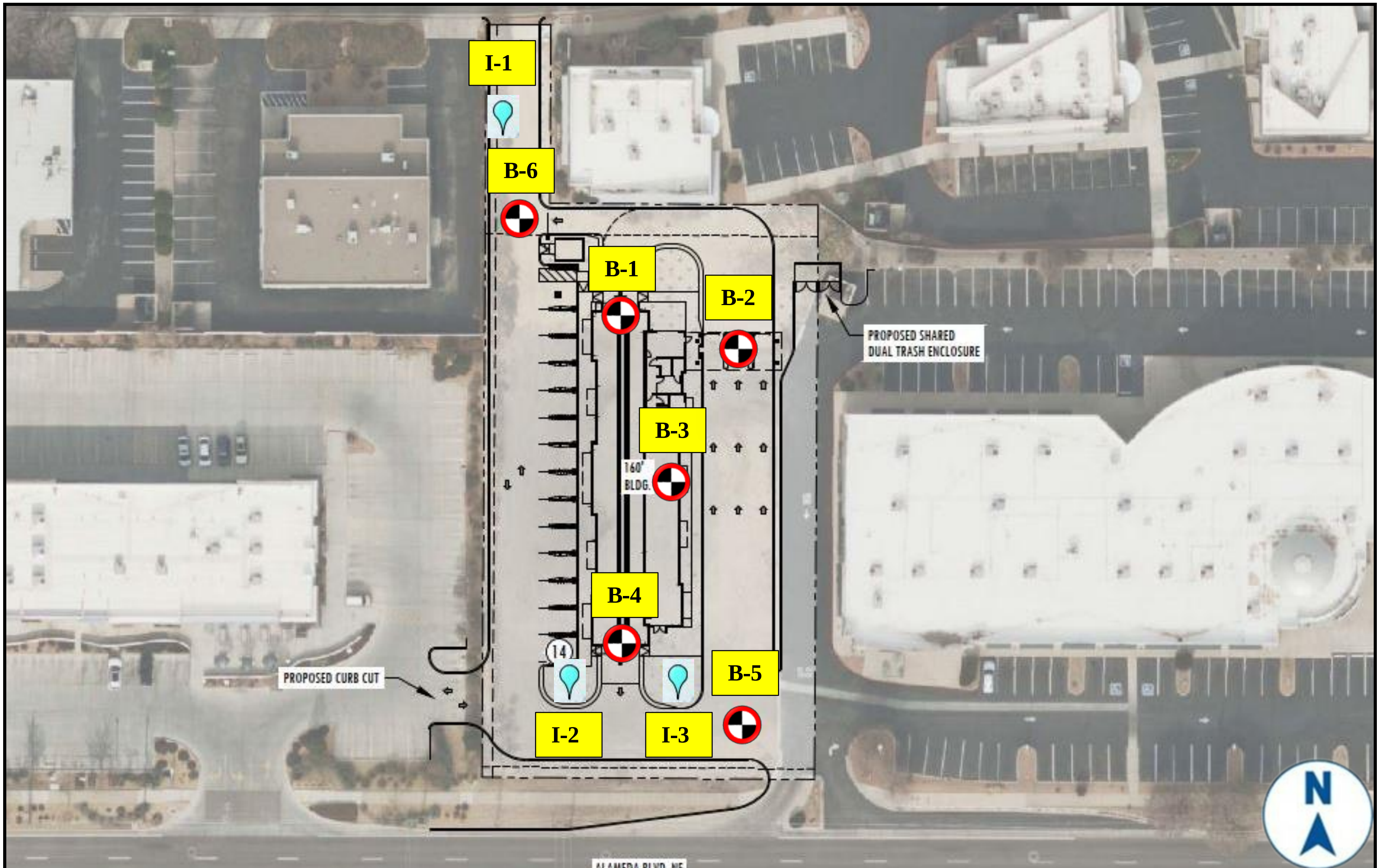
Approximate Site Location



Source: Google Earth Pro

FIGURE 2 – APPROXIMATE SITE LIMITS

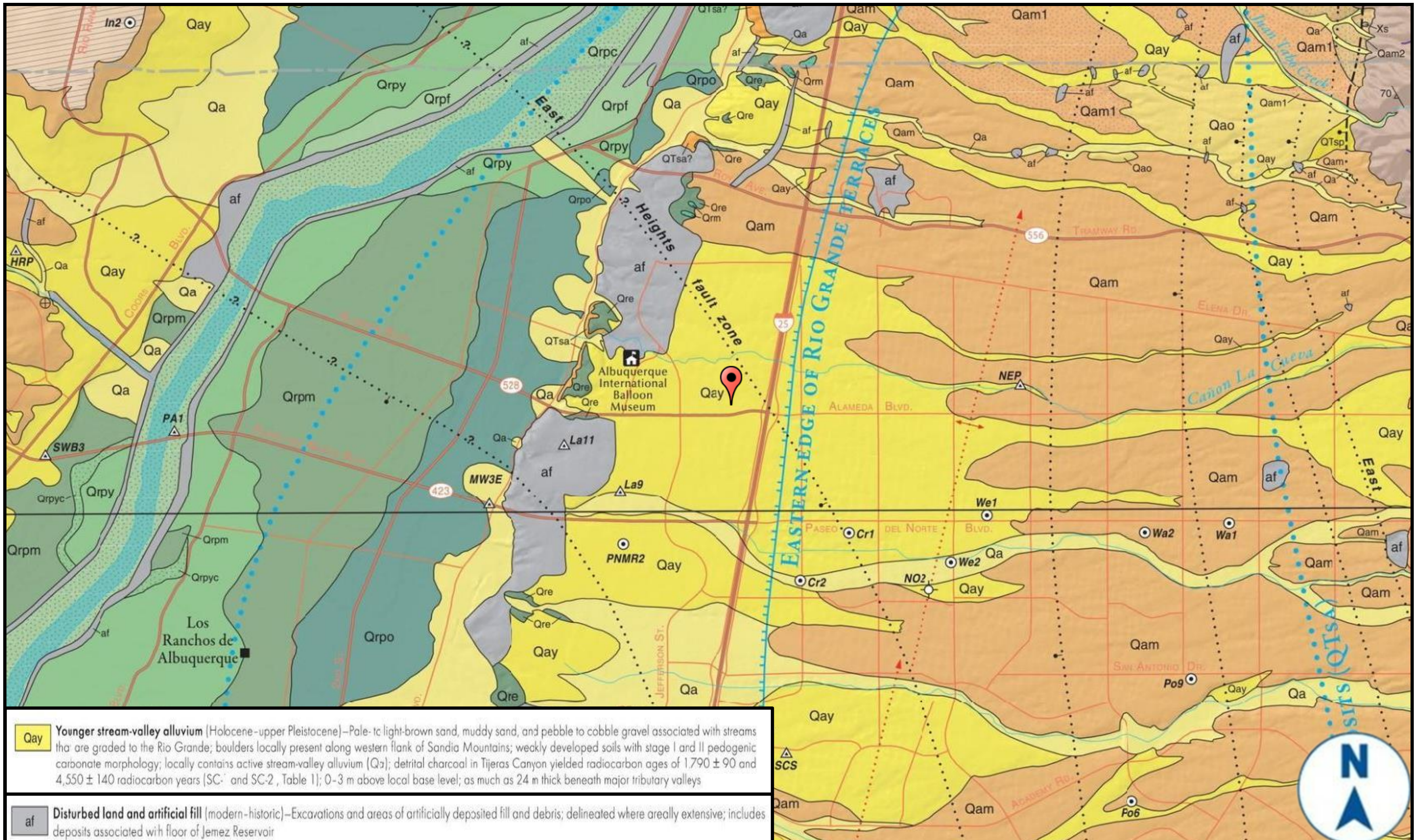
KEY Approximate Project Site Limits Approximate Boring Location Approximate Infiltration Test Location



Source: Site Plan – Mister Car Wash – NM 2502 Fiesta Park, dated May 10, 2023

FIGURE 3 – BORING LOCATION PLAN

KEY Approximate Boring Location Approximate Infiltration Test Location



Source: Geologic Map of the Albuquerque – Rio Rancho Metropolitan Area and Vicinity, Bernalillo and Sandoval Counties, New Mexico, 2008, S.D. Connell, scale 1:50,000.

FIGURE 4 – GEOLOGIC MAP

KEY

Approximate Site Location

APPENDIX A

Boring Logs

PARTNER

BORING LOG KEY - EXPLANATION OF TERMS

SURFACE COVER: General discription with thickness to the inch, ex. Topsoil, Concrete, Asphalt, etc,

FILL: General description with thickness to the 0.5 feet. Ex. Roots, Debris, Processed Materials (Pea Gravel, etc.)

NATIVE GEOLOGIC MATERIAL: Deposit type, 1.Color, 2.moisture, 3.density, 4.SOIL TYPE, other notes - Thickness to 0.5 feet

1. Color - Generalized
Light Brown (usually indicates dry soil, rock, caliche)
Brown (usually indicates moist soil)
Dark Brown (moist to wet soil, organics, clays)
Reddish (or other bright colors) Brown (moist, indicates some soil development/or residual soil)
Greyish Brown (Marine, sub groundwater - not the same as light brown above)
Mottled (brown and gray, indicates groundwater fluctuations)

2. Moisture
dry - only use for wind-blown silts in the desert
damp - soil with little moisture content
moist - near optimum, has some cohesion and stickyness
wet - beyond the plastic limit for clayey soils, and feels wet to the touch for non clays
saturated - Soil below the groundwater table, sampler is wet on outside

3A. Relative Density for Granular Soils

Relative Density	Ring	SPT
very loose	0-7	0-4
loose	7-14	4-10
medium dense	14-28	10-30
dense	28-100	30-50
very dense	100+	Over 50

3B. Consistency of Fine-Grained Cohesive Soils

Consistnecy	SPT	Undrained Shear Strength, tsf
very soft	0-2	less than 0.125
soft	2-4	0.125 - 0.25
medium stiff	4-8	0.25 - 0.50
stiff	8-15	0.50 - 1.0
very stiff	15-30	1.0 - 2.0
hard	Over 30	Over 2.0

4. Classification
Determine percent Gravel (Material larger than the No. 4 Sieve)
Determine percent fines (Material passing the No. 200 Sieve)
Determine percent sand (Passing the No. 4 and retained on the No. 200 Sieve)
Determine if clayey (make soil moist, if it easily roll into a snake it is clayey)

Coarse Grained Soils (Less than 50% Passing the No. 200 Sieve)

GP	SP	Mostly sand and gravel, with less than 5 % fines	sandy GRAVEL	SAND
GP-GM	SP-SM	Mostly sand and gravel 5-12% fines, non-clayey	sandy GRAVEL with silt	SAND with Silt
GP-GC	SP-SC	Mostly sand and gravel 5-12% fines, clayey	sandy GRAVEL with clay	SAND with clay
GC	SC	Mostly sand and gravel >12% fines clayey	clayey GRAVEL	clayey SAND
GM	SM	Mostly sand and gravel >12% fines non-clayey	silty GRAVEL	silty SAND

Fine Grained Soils (50% or more passes the No. 200 Sieve)

ML	Soft, non clayey	SILT with sand
MH	Very rare, holds a lot of water, and is pliable with very low strength	high plasticity SILT
CL	If sandy can be hard when dry, will be stiff/plastic when wet	CLAY with sand/silt
CH	Hard and resiliant when dry, very strong/sticky when wet (may have sand in it)	FAT CLAY

H = Liquid limit over 50%, L - LL under 50%
C = Clay
M = Silt

Samplers
S = Standard split spoon (SPT)
R = Modified ring
Bulk = Excavation spoils
ST = Shelby tube
C = Rock core

Boring Number:	B-1	Boring Log Page 1 of 2	
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
0				SURFACE COVER: TOPSOIL/Light Vegetation (Thickness Varies)
0.5				
1				
1.5			SM	NATIVE: Brown, moist, loose, Silty SAND, fine-grained
2	S	6		(Moisture: 6.7%)
2.5				
3				
3.5				
4				
4.5				
5	S	11	SC	Brown, moist, medium dense, Clayey SAND with Silt, fine-grained
5.5				(Moisture: 11.2%, Fines: 48.2%, PI: 16, LL: 28, PL: 12)
6				
6.5				
7	S	11	SM	Light brown, moist, medium dense, Silty SAND, fine-grained
7.5				
8				
8.5				
9				
9.5				
10	S	9		---Loose
10.5				(Moisture: 2.4%)
11				
11.5				
12				
12.5				
13				
13.5				
14				
14.5				
15	S	12		---Medium dense
15.5				
16				
16.5				
17				
17.5				
18				
18.5				
19				
19.5				
20	S	8		(Continued on next page)

Boring Number:	B-1	Boring Log Page 2 of 2	
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
20	S	8	SM	Light brown, moist, loose, Silty SAND, fine-grained
20.5				
21				
21.5				Boring terminated at 21.5 feet below the ground surface
22				Boring backfilled with soil cuttings upon completion
22.5				Groundwater not encountered (6/13/2023)
23				
23.5				
24				
24.5				
25				
25.5				
26				
26.5				
27				
27.5				
28				
28.5				
29				
29.5				
30				
30.5				
31				
31.5				
32				
32.5				
33				
33.5				
34				
34.5				
35				
35.5				
36				
36.5				
37				
37.5				
38				
38.5				
39				
39.5				
40				

Boring Number:	B-2		Boring Log Page 1 of 2		
Location:	See Figure 3		Date Started:	6/13/2023	
Site Address:	4703 Alameda Boulevard NE		Date Completed:	6/13/2023	
	Albuquerque, New Mexico 87113		Depth to Groundwater:	N/E	
Project Number:	23-412531.1		Field Technician	Enviro-Drill	
Drill Rig Type:	CME-75		Partner Engineering and Science		
Sampling Equipment:	Hollow Stem Auger/ SPT		600 Grant Street, Suite 450		
Borehole Diameter:	6"		Denver, Colorado 80203		

Depth, FT	Sample	N-Value	USCS	Description
0				SURFACE COVER: TOPSOIL/Light Vegetation (Thickness Varies)
0.5				
1				
1.5			SC	NATIVE: Brown, moist, loose, Clayey SAND with Silt, fine-grained
2	S	6		(Moisture: 4.8%, Fines: 44.9%)
2.5				
3				
3.5				
4				
4.5				
5	S	15		---Medium dense
5.5				
6				
6.5				
7	S	19	SM	Light brown, moist, medium dense, Silty SAND, fine-grained
7.5				(Moisture: 4.8%, Fines: 26.7%)
8				
8.5				
9				
9.5				
10	S	20		(Moisture: 3.4%)
10.5				
11				
11.5				
12				
12.5				
13				
13.5				
14				
14.5				
15	S	16		
15.5				
16				
16.5				
17				
17.5				
18				
18.5				
19				
19.5				
20	S	12		(Continued on next page)

Boring Number:	B-2	Boring Log Page 2 of 2	
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
20	S	12	SM	Light brown, moist, medium dense, Silty SAND, fine-grained
20.5				
21				
21.5				Boring terminated at 21.5 feet below the ground surface
22				Boring backfilled with soil cuttings upon completion
22.5				Groundwater not encountered (6/13/2023)
23				
23.5				
24				
24.5				
25				
25.5				
26				
26.5				
27				
27.5				
28				
28.5				
29				
29.5				
30				
30.5				
31				
31.5				
32				
32.5				
33				
33.5				
34				
34.5				
35				
35.5				
36				
36.5				
37				
37.5				
38				
38.5				
39				
39.5				
40				

Boring Number:	B-3	Boring Log Page 1 of 2	
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
0				SURFACE COVER: TOPSOIL/Light Vegetation (Thickness Varies)
0.5				
1				
1.5			SM	NATIVE: Brown, damp, medium dense, Silty SAND, fine-grained
2	S	13		(Moisture: 2.3%)
2.5				
3				
3.5				
4				
4.5				
5	S	20	CL	Brown, moist, very stiff, Sandy CLAY
5.5				(Moisture: 5.1%, Fines: 55.5%)
6				
6.5				
7	S	10	SM	Light brown, damp, loose to medium dense, Silty SAND, fine-grained
7.5				
8				
8.5				
9				
9.5				
10	S	13		---Medium dense
10.5				(Moisture: 2.5%, Fines: 17.2%)
11				
11.5				
12				
12.5				
13				
13.5				
14				
14.5				
15	S	22		
15.5				
16				
16.5				
17				
17.5				
18				
18.5				
19				
19.5				
20	S	19		(Continued on next page)

Boring Number:	B-3	Boring Log Page 2 of 2	
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
20	S	19	SM	Light brown, damp, medium dense, Silty SAND, fine-grained
20.5				
21				
21.5				Boring terminated at 21.5 feet below the ground surface
22				Boring backfilled with soil cuttings upon completion
22.5				Groundwater not encountered (6/13/2023)
23				
23.5				
24				
24.5				
25				
25.5				
26				
26.5				
27				
27.5				
28				
28.5				
29				
29.5				
30				
30.5				
31				
31.5				
32				
32.5				
33				
33.5				
34				
34.5				
35				
35.5				
36				
36.5				
37				
37.5				
38				
38.5				
39				
39.5				
40				

Boring Number:	B-4	Boring Log Page 1 of 2	
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
0				SURFACE COVER: TOPSOIL/Light Vegetation (Thickness Varies)
0.5				
1				
1.5			SC	NATIVE: Brown, moist, loose, Clayey SAND with Silt, fine-grained
2	S	6		(Moisture: 6.3%, Fines: 42.9%)
2.5				
3				
3.5				
4				
4.5				
5	S	8		
5.5				
6				
6.5				
7	S	13		(Moisture: 6.9%, Fines: 22.4%)
7.5				
8				
8.5				
9				
9.5				
10	S	12	SM	Light brown, damp, medium dense, Silty SAND, fine-grained
10.5				
11				
11.5				
12				
12.5				
13				
13.5				
14				
14.5				
15	S	15		(Moisture: 3.2%)
15.5				
16				
16.5				
17				
17.5				
18				
18.5				
19				
19.5				
20	S	21		(Continued on next page)

Boring Number:	B-4	Boring Log Page 2 of 2	
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
20	S	21	SM	Light brown, damp, medium dense, Silty SAND, fine-grained
20.5				
21				
21.5				Boring terminated at 21.5 feet below the ground surface
22				Boring backfilled with soil cuttings upon completion
22.5				Groundwater not encountered (6/13/2023)
23				
23.5				
24				
24.5				
25				
25.5				
26				
26.5				
27				
27.5				
28				
28.5				
29				
29.5				
30				
30.5				
31				
31.5				
32				
32.5				
33				
33.5				
34				
34.5				
35				
35.5				
36				
36.5				
37				
37.5				
38				
38.5				
39				
39.5				
40				

Boring Number:	B-5		Boring Log Page 1 of 1
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
0				SURFACE COVER: TOPSOIL/Light Vegetation (Thickness Varies)
0.5				
1				
1.5			SC	NATIVE: Brown, damp, loose, Clayey SAND with Silt, fine-grained
2	S	6		(Moisture: 3.1%)
2.5				
3				
3.5				
4				
4.5				
5	S	7		
5.5				
6				
6.5				Boring terminated at 6.5 feet below the ground surface
7				Boring backfilled with soil cuttings upon completion
7.5				Groundwater not encountered (6/13/2023)
8				
8.5				
9				
9.5				
10				
10.5				
11				
11.5				
12				
12.5				
13				
13.5				
14				
14.5				
15				
15.5				
16				
16.5				
17				
17.5				
18				
18.5				
19				
19.5				
20				

Boring Number:	B-6	Boring Log Page 1 of 1	
Location:	See Figure 3	Date Started:	6/13/2023
Site Address:	4703 Alameda Boulevard NE	Date Completed:	6/13/2023
	Albuquerque, New Mexico 87113	Depth to Groundwater:	N/E
Project Number:	23-412531.1	Field Technician	Enviro-Drill
Drill Rig Type:	CME-75	Partner Engineering and Science	
Sampling Equipment:	Hollow Stem Auger/ SPT	600 Grant Street, Suite 450	
Borehole Diameter:	6"	Denver, Colorado 80203	

Depth, FT	Sample	N-Value	USCS	Description
0				SURFACE COVER: TOPSOIL/Light Vegetation (Thickness Varies)
0.5				
1				
1.5			SC	NATIVE: Brown, moist, loose, Clayey SAND with Silt, fine-grained
2	S	7		
2.5				
3				
3.5				
4				
4.5				
5	S	6		(Moisture: 4.5%)
5.5				
6				
6.5				Boring terminated at 6.5 feet below the ground surface
7				Boring backfilled with soil cuttings upon completion
7.5				Groundwater not encountered (6/13/2023)
8				
8.5				
9				
9.5				
10				
10.5				
11				
11.5				
12				
12.5				
13				
13.5				
14				
14.5				
15				
15.5				
16				
16.5				
17				
17.5				
18				
18.5				
19				
19.5				
20				

Technician: Joey Masters
Date: 6/14/2023
Project and #: MCW Albuquerque, NM 23-412531.1

PERCOLATION FIELD TEST REPORT

Notes & Observations

Perc Test #	I-1	Location:	Weather: Sunny

Time	Comments
9:15	Presoak
9:45	Start Test
11:05	Test Completed

Percolation Test - Pre Soak

Pre Soaking Time -
Time: 9:15 depth 48 inches

Percolation Reading	Start Time/ End Time	Elapsed Time	WL BTP (in)	WL Δ (in)	Infiltration Rate* (in/hr)
1	9:45	10.00	17.5	0.0	0.0
	9:55		17 1/2		
2	9:55	10.00	17 1/2	0.0	0.0
	10:05		17 1/2		
3	10:05	10.00	17 1/2	0.0	0.0
	10:15		17 1/2		
4	10:15	10.00	17 1/2	0.5	0.1
	10:25		18		
5	10:25	10.00	18	0.5	0.1
	10:35		18 1/2		
6	10:35	10.00	18 1/2	0.5	0.1
	10:45		19		
7	10:45	10.00	19	0.3	0.1
	10:55		19.25		
8	10:55	10.00	19.25	0.8	0.2
	11:05		20		

Calculations

d 1 = 30.5
Δd = 0.5
DIA = 6
(Rf) = 2.00

Design Infiltration Rate = $\frac{\text{Pre-adjusted Infiltration Rate}}{\text{Reduction Factor}}$

Pre adjusted Infiltration Rate* = 0.15
Reduction Factor = 2.00

Raw Infiltration Rate (in/hr) = 0.07

Notes:

btp - below top of pipe	*d* 1 = Depth to Initial Water Depth (in.)
WL - water level	Δd = Water Level Drop of the Final Period or Stablixed Rate (in)
min - minutes	DIA - Diameter of the boring (in.)
ft - feet	

*Infiltration Rate percolation rate is the flow volume/ flo change/ change in time

Technician: Joey Masters

Date: 6/14/2023

Project and #: MCW Albuquerque, NM 23-412531.1

Page: 1

PERCOLATION FIELD TEST REPORT

Notes & Observations

Perc Test #	I-2	Location:	Weather: Sunny

Time	Comments
9:00	Presoak
9:30	Start Test
10:50	Test Completed

Percolation Test - Pre Soak

Pre Soaking Time -

Time: 9:00 depth 48 inches

Percolation Reading	Start Time/ End Time	Elapsed Time	WL BTP (in)	WL **Δ** (in)	Infiltration Rate* (in/hr)
1	9:30	10.00	15	0.5	0.1
	9:40		15 1/2		
2	9:40	10.00	15 1/2	0.0	0.0
	9:50		15 1/2		
3	9:50	10.00	15 1/2	0.0	0.0
	10:00		15 1/2		
4	10:00	10.00	15 1/2	0.0	0.0
	10:10		15 1/2		
5	10:10	10.00	15 1/2	0.0	0.0
	10:20		15 1/2		
6	10:20	10.00	15 1/2	0.5	0.1
	10:30		16		
7	10:30	10.00	16	0.3	0.1
	10:40		16.25		
8	10:40	10.00	16.25	0.3	0.1
	10:50		16.5		

Calculations

d 1 = 33.0

Δd = 0.5

DIA = 6

(Rf) = 2.00

Design Infiltration Rate = $\frac{\text{Pre-adjusted Infiltration Rate}}{\text{Reduction Factor}}$

Pre adjusted Infiltration Rate* = 0.09

Reduction Factor = 2.00

Raw Infiltration Rate (in/hr) = 0.04

Notes:	
btp - below top of pipe	*d* 1 = Depth to Initial Water Depth (in.)
WL - water level	Δd = Water Level Drop of the Final Period or Stablixed Rate (in)
min - minutes	DIA - Diameter of the boring (in.)
ft - feet	

*Infiltration Rate percolation rate is the flow volume/ flo change/ change in time

Technician: Joey Masters
Date: 6/14/2023
Project and #: MCW Albuquerque, NM 23-412531.1

Page: 1

PERCOLATION FIELD TEST REPORT

Notes & Observations

Perc Test # I-2	Location:	Weather: Sunny

Time	Comments
9:00	Presoak
9:30	Start Test
10:50	Test Completed

Percolation Test - Pre Soak

Pre Soaking Time -
Time: 9:00 depth 48 inches

Percolation Reading	Start Time/ End Time	Elapsed Time	WL BTP (in)	WL Δ (in)	Infiltration Rate* (in/hr)
1	9:30	10.00	21	0.5	0.2
	9:40		21 1/2		
2	9:40	10.00	21 1/2	0.0	0.0
	9:50		21 1/2		
3	9:50	10.00	21 1/2	0.0	0.0
	10:00		21 1/2		
4	10:00	10.00	21 1/2	0.0	0.0
	10:10		21 1/2		
5	10:10	10.00	21 1/2	0.0	0.0
	10:20		21 1/2		
6	10:20	10.00	21 1/2	0.5	0.2
	10:30		22		
7	10:30	10.00	22	0.5	0.2
	10:40		22.5		
8	10:40	10.00	22.5	0.3	0.1
	10:50		22.75		

Calculations

d 1 = 27.0
Δd = 0.5
DIA = 6
(Rf) = 2.00

Design Infiltration Rate = $\frac{\text{Pre-adjusted Infiltration Rate}}{\text{Reduction Factor}}$

Pre adjusted Infiltration Rate* = 0.14
Reduction Factor = 2.00

Raw Infiltration Rate (in/hr) = 0.07

Notes:

btp - below top of pipe	d 1 = Depth to Initial Water Depth (in.)
WL - water level	Δd = Water Level Drop of the Final Period or Stablixed Rate (in)
min - minutes	DIA - Diameter of the boring (in.)
ft - feet	

*Infiltration Rate percolation rate is the flow volume/ flo change/ change in time

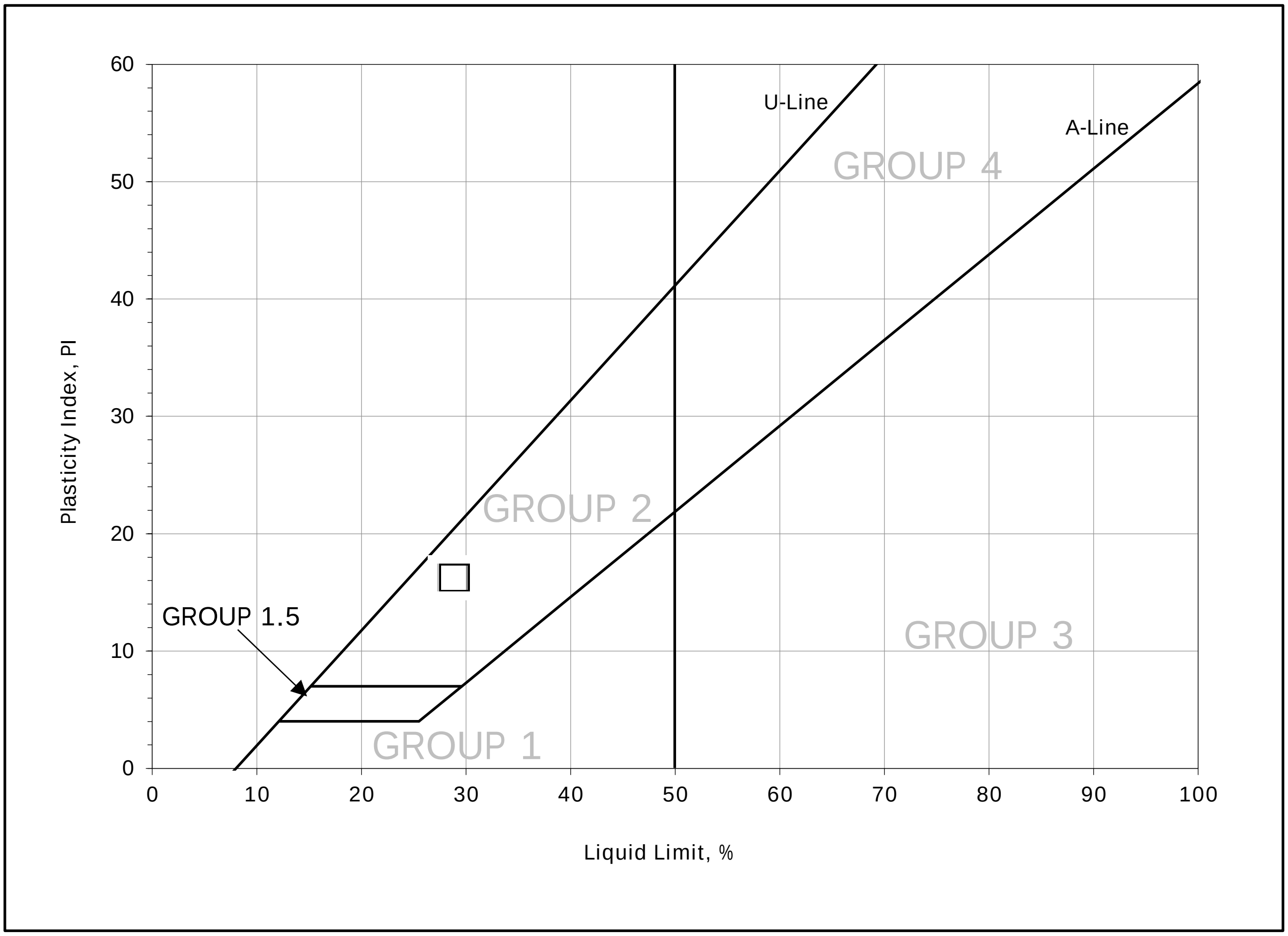
APPENDIX B

Lab Data

PARTNER

Plasticity Index Data

Symbol	Boring	Depth, ft	Natural Moisture Content (%)	Plasticity Index	Plastic Limit	Liquid Limit
□	B-1	5	11.2	16	12	28



Group and USCS Symbols	Soil Descriptions
GROUP 1 – ML, SM, GM, OL*	SILTS, SANDS, AND GRAVELS WITH NO TO MEDIUM PLASTICITY
GROUP 1.5 – ML-CL, SM-SC, GM-GC, OL*	CLAYS, SILTS, SANDS, AND GRAVELS WITH LOW PLASTICITY
GROUP 2 – CL, SC, GC, OL*	CLAYS, SANDS, AND GRAVELS WITH LOW TO MEDIUM PLASTICITY
GROUP 3 – MH, SM, GM, OH*	SILTS, SANDS, AND GRAVELS WITH NO TO HIGH PLASTICITY
GROUP 4 – CH, SC, GC, OH*	CLAYS, SANDS, AND GRAVELS WITH HIGH PLASTICITY
Or combinations of any within the same group (example ML-SM or CL-SC)	

Index Test Data

Boring	Depth, ft	Plasticity Index	Plastic Limit	Liquid Limit	Moisture Content (%)	Percent Passing the No. 200 Sieve
B-1	2	-	-	-	6.7	-
B-1	5	16	12	28	11.2	48.2
B-1	10	-	-	-	2.4	-
B-2	2	-	-	-	4.8	44.9
B-2	7	-	-	-	4.8	26.7
B-2	10	-	-	-	3.4	-
B-3	2	-	-	-	2.3	-
B-3	5	-	-	-	5.1	55.5
B-3	10	-	-	-	2.5	17.2
B-4	2	-	-	-	6.3	42.9
B-4	7	-	-	-	6.9	22.4
B-4	15	-	-	-	3.2	-
B-5	2	-	-	-	3.1	-
B-6	5	-	-	-	4.5	-

APPENDIX C

General Geotechnical Design and Construction Considerations

Subgrade Preparation

Earthwork – Structural Fill/Excavations

Underground Pipeline Installation – Structural Backfill

Cast-in-Place Concrete

Foundations

Laterally Loaded Structures

Excavations and Dewatering

Waterproofing and Drainage

Chemical Treatment of Soils

Paving

Site Grading and Drainage

PARTNER

SUBGRADE PREPARATION

1. In general, construction should proceed per the project specifications and contract documents, as well as governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Subgrade preparation in this section is considered to apply to the initial modifications to existing site conditions to prepare for new planned construction.
3. Prior to the start of subgrade preparation, a detailed conflict study including as-builts, utility locating, and potholing should be conducted. Existing features that are to be demolished should also be identified and the geotechnical study should be referenced to determine the need for subgrade preparation, such as over-excavation, scarification and compaction, moisture conditioning, and/or other activities below planned new structural fills, slabs on grade, pavements, foundations, and other structures.
4. The site conflicts, planned demolitions, and subgrade preparation requirements should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and others.
5. In the event of preparations that will require work near to existing structures to remain in-place, protection of the existing structures should be considered. This also includes a geotechnical review of excavations near to existing structures and utilities and other concerns discussed in General Geotechnical Design and Construction Considerations, EARTHWORK and UNDERGROUND PIPELINE INSTALLATION.
6. Features to be demolished should be completely removed and disposed of per jurisdictional requirements and/or other conditions set forth as a part of the project. Resulting excavations or voids should be backfilled per the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section.
7. Vegetation, roots, soils containing organic materials, debris and/or other deleterious materials on the site should be removed from structural areas and should be disposed of as above. Replacement of such materials should be in accordance with the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section
8. Subgrade preparation required by the geotechnical report may also call for as over-excavation, scarification and compaction, moisture conditioning, and/or other activities below planned structural fills, slabs on grade, pavements, foundations, and other structures. These requirements should be provided within the geotechnical report. The execution of this work should be observed by the geotechnical engineering representative or inspector for the site. Testing of the subgrade preparation should be performed per the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section.

9. Subgrade Preparation cannot be completed on frozen ground or on ground that is not at a proper moisture condition. Wet subgrades may be dried under favorable weather if they are disked and/or actively worked during hot, dry, weather, when exposed to wind and sunlight. Frozen ground or wet material can be removed and replaced with suitable material. Dry material can be pre-soaked, or can have water added and worked in with appropriate equipment. The soil conditions should be monitored by the geotechnical engineer prior to compaction. Following this type of work, approved subgrades should be protected by direction of surface water, covering, or other methods, otherwise, re-work may be needed.

EARTHWORK – STRUCTURAL FILL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Earthwork in this section is considered to apply to the re-shaping and grading of soil, rock, and aggregate materials for the purpose of supporting man-made structures. Where earthwork is needed to raise the elevation of the site for the purpose of supporting structures or forming slopes, this is referred to as the placement of structural fill. Where lowering of site elevations is needed prior to the installation of new structures, this is referred to as earthwork excavations.
3. Prior to the start of earthwork operations, the geotechnical study should be referenced to determine the need for subgrade preparation, such as over-excavation or scarification and compaction of unsuitable soils below planned structural fills, slabs on grade, pavements, foundations, and other structures. These required preparations should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and others. The preparations should be observed by the inspector or geotechnical engineer representative, and following such subgrade preparation, the geotechnical engineer should observe the prepared subgrade to approve it for the placement of earthwork fills or new structures.
4. Structural fill materials should be relatively free of organic materials, man-made debris, environmentally hazardous materials, and brittle, non-durable aggregate, frozen soil, soil clods or rocks and/or any other materials that can break down and degrade over time.
5. In deeper structural fill zones, expansive soils (greater than 1.5 percent swell at 100 pounds per square foot surcharge) and rock fills (fills containing particles larger than 4 inches and/or containing more than 35 percent gravel larger than ¾-inch diameter or more than 50 percent gravel) may be used with the approval and guidance of the geotechnical report or geotechnical engineer. This may require the placement of geotextiles or other added costs and/or conditions. These conditions may also apply to corrosive soils (less than 2,000 ohm-cm resistivity, more than 50 ppm chloride content, more than 0.1 percent sulfates)
6. For structural fill zones that are closer in depth below planed structures, low expansive materials, and materials with smaller particle size are generally recommended, as directed by the geotechnical report (see criteria above in 5). This may also apply to corrosive soils.
7. For structural fill materials, in general the compaction equipment should be appropriate for the thickness of the loose lift being placed, and the thickness of the loose lift being placed should be at least two times the maximum particle size incorporated in the fill.
8. Fill lift thickness (including bedding) should generally be proportioned to achieve 95 percent or more of a standard proctor (ASTM D689) maximum dry density (MDD) or 90 percent or more of a

modified proctor (ASTM D1557) MDD, depending on the state practices. For subgrades below roadways, the general requirement for soil compaction is usually increased to 100 percent or more of the standard proctor MDD and 95 percent or more of the modified proctor MDD.

9. Soil compaction should be performed at a moisture content generally near optimum moisture content determined by either standard or modified proctor, and ideally within 3 percent below to 1 percent over the optimum for a standard proctor, and from 2 percent below to 2 percent above optimum for a modified proctor.
10. In some instances fill areas are difficult to access. In such cases a low-strength soil-cement slurry can be used in the place of compacted fill soil. In general such fills should be rated to have a 28-day strength of 75 to 125 psi, which in some areas is referred to as a “1-sack” slurry. It should be noted that these materials are wet during placement, and require a period of 2 days (24 hours) to cure before additional fill can be placed above them. Testing of this material can be done using concrete cylinder compression strength testing equipment, but care is needed in removing the test specimens from the molds. Field testing using the ball method, and spread or flow testing is also acceptable.
11. For fills to be placed on slopes, benching of fill lifts is recommended, which may require cutting into existing slopes to create a bench perpendicular to the slope where soil can be placed in a relatively horizontal orientation. For the construction of slopes, the slopes should be over-built and cut back to grade, as the material in the outer portion of the slope may not be well compacted.
12. For subgrade below roadways, runways, railways or other areas to receive dynamic loading, a proofroll of the finished, compacted subgrade should be performed by the geotechnical engineer or inspector prior to the placement of structural aggregate, asphalt or concrete. Proofrolling consists of observing the performance of the subgrade under heavy-loaded equipment, such as full, 4,000 Gallon water truck, loaded tandem-axel dump truck or similar. Areas that exhibit instability during proofroll should be marked for additional work prior to approval of the subgrade for the next stage of construction.
13. Quality control testing should be provided on earthwork. Proctor testing should be performed on each soil type, and one-point field proctors should be used to verify the soil types during compaction testing. If compaction testing is performed with a nuclear density gauge, it should be periodically correlated with a sand cone test for each soil type. Density testing should be performed per project specifications and or jurisdictional requirements, but not less than once per 12 inches elevation of any fill area, with additional tests per 12-inch fill area for each additional 7,500 square-foot section or portion thereof.
14. For earthwork excavations, OSHA guidelines should be referenced for sloping and shoring. Excavations over a depth of 20 feet require a shoring design. In the event excavations are planned near to existing structures, the geotechnical engineer should be consulted to evaluate whether such excavation will call for shoring or underpinning the adjacent structure. Pre-construction and post-construction condition surveys and vibration monitoring might also be helpful to evaluate any potential damage to surrounding structures.

15. Excavations into rock, partially weathered rock, cemented soils, boulders and cobbles, and other hard soil or "hard-pan" materials, may result in slower excavation rates, larger equipment with specialized digging tools, and even blasting. It is also not unusual in these situations for screening and or crushing of rock to be called for. Blasting, hard excavating, and material processing equipment have special safety concerns and are more costly than the use of soil excavation equipment. Additionally, this type of excavation, especially blasting, is known to cause vibrations that should be monitored at nearby structures. As above, a pre-blast and post-blast conditions assessment might also be warranted.

UNDERGROUND PIPELINE – STRUCTURAL BACKFILL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, the State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County Public Works, Occupational Safety and Health Administration (OSHA), Private Utility Companies, and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered, and in some cases work may take place to multiple different standards. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Underground pipeline in this section is considered to apply to the installation of underground conduits for water, storm water, irrigation water, sewage, electricity, telecommunications, gas, etc. Structural backfill refers to the activity of restoring the grade or establishing a new grade in the area where excavations were needed for the underground pipeline installation.
3. Prior to the start of underground pipeline installation, a detailed conflict study including as-builts, utility locating, and potholing should be conducted. The geotechnical study should be referenced to determine subsurface conditions such as caving soils, unsuitable soils, shallow groundwater, shallow rock and others. In addition, the utility company responsible for the line also will have requirements for pipe bedding and support as well as other special requirements. Also, if the underground pipeline traverses other properties, rights-of-way, and/or easements etc. (for roads, waterways, dams, railways, other utility corridors, etc.) those owners may have additional requirements for construction.
4. The required preparations above should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and other stake holders.
5. For pipeline excavations, OSHA guidelines should be referenced for sloping and shoring. Excavations over a depth of 20 feet require a shoring design. In the event excavations are planned near to existing structures or pipelines, the geotechnical engineer should be consulted to evaluate whether such excavation will call for shoring or supporting the adjacent structure or pipeline. A pre-construction and post-construction condition survey and vibration monitoring might also be helpful to evaluate any potential damage to surrounding structures.
6. Excavations into rock, partially weathered rock, cemented soils, boulders and cobbles, and other hard soil or "hard-pan" materials, may result in slower excavation rates, larger equipment with specialized digging tools, and even blasting. It is also not unusual in these situations for screening and or crushing of rock to be called for. Blasting, hard excavating and material processing equipment have special safety concerns and are more costly than the use soil excavation equipment. Additionally, this type of excavation, especially blasting, is known to cause vibrations that should be monitored at nearby structures. As above, a pre-blast and post-blast conditions assessment might also be warranted.

7. Bedding material requirements vary between utility companies and might depend of the type of pipe material and availability of different types of aggregates in different locations. In general, bedding refers to the material that supports the bottom of the pipe, and extends to 1 foot above the top of the pipe. In general the use of aggregate base for larger diameter pipes (6-inch diameter or more) is recommended lacking a jurisdictionally specified bedding material. Gas lines and smaller diameter lines are often backfilled with fine aggregate meeting the ASTM requirements for concrete sand. In all cases bedding with less than 2,000 ohm-cm resistivity, more than 50 ppm chloride content or more than 0.1 percent sulfates should not be used.
8. Structural backfill materials above the bedding should be relatively free of organic materials, man-made debris, environmentally hazardous materials, frozen material, and brittle, non-durable aggregate, soil clods or rocks and/or any other materials that can break down and degrade over time.
9. In general the backfill soil requirements will depend on the future use of the land above the buried line, but in most cases, excessive settlement of the pipe trench is not considered advisable or acceptable. As such, the structural backfill compaction equipment should be appropriate for the thickness of the loose lift being placed. The thickness of the loose lift being placed should be at least two times the maximum particle size incorporated in the fill. Care should be taken not to damage the pipe during compaction or compaction testing.
10. Fill lift thickness (including bedding) should generally be proportioned to achieve 95 percent or more of a standard proctor (ASTM D689) maximum dry density (MDD) or 90 percent or more of a modified proctor (ASTM D1557) MDD, depending on the state practices (in general the modified proctor is required in California and for projects in the jurisdiction of the Army Corps of Engineers). For backfills within the upper poritons of roadway subgrades, the general requirement for soil compaction is usually increased to 100 percent or more of the standard proctor MDD and 95 percent or more of the modified proctor MDD.
11. Soil compaction should be performed at a moisture content generally near optimum moisture content determined by either standard or modified proctor, and ideally within 3 percent below to 1 percent over the optimum for a standard proctor, and from 2 percent below to 2 percent above optimum for a modified proctor.
12. In some instances fill areas are difficult to access. In such cases a low-strength soil-cement slurry can be used in the place of compacted fill soil. In general such fills should be rated to have a 28-day strength of 75 to 125 psi, which in some areas is referred to as a "1-sack" slurry. It should be noted that these materials are wet, and require a period of 2 days (24 hours) to cure before additional fill can be placed above it. Testing of this material can be done using concrete cylinder compression strength testing equipment, but care is needed in removing the test specimens from the molds. Field testing using the ball method, and spread or flow testing is also acceptable.
13. Quality control testing should be provided on structural backfill to assist the contractor in meeting project specifications. Proctor testing should be performed on each soil type, and one-point field proctors should be used to verify the soil types during compaction testing. If compaction testing is

performed with a nuclear density gauge, it should be periodically correlated with a sand cone test for each soil type.

14. Density testing should be performed on structural backfill per project specifications and or jurisdictional requirements, but not less than once per 12 inches elevation in each area, and additional tests for each additional 500 linear-foot section or portion thereof.

CAST-IN-PLACE CONCRETE

SLABS-ON-GRADE/STRUCTURES/PAVEMENTS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Cast-in-place concrete (concrete) in this section is considered to apply to the installation of cast-in-place concrete slabs on grade, including reinforced and non-reinforced slabs, structures, and pavements.
3. In areas where concrete is bearing on prepared subgrade or structural fill soils, testing and approval of this work should be completed prior to the beginning of concrete construction.
4. In locations where a concrete is approved to bear on in-place (native) soil or in locations where approved documented fills have been exposed to weather conditions after approval, a concrete subgrade evaluation should be performed prior to the placement of reinforcing steel and or concrete. This can consist of probing with a “t”-handled rod, borings, penetrometer testing, dynamic cone penetration testing and/or other methods requested by the geotechnical engineer and/or inspector. Where unsuitable, wet, or frozen bearing material is encountered, the geotechnical engineer should be consulted for additional recommendations.
5. Slabs on grade should be placed on a 4-inch thick or more capillary barrier consisting of non-corrosive (more than 2,000 ohm-cm resistivity, less than 50 ppm chloride content and less than 0.1 percent sulfates) aggregate base or open-graded aggregate material. This material should be compacted or consolidated per the recommendations of the structural engineer or otherwise would be covered by the General Considerations for <u>EARTHWORK</u>.
6. Depending on the site conditions and climate, vapor barriers may be required below in-door grade-slabs to receive flooring. This reduces the opportunity for moisture vapor to accumulate in the slab, which could degrade flooring adhesive and result in mold or other problems. Vapor barriers should be specified by the structural engineer and/or architect. The installation of the barrier should be inspected to evaluate the correct product and thickness is used, and that it has not been damaged or degraded.
7. At times when rainfall is predicted during construction, a mud-mat or a thin concrete layer can be placed on prepared and approved subgrades prior to the placement of reinforcing steel or tendons. This serves the purpose of protecting the subgrades from damage once the reinforcement placement has begun.
8. Prior to the placement of concrete, exposed subgrade or base material and forms should be wetted, and form release compounds should be applied. Reinforcement support stands or ties should be

checked. Concrete bases or subgrades should not be so wet that they are softened or have standing water.

9. For a cast-in-place concrete, the form dimensions, reinforcement placement and cover, concrete mix design, and other code requirements should be carefully checked by an inspector before and during placement. The reinforcement should be specified by the structural engineering drawings and calculations.
10. For post-tension concrete, an additional check of the tendons is needed, and a tensioning inspection form should be prepared prior to placement of concrete.
11. For Portland cement pavements, forms an additional check of reinforcing dowels should performed per the design drawings.
12. During placement, concrete should be tested, and should meet the ACI and jurisdictional requirements and mix design targets for slump, air entrainment, unit weight, compressive strength, flexural strength (pavements), and any other specified properties. In general concrete should be placed within 90 minutes of batching at a temperature of less than 90 degrees Fahrenheit. Adding of water to the truck on the jobsite is generally not encouraged.
13. Concrete mix designs should be created by the accredited and jurisdictionally approved supplier to meet the requirements of the structural engineer. In general a water/cement ratio of 0.45 or less is advisable, and aggregates, cement, flyash, and other constituents should be tested to meet ASTM C-33 standards, including Alkali Silica Reaction (ASR). To further mitigate the possibility of concrete degradation from corrosion and ASR, Type II or V Portland Cement should be used, and fly ash replacement of 25 percent is also recommended. Air entrained concrete should be used in areas where concrete will be exposed to frozen ground or ambient temperatures below freezing.
14. Control joints are recommended to improve the aesthetics of the finished concrete by allowing for cracking within partially cut or grooved joints. The control joints are generally made to depths of about 1/4 of the slab thickness and are generally completed within the first day of construction. The spacing should be laid out by the structural engineer, and is often in a square pattern. Joint spacing is generally 5 to 15 feet on-center but this can vary and should be decided by the structural engineer. For pavements, construction joints are generally considered to function as control joints. Post-tensioned slabs generally do not have control joints.
15. Some slabs are expected to meet flatness and levelness requirements. In those cases, testing for flatness and levelness should be completed as soon as possible, usually the same day as concrete placement, and before cutting of control joints if possible. Roadway smoothness can also be measured, and is usually specified by the jurisdictional owner if is required.
16. Prior to tensioning of post-tension structures, placement of soil backfills or continuation of building on newly-placed concrete, a strength requirement is generally required, which should be specified by the structural engineer. The strength progress can be evaluated by the use of concrete compressive strength cylinders or maturity monitoring in some jurisdictions. Advancing with backfill, additional concrete work or post-tensioning without reaching strength benchmarks could result in damage and failure of the concrete, which could result in danger and harm to nearby people and property.

17. In general, concrete should not be exposed to freezing temperatures in the first 7 days after placement, which may require insulation or heating. Additionally, in hot or dry, windy weather, misting, covering with wet burlap or the use of curing compounds may be called for to reduce shrinkage cracking and curling during the first 7 days.

FOUNDATIONS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Foundations in this section are considered to apply to the construction of structural supports which directly transfer loads from man-made structures into the earth. In general, these include shallow foundations and deep foundations. Shallow foundations are generally constructed for the purpose of distributing the structural loads horizontally over a larger area of earth. Some types of shallow foundations (or footings) are spread footings, continuous footings, mat foundations, and reinforced slabs-on-grade. Deep foundations are generally designed for the purpose of distributing the structural loads vertically deeper into the soil by the use of end bearing and side friction. Some types of deep foundations are driven piles, auger-cast piles, drilled shafts, caissons, helical piers, and micro-piles.
3. For shallow foundations, the minimum bearing depth considered should be greater than the maximum design frost depth for the location of construction. This can be found on frost depth maps (ICC), but the standard of practice in the city and/or county should also be consulted. In general the bearing depth should never be less than 18 inches below planned finished grades.
4. Shallow continuous foundations should be sized with a minimum width of 18 inches and isolated spread footings should be a minimum of 24 inches in each direction. Foundation sizing, spacing, and reinforcing steel design should be performed by a qualified structural engineer.
5. The geotechnical engineer will provide an estimated bearing capacity and settlement values for the project based on soil conditions and estimated loads provided by the structural engineer. It is assumed that appropriate safety factors will be applied by the structural engineer.
6. In areas where shallow foundations are bearing on prepared subgrade or structural fill soils, testing and approval of this work should be completed prior to the beginning of foundation construction.
7. In locations where the shallow foundations are approved to bear on in-place (native) soil or in locations where approved documented fills have been exposed to weather conditions after approval, a foundation subgrade evaluation should be performed prior to the placement of reinforcing steel. This can consist of probing with a “t”-handled rod, borings, penetrometer testing, dynamic cone penetration testing and/or other methods requested by the geotechnical engineer and/or inspector. Where unsuitable foundation bearing material is encountered, the geotechnical engineer should be consulted for additional recommendations.
8. For shallow foundations to bear on rock, partially weathered rock, hard cemented soils, and/or boulders, the entire foundation system should bear directly on such material. In this case, the rock surface should be prepared so that it is clean, competent, and formed into a roughly horizontal,

stepped base. If that is not possible, then the entire structure should be underlain by a zone of structural fill. This may require the over-excavation in areas of rock removal and/or hard dig. In general this zone can vary in thickness but it should be a minimum of 1 foot thick. The geotechnical engineer should be consulted in this instance.

9. At times when rainfall is predicted during construction, a mud-mat or a thin concrete layer can be placed on prepared and approved subgrades prior to the placement of reinforcing steel. This serves the purpose of protecting the subgrades from damage once the reinforcing steel placement has begun.
10. For cast-in-place concrete foundations, the excavations dimensions, reinforcing steel placement and cover, structural fill compaction, concrete mix design, and other code requirements should be carefully checked by an inspector before and during placement.

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11. For deep foundations, the geotechnical engineer will generally provide design charts that provide foundations axial capacity and uplift resistance at various depths given certain-sized foundations. These charts may be based on blow count data from drilling and or laboratory testing. In general safety factors are included in these design charts by the geotechnical engineer.
12. In addition, the geotechnical engineer may provide other soil parameters for use in the lateral resistance analysis. These parameters are usually raw data, and safety factors should be provided by the shaft designer. Sometimes, direct shear and or tri-axial testing is performed for this analysis.
13. In general the spacing of deep foundations is expected to be 6 shaft diameters or more. If that spacing is reduced, a group reduction factor should be applied by the structural engineer to the foundation capacities per FHWA guidelines. The spacing should not be less than 2.5 shaft diameters.
14. For deep foundations, a representative of the geotechnical engineer should be on-site to observe the excavations (if any) to evaluate that the soil conditions are consistent with the findings of the geotechnical report. Soil/rock stratigraphy will vary at times, and this may result in a change in the planned construction. This may require the use of fall protection equipment to perform observations close to an open excavation.
15. For driven foundations, a representative of the geotechnical engineer should be on-site to observe the driving process and to evaluate that the resistance of driving is consistent with the design assumptions. Soil/rock stratigraphy will vary at times and may this may result in a change in the planned construction.
16. For deep foundations, the size, depth, and ground conditions should be verified during construction by the geotechnical engineer and/or inspector responsible. Open excavations should be clean, with any areas of caving and groundwater seepage noted. In areas below the groundwater table, or areas where slurry is used to keep the trench open, non-destructive testing techniques should be used as outlined below.
17. Steel members including structural steel piles, reinforcing steel, bolts, threaded steel rods, etc. should be evaluated for design and code compliance prior to pick-up and placement in the foundation. This includes verification of size, weight, layout, cleanliness, lap-splices, etc. In addition, if non-destructive testing such as crosshole sonic logging or gamma-gamma logging is required,

access tubes should be attached to the steel reinforcement prior to placement, and should be relatively straight, capped at the bottom, and generally kept in-round. These tubes must be filled with water prior to the placement of concrete.

18. In cases where steel welding is required, this should be observed by a certified welding inspector.
19. In many cases, a crane will be used to lower steel members into the deep foundations. Crane picks should be carefully planned, including the ground conditions at placement of outriggers, wind conditions, and other factors. These are not generally provided in the geotechnical report, but can usually be provided upon request.
20. Cast-in-place concrete, grout or other cementations materials should be pumped or distributed to the bottom of the excavation using a tremmie pipe or hollow stem auger pipe. Depending on the construction type, different mix slumps will be used. This should be carefully checked in the field during placement, and consolidation of the material should be considered. Use of a vibrator may be called for.
21. For work in a wet excavation (slurry), the concrete placed at the bottom of the excavation will displace the slurry as it comes up. The upper layer of concrete that has interacted with the slurry should be removed and not be a part of the final product.
22. Bolts or other connections to be set in the top after the placement is complete should be done immediately after final concrete placement, and prior to the on-set of curing.
23. For shafts requiring crosshole sonic logging or gamma-gamma testing, this should be performed within the first week after placement, but not before a 2 day curing period. The testing company and equipment manufacturer should provide more details on the requirements of the testing.
24. Load testing of deep foundations is recommended, and it is often a project requirement. In some cases, if test piles are constructed and tested, it can result in a significant reduction of the amount of needed foundations. The load testing frame and equipment should be sized appropriately for the test to be performed, and should be observed by the geotechnical engineer or inspector as it is performed. The results are provided to the structural engineer for approval.

LATERALLY LOADED STRUCTURES - RETAINING WALLS/SLOPES/DEEP FOUNDATIONS/MISCELLANEOUS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Laterally loaded structures for this section are generally meant to describe structures that are subjected to loading roughly horizontal to the ground surface. Such structures include retaining walls, slopes, deep foundations, tall buildings, box culverts, and other buried or partially buried structures.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. Laterally loaded structures are generally affected by overburden pressure, water pressure, surcharges, and other static loads, as well as traffic, seismic, wind, and other dynamic loads. The structural engineer must account for these loads. In addition, eccentric loading of the foundation should be evaluated and accounted for by the structural engineer. The structural engineer is also responsible for applying the appropriate factors of safety to the raw data provided by the geotechnical engineer.
5. The geotechnical report should provide data regarding soil lateral earth pressures, seismic design parameters, and groundwater levels. In the report the pressures are usually reported as raw data in the form of equivalent fluid pressures for three cases. 1. Static is for soil pressure against a structure that is fixed at top and bottom, like a basement wall or box culvert. 2. Active is for soil pressure against a wall that is free to move at the top, like a retaining wall. 3. Passive is for soil that is resisting the movement of the structure, usually at the toe of the wall where the foundation and embedded section are located. The structural engineer is responsible for deciding on safety factors for design parameters and groundwater elevations based on the raw data in the geotechnical report.
6. Generally speaking, direct shear or tri-axial shear testing should be performed for this evaluation in cases of soil slopes or unrestrained soil retaining walls over 6 feet in height or in lower walls in some cases based on the engineer's judgment. For deep foundations and completely buried structures, this testing will be required per the discretion of the structural engineer.
7. For non-confined retaining walls (walls that are not attached at the top) and slopes, a geotechnical engineer should perform overall stability analysis for sliding, overturning, and global stability. For walls that are structurally restrained at the top, the geotechnical engineer does not generally perform this analysis. Internal wall stability should be designed by the structural engineer.

8. Cut slopes into rock should be evaluated by an engineering geologist, and rock coring to identify the orientation of fracture plans, faults, bedding planes, and other features should be performed. An analysis of this data will be provided by the engineering geologist to identify modes of failure including sliding, wedge, and overturning, and to provide design and construction recommendations.
9. For laterally loaded deep foundations that support towers, bridges or other structures with high lateral loads, geotechnical reports generally provide parameters for design analysis which is performed by the structural engineer. The structural engineer is responsible for applying appropriate safety factors to the raw data from the geotechnical engineer.
10. Construction recommendations for deep foundations can be found in the General Geotechnical Design and Construction Considerations-FOUNDATIONS section.
11. Construction of retaining walls often requires temporary slope excavations and shoring, including soil nails, soldier piles and lagging or laid-back slopes. This should be done per OSHA requirements and may require specialty design and contracting.
12. In general, surface water should not be directed over a slope or retaining wall, but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.
13. Waterproofing for retaining walls is generally required on the backfilled side, and they should be backfilled with an 18-inch zone of open graded aggregate wrapped in filter fabric or a synthetic draining product, which outlets to weep holes or a drain at the base of the wall. The purpose of this zone, which is immediately behind the wall is to relieve water pressures from building behind the wall.
14. Backfill compaction around retaining walls and slopes requires special care. Lighter equipment should be considered, and consideration to curing of cementitious materials used during construction will be called for. Additionally, if mechanically stabilized earth walls are being constructed, or if tie-backs are being utilized, additional care will be necessary to avoid damaging or displacing the materials. Use of heavy or large equipment, and/or beginning of backfill prior to concrete strength verification can create dangers to construction and human safety. Please refer to the General Geotechnical Design and Construction Considerations-CAST-IN-PLACE CONCRETE section. These concerns will also apply to the curing of cell grouting within reinforced masonry walls.
15. Usually safety features such as handrails are designed to be installed at the top of retaining walls and slopes. Prior to their installation, workers in those areas will need to be equipped with appropriate fall protection equipment.

EXCAVATION AND DEWATERING

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Excavation and Dewatering for this section are generally meant to describe structures that are intended to create stable, excavations for the construction of infrastructure near to existing development and below the groundwater table.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for LATERALLY LOADED STRUCTURES, FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. The site excavations will generally be affected by overburden pressure, water pressure, surcharges, and other static loads, as well as traffic, seismic, wind, and other dynamic loads. The structural engineer must account for these loads as described in Section 5.2 of this report. In addition, eccentric loading of the foundation should be evaluated and accounted for by the structural engineer. The structural engineer is also responsible for applying the appropriate factors of safety to the raw data provided by the geotechnical engineer.
5. The geotechnical report should provide data regarding soil lateral earth pressures, seismic design parameters, and groundwater levels. In the report the pressures are usually reported as raw data in the form of equivalent fluid pressures for three cases. 1. Static is for soil pressure against a structure that is fixed at top and bottom, like a basement wall or box culvert. 2. Active is for soil pressure against a wall that is free to move at the top, like a retaining wall. 3. Passive is for soil that is resisting the movement of the structure, usually at the toe of the wall where the foundation and embedded section are located. The structural engineer is responsible for deciding on safety factors for design parameters and groundwater elevations based on the raw data in the geotechnical report.
6. The parameters provided above are based on laboratory testing and engineering judgement. Since numerous soil layers with different properties will be encountered in a large excavation, assumptions and judgement are used to generate the equivalent fluid pressures to be used in design. Factors of safety are not included in those numbers and should be evaluated prior to design.
7. Groundwater, if encountered will dramatically change the stability of the excavation. In addition, pumping of groundwater from the bottom of the excavation can be difficult and costly, and it can result in potential damage to nearby structures if groundwater drawdown occurs. As such, we recommend that groundwater monitoring be performed across the site during design and prior to construction to assist in the excavation design and planning.

8. Groundwater pumping tests should be performed if groundwater pumping will be needed during construction. The pumping tests can be used to estimate drawdown at nearby properties, and also will be needed to determine the hydraulic conductivity of the soil for the design of the dewatering system.
9. For excavation stabilization in granular and dense soil, the use of soldier piles and lagging is recommended. The soldier pile spacing and size should be determined by the structural engineer based on the lateral loads provided in the report. In general, the spacing should be more than two pile diameters, and less than 8 feet. Soldier piles should be advanced 5 feet or more below the base of the excavation. Passive pressures from Section 5.2 can be used in the design of soldier piles for the portions of the piles below the excavation.
10. If the piles are drilled, they should be grouted in-place. If below the groundwater table, the grouting should be accomplished by tremmie pipe, and the concrete should be a mix intended for placement below the groundwater table. For work in a wet excavation, the concrete placed at the bottom of the excavation will displace the water as it comes up. The upper layer of concrete that has interacted with the water should be removed and not be a part of the final product. Lagging should be specially designed timber or other lagging. The temporary excavation will need to account for seepage pressures at the toe of the wall as well as hydrostatic forces behind the wall.
11. Depending on the loading, tie back anchors and/or soil nails may be needed. These should be installed beyond the failure envelope of the wall. This would be a plane that is rotated upward 55 degrees from horizontal. The strength of the anchors behind this plane should be considered, and bond strength inside the plane should be ignored. If friction anchors are used, they should extend 10 feet or more beyond the failure envelope. Evaluation of the anchor length and encroachment onto other properties, and possible conflicts with underground utilities should be carefully considered. Anchors are typically installed 25 to 40 degrees below horizontal. The capacity of the anchors should be checked on 10% of locations by loading to 200% of the design strength. All should be loaded to 120% of design strength, and should be locked off at 80%
12. The shoring and tie backs should be designed to allow less than ½ inch of deflection at the top of the excavation wall, where the wall is within an imaginary 1:1 line extending downward from the base of surrounding structures. This can be expanded to 1 inch of deflection if there is no nearby structure inside that plane. An analysis of nearby structures to locate their depth and horizontal position should be conducted prior to shored excavation design.
13. Assuming that the excavations will encroach below the groundwater table, allowances for drainage behind and through the lagging should be made. The drainage can be accomplished by using an open-graded gravel material that is wrapped in geotextile fabric. The lagging should allow for the collected water to pass through the wall at select locations into drainage trenches below the excavation base. These trenches should be considered as sump areas where groundwater can be pumped out of the excavation.
14. The pumped groundwater needs to be handled properly per jurisdictional guidelines.

15. In general, surface water should not be directed over a slope or retaining wall, but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.
16. Safety features such as handrails or barriers are to be designed to be installed at the top of retaining walls and slopes. Prior to their installation, workers in those areas will need to be equipped with appropriate fall protection equipment.

Waterproofing and Back Drainage

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Waterproofing and Back drainage structures for this section are generally meant to describe permanent subgrade structures that are planned to be below the historic high groundwater elevation.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. In general, surface water should not be directed over a slope or retaining wall, but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.
5. Waterproofing for retaining walls is generally required on the backfilled side, and they should be backfilled with an 18-inch zone of open graded aggregate wrapped in filter fabric or a synthetic draining product, which outlets to weep holes or a drain at the base of the wall. The purpose of this zone, which is immediately behind the wall is to relieve water pressures from building behind the wall.
6. If basement walls below groundwater table are planned on this site, sump pumps will be needed to reduce the build-up of water in the basement. The design should be the historic high groundwater. The pumping system should be designed to keep the slab and walls relatively dry so that mold, efflorescence, and other detrimental effects to the concrete structure will not result.
7. Backfill compaction around retaining walls and slopes requires special care. Lighter equipment should be considered, and consideration to curing of cementitious materials used during construction will be called for. Additionally, if mechanically stabilized earth walls are being constructed, or if tie-backs are being utilized, additional care will be necessary to avoid damaging or displacing the materials. Use of heavy or large equipment, and/or beginning of backfill prior to concrete strength verification can create dangers to construction and human safety. Please refer to the General Geotechnical Design and Construction Considerations-CAST-IN-PLACE CONCRETE section. These concerns will also apply to the curing of cell grouting within reinforced masonry walls.

CHEMICAL TREATMENT OF SOIL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Chemical treatment of soil for this section is generally meant to describe the process of improving soil properties for a specific purpose, using cement or chemical lime.
3. A mix design should be performed by the geotechnical engineer to help it meet the specific strength, plasticity index, durability, and/or other desired properties. The mix design should be performed using the proposed chemical lime or cement proposed for use by the contractor, along with samples of the site soil that are taken from the material to be used in the process.
4. For the mix design the geotechnical engineer should perform proctor testing to determine optimum moisture content of the soil, and then mix samples of the soil at 3 percent above optimum moisture content with varying concentrations of lime or cement. The samples will be prepared and cured per ASTM standards, and then after 7-days for curing, they will be tested for compression strength. Durability testing goes on for 28 days.
5. Following this testing, the geotechnical engineer will provide a recommended mix ratio of cement or chemical lime in the geotechnical report for use by the contractor. The geotechnical engineer will generally specify a design ratio of 2 percent more than the minimum to account for some error during construction.
6. Prior to treatment, the in-place soil moisture should be measured so that the correct amount of water can be used during construction. Work should not be performed on frozen ground.
7. During construction, special considerations for construction of treated soils should be followed. The application process should be conducted to prevent the loss of the treatment material to wind which might transport the materials off site, and workers should be provided with personal protective equipment for dust generated in the process.
8. The treatment should be applied evenly over the surface, and this can be monitored by use of a pan placed on the subgrade. This can also be tested by preparing test specimens from the in-place mixture for laboratory testing.
9. Often, after or during the chemical application, additional water may be needed to activate the chemical reaction. In general, it should be maintained at about 3 percent or more above optimum moisture. Following this, mixing of the applied material is generally performed using specialized equipment.

10. The total amount of chemical provided can be verified by collecting batch tickets from the delivery trucks, and the depth of the treatment can be verified by digging of test pits, and the use of reagents that react with lime and or cement.
11. For the use of lime treatment, compaction should be performed after a specified amount of time has passed following mixing and re-grading. For concrete, compaction should be performed immediately after mixing and re-grading. In both cases, some swelling of the surface should be expected. Final grading should be performed the following day of the initial work for lime treatment, and within 2 to 4 hours for soil cement.
12. Quality control testing of compacted treated subgrades should be performed per the recommendations of the geotechnical report, and generally in accordance with General Geotechnical Design and Construction Considerations - EARTHWORK

PAVING

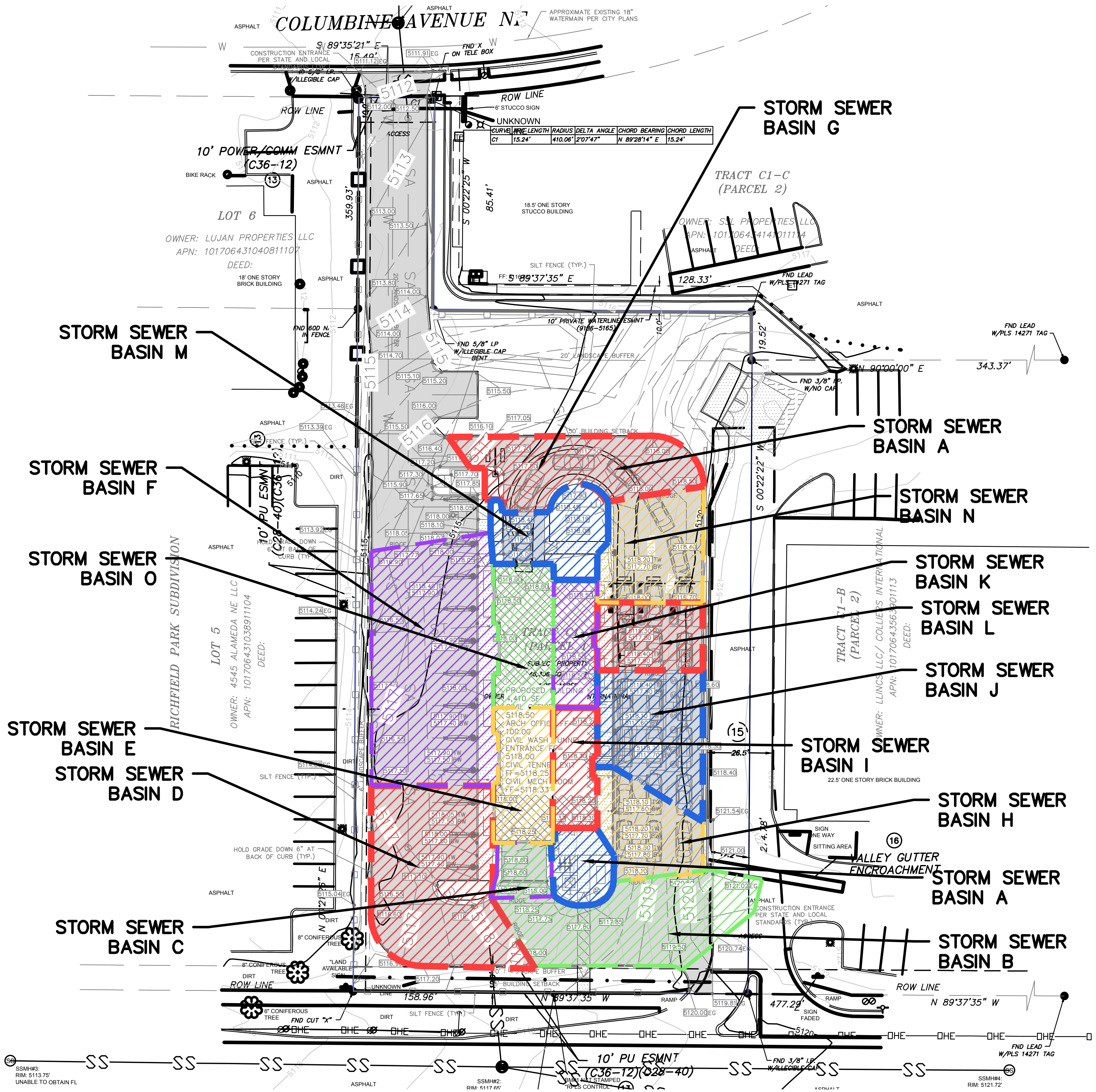
1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Paving for this section is generally meant to describe the placement of surface treatments on travel-ways to be used by rubber-tired vehicles, such as roadways, runways, parking lots, etc.
3. The geotechnical engineer is generally responsible for providing structural analysis to recommend the thickness of pavement sections, which can include asphalt, concrete pavements, aggregate base, cement or lime treated aggregate base, and cement or lime treated subgrades.
4. The civil engineer is generally responsible for determining which surface finishes and mixes are appropriate, and often the owner, general contractor and/or other party will decide on lift thickness, the use of tack coats and surface treatments, etc.
5. The geotechnical engineer will generally be provided with the planned traffic loading, as well as reliability, design life, and serviceability factors by the jurisdiction, traffic engineer, designer, and/or owner. The geotechnical study will provide data regarding soil resiliency and strength. A pavement modeling software is generally used to perform the analysis for design, however, jurisdictional minimum sections also must be considered, as well as construction considerations and other factors.
6. The geotechnical report report will generally provide pavement section thicknesses if requested.
7. For construction of overlays, where new pavement is being placed on old pavement, an evaluation of the existing pavement is needed, which should include coring the pavement, evaluation of the overall condition and thickness of the pavement, and evaluation of the pavement base and subgrade materials.
8. In general, the existing pavement is milled and treated with a tack coat prior to the placement of new pavement for the purpose of creating a stronger bond between the old and new material. This is also a way of removing aged asphalt and helping to maintain finished grades closer to existing conditions grading and drainage considerations.
9. If milling is performed, a minimum of 2 inches of existing asphalt should be left in-place to reduce the likelihood of equipment breaking through the asphalt layer and destroying its integrity. After milling and before the placement of tack coat, the surface should be evaluated for cracking or degradation. Cracked or degraded asphalt should be removed, spanned with geosynthetic reinforcement, or be otherwise repaired per the direction of the civil and or geotechnical engineer prior to continuing construction. Proofrolling may be requested.

10. For pavements to be placed on subgrade or base materials, the subgrade and base materials should be prepared per the General Geotechnical Design and Construction Considerations – EARTHWORK section.
11. Following the proofrolling as described in the General Geotechnical Design and Construction Considerations – EARTHWORK section, the application of subgrade treatment, base material, and paving materials can proceed per the recommendations in the geotechnical report and/or project plans. The placement of pavement materials or structural fills cannot take place on frozen ground.
12. The placement of aggregate base material should conform to the jurisdictional guidelines. In general the materials should be provided by an accredited supplier, and the material should meet the standards of ASTM C-33. Material that has been stockpiled and exposed to weather including wind and rain should be retested for compliance since fines could be lost. Frozen material cannot be used.
13. The placement of asphalt material should conform to the jurisdictional guidelines. In general the materials should be provided by an accredited supplier, and the material should meet the standards of ASTM C-33. The material can be placed in a screed by end-dumping, or it can be placed directly on the paving surface. The temperature of the mix at placement should generally be on the order of 300 degrees Fahrenheit at time of placement and screeding.
14. Compaction of the screeded asphalt should begin as soon as practical after placement, and initial rolling should be performed before the asphalt has cooled significantly. Compaction equipment should have vibratory capabilities, and should be of appropriate size and weight given the thickness of the lift being placed and the sloping of the ground surface.
15. In cold and/or windy weather, the cooling of the screeded asphalt is a quality issue, so preparations should be made to perform screeding immediately after placement, and compaction immediately after screeding.
16. Quality control testing of the asphalt should be performed during placement to verify compaction and mix design properties are being met and that delivery temperatures are correct. Results of testing data from asphalt laboratory testing should be provided within 24 hours of the paving.

SITE GRADING AND DRAINAGE

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Site grading and drainage for this section is generally meant to describe the effect of new construction on surface hydrology, which impacts the flow of rainfall or other water running across, onto or off-of, a newly constructed or modified development.
3. This section does not apply to the construction of site grading and drainage features. Recommendations for the construction of such features are covered in General Geotechnical Design and Construction Considerations for Earthwork – Structural Fills section and Underground Pipeline Installation – Backfill section.
4. In general, surface water flows should be directed towards storm drains, natural channels, retention or detention basins, swales, and/or other features specifically designed to capture, store, and or transmit them to specific off-site outfalls.
5. The surface water flow design is generally performed by a site civil engineer, and it can be impacted by hydrology, roof lines, and other site structures that do not allow for water to infiltrate into the soil, and that modify the topography of the site.
6. Soil permeability, density, and strength properties are relevant to the design of storm drain systems, including dry wells, retention basins, swales, and others. These properties are usually only provided in a geotechnical report if specifically requested, and recommendations will be provided in the geotechnical report in those cases.
7. Structures or site features that are not a part of the surface water drainage system should not be exposed to surface water flows, standing water or water infiltration. In general, roof drains and scuppers, exterior slabs, pavements, landscaping, etc. should be constructed to drain water away from structures and foundations. The purpose of this is to reduce the opportunity for water damage, erosion, and/or altering of structural soil properties by wetting. In general, a 5 percent or more slope away from foundations, structural fills, slopes, structures, etc. should be maintained.
8. Special considerations should be used for slopes and retaining walls, as described in the General Geotechnical Design and Construction Considerations - LATERALLY LOADED STRUCTURES section.
9. Additionally, landscaping features including irrigation emitters and plants that require large amounts of water should not be placed near to new structures, as they have the potential to alter soil moisture states. Changing of the moisture state of soil that provides structural support can lead to damage to the supported structures.

Appendix G: Storm Sewer Basin Map



PIPE BASIN	TOTAL (SF)	TOTAL (AC)	BLDG (SF)	BLDG (AC)	PAVEMENT (SF)	PAVEMENT (AC)	OPEN (SF)	OPEN (AC)
A	687	0.02	0	0.00	687	0.02	0	0.00
B	2,885	0.07	0	0.00	2,885	0.07	0	0.00
C	495	0.01	0	0.00	495	0.01	0	0.00
D	3,704	0.09	0	0.00	3,704	0.09	0	0.00
E	1,273	0.03	1,273	0.03	0	0.00	0	0.00
F	4,882	0.11	0	0.00	4,882	0.11	0	0.00
G	2,207	0.05	0	0.00	2,207	0.05	0	0.00
H	1,403	0.03	0	0.00	1,403	0.03	0	0.00
I	850	0.02	850	0.02	0	0.00	0	0.00
J	2,061	0.05	0	0.00	2,061	0.05	0	0.00
K	908	0.02	908	0.02	0	0.00	0	0.00
L	1,165	0.03	0	0.00	1,165	0.03	0	0.00
M	1,296	0.03	0	0.00	624	0.01	672	0.02
N	1,785	0.04	0	0.00	1,785	0.04	0	0.00
O	1,366	0.03	1,366	0.03	0	0.00	0	0.00

STORM SEWER BASIN MAP

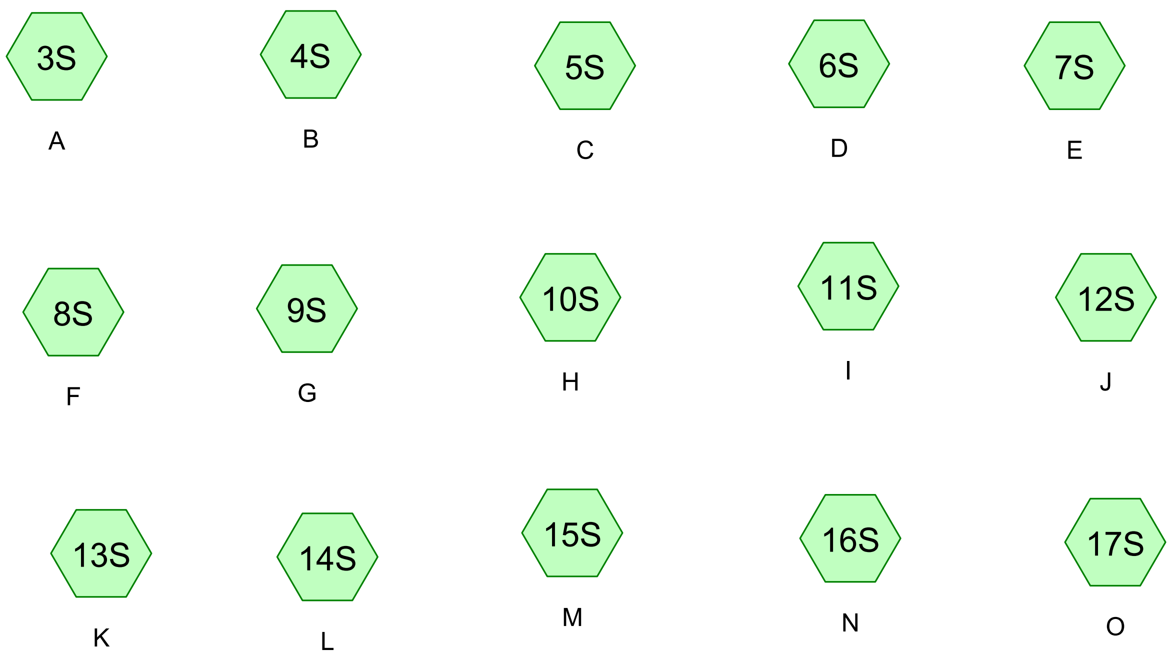
NORTH

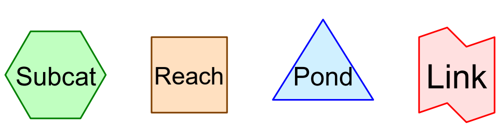
1"= 50'

50' 0 50' 100'

SCALE 1"=50' (11X17) FEET

Appendix H: Storm Sewer TR-55 Calculations





Routing Diagram for 230193300hydro
Prepared by Excel Engineering, Printed 5/16/2024
HydroCAD® 10.20-4a s/n 01178 © 2023 HydroCAD Software Solutions LLC

Time span=5.00-20.00 hrs, dt=0.01 hrs, 1501 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 3S: A Runoff Area=687 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.06 cfs 0.003 af

Subcatchment 4S: B Runoff Area=2,885 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.25 cfs 0.013 af

Subcatchment 5S: C Runoff Area=495 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.04 cfs 0.002 af

Subcatchment 6S: D Runoff Area=3,704 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.32 cfs 0.017 af

Subcatchment 7S: E Runoff Area=1,273 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.11 cfs 0.006 af

Subcatchment 8S: F Runoff Area=4,882 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.42 cfs 0.022 af

Subcatchment 9S: G Runoff Area=2,207 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.19 cfs 0.010 af

Subcatchment 10S: H Runoff Area=1,403 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.12 cfs 0.006 af

Subcatchment 11S: I Runoff Area=850 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.07 cfs 0.004 af

Subcatchment 12S: J Runoff Area=2,061 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.18 cfs 0.009 af

Subcatchment 13S: K Runoff Area=908 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.08 cfs 0.004 af

Subcatchment 14S: L Runoff Area=1,165 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.10 cfs 0.005 af

Subcatchment 15S: M Runoff Area=1,296 sf 48.15% Impervious Runoff Depth>1.78"
Tc=6.0 min CN=92 Runoff=0.09 cfs 0.004 af

Subcatchment 16S: N Runoff Area=1,785 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.15 cfs 0.008 af

Subcatchment 17S: O Runoff Area=1,366 sf 100.00% Impervious Runoff Depth>2.34"
Tc=6.0 min CN=98 Runoff=0.12 cfs 0.006 af

Summary for Subcatchment 3S: A

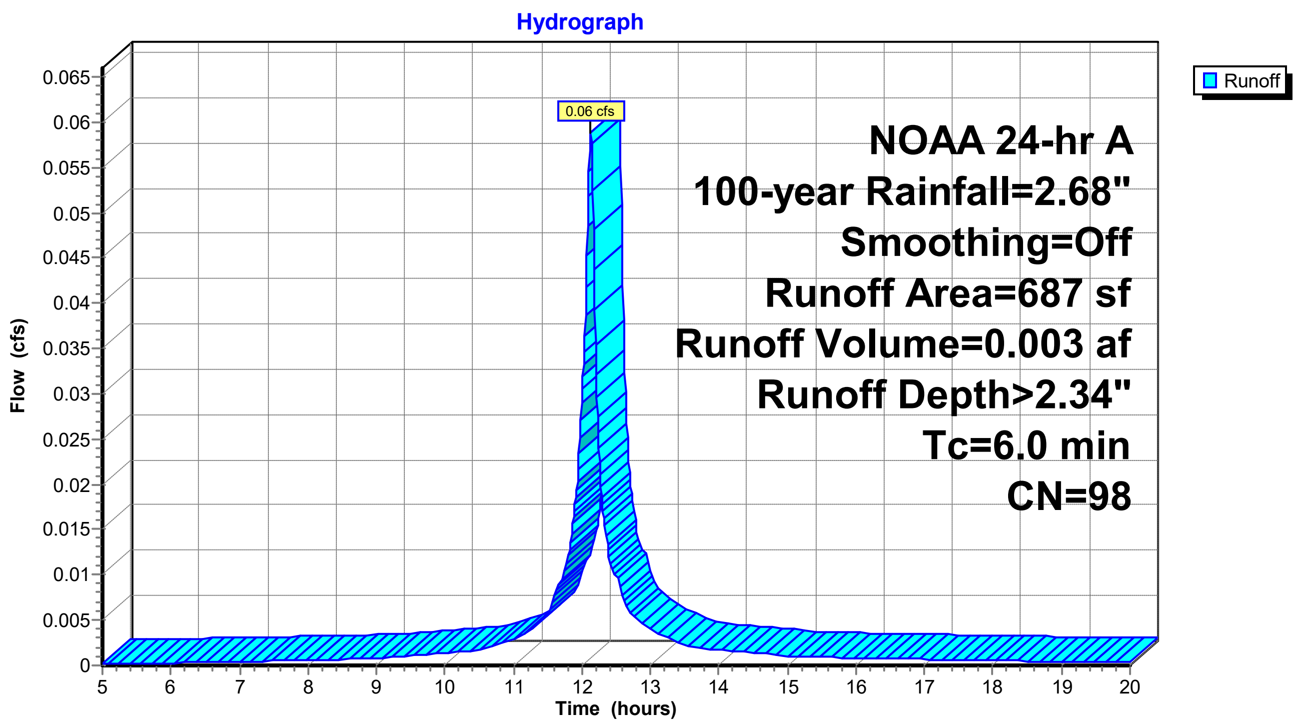
Runoff = 0.06 cfs @ 12.13 hrs, Volume= 0.003 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	687	98	
	687		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 3S: A



Summary for Subcatchment 4S: B

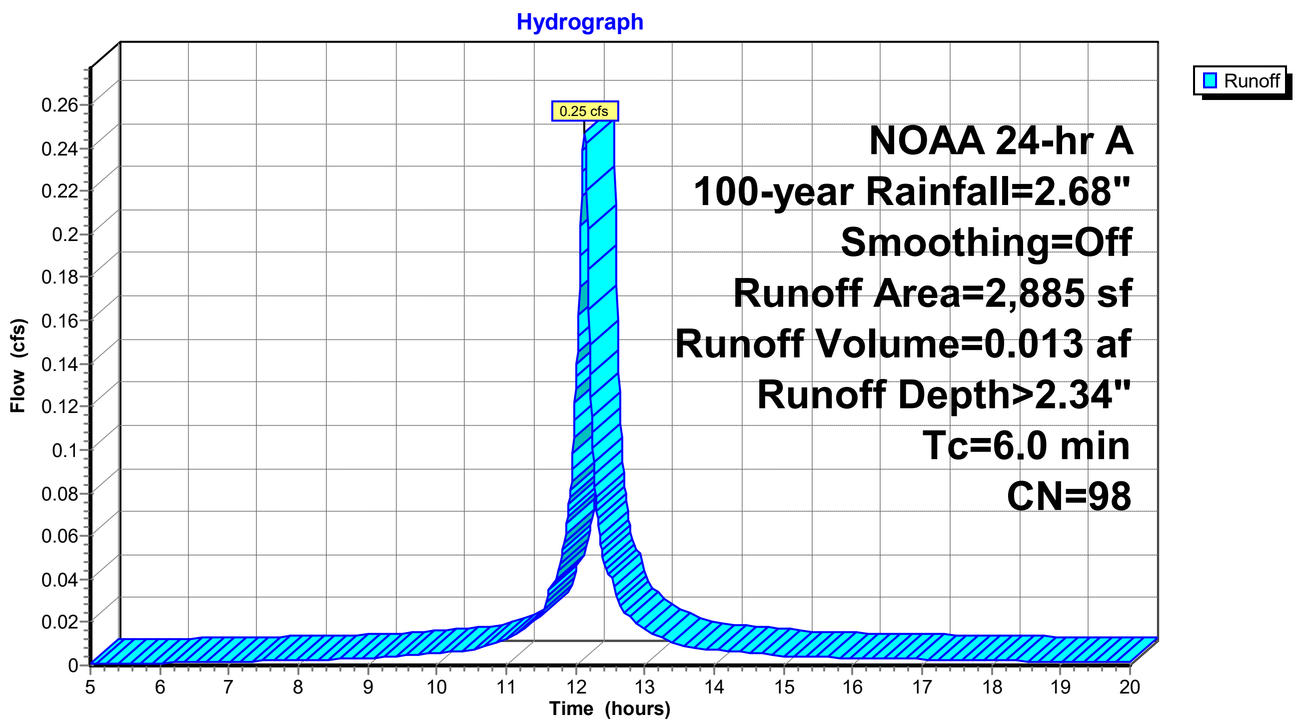
Runoff = 0.25 cfs @ 12.13 hrs, Volume= 0.013 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	2,885	98	
	2,885		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 4S: B



Summary for Subcatchment 5S: C

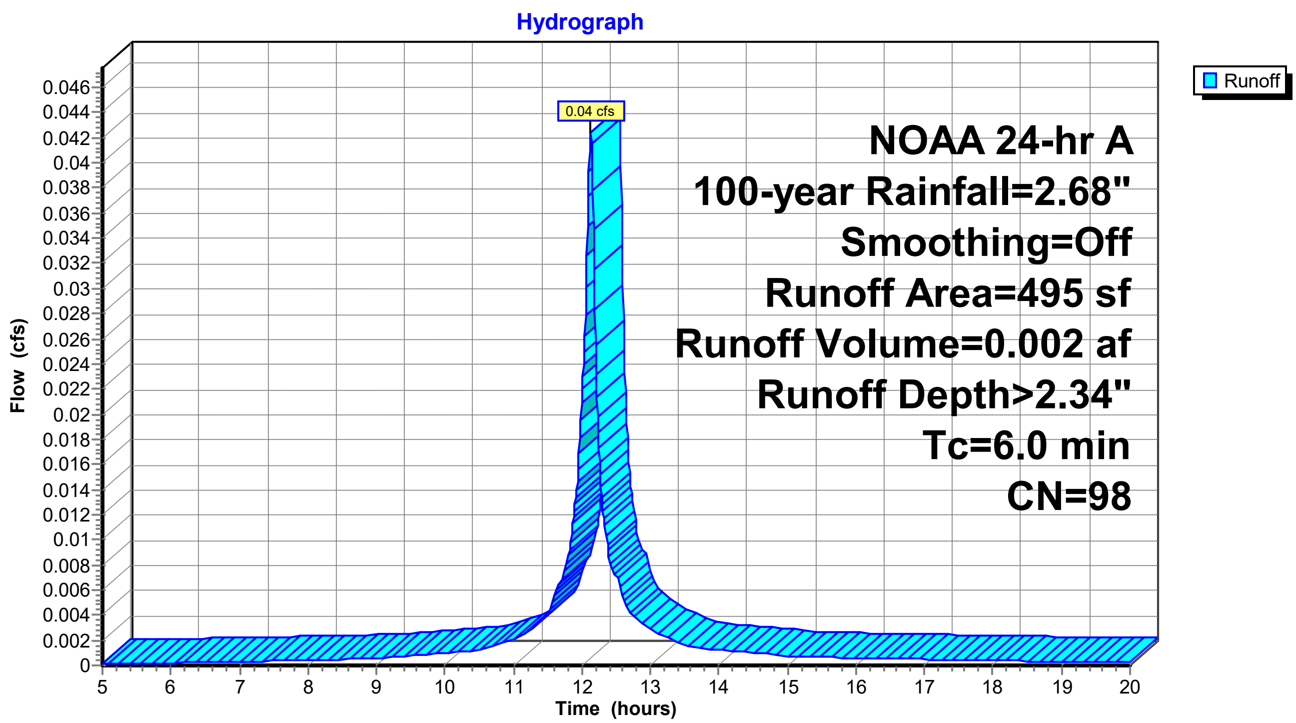
Runoff = 0.04 cfs @ 12.13 hrs, Volume= 0.002 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	495	98	
	495		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 5S: C



Summary for Subcatchment 6S: D

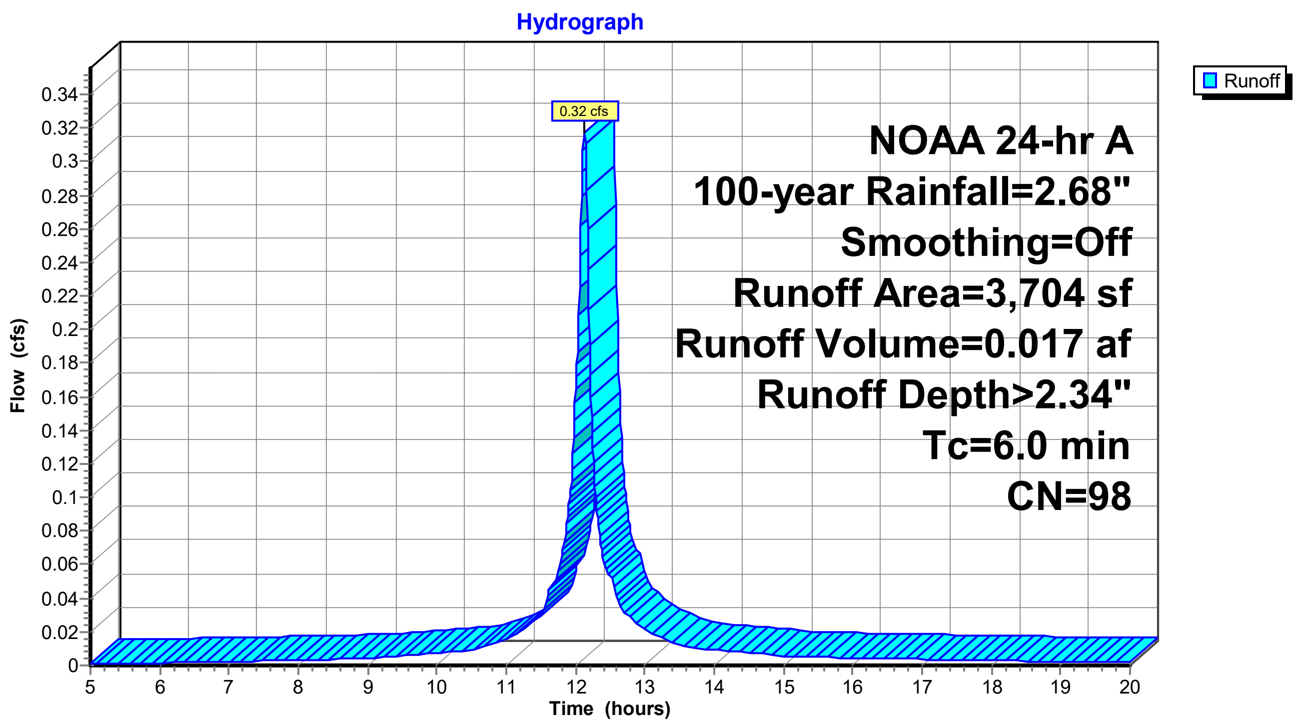
Runoff = 0.32 cfs @ 12.13 hrs, Volume= 0.017 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	3,704	98	
	3,704		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 6S: D



Summary for Subcatchment 7S: E

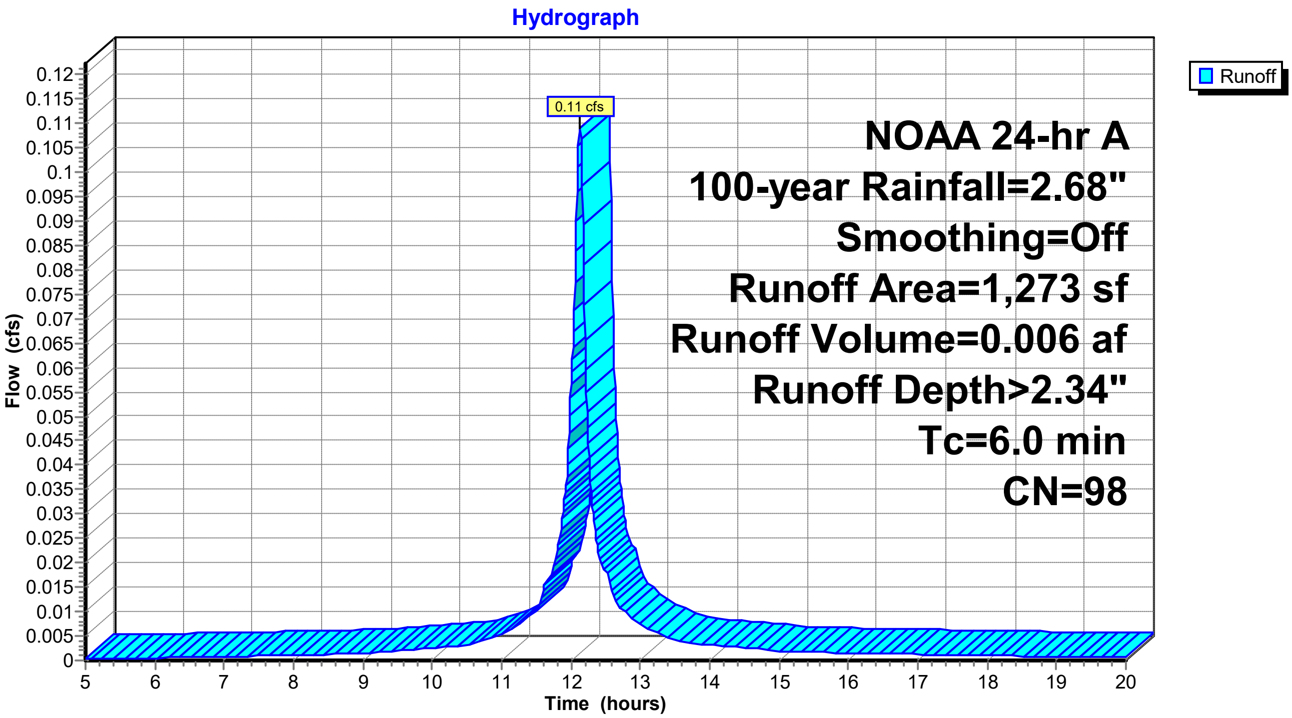
Runoff = 0.11 cfs @ 12.13 hrs, Volume= 0.006 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	1,273	98	
	1,273		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 7S: E



Summary for Subcatchment 8S: F

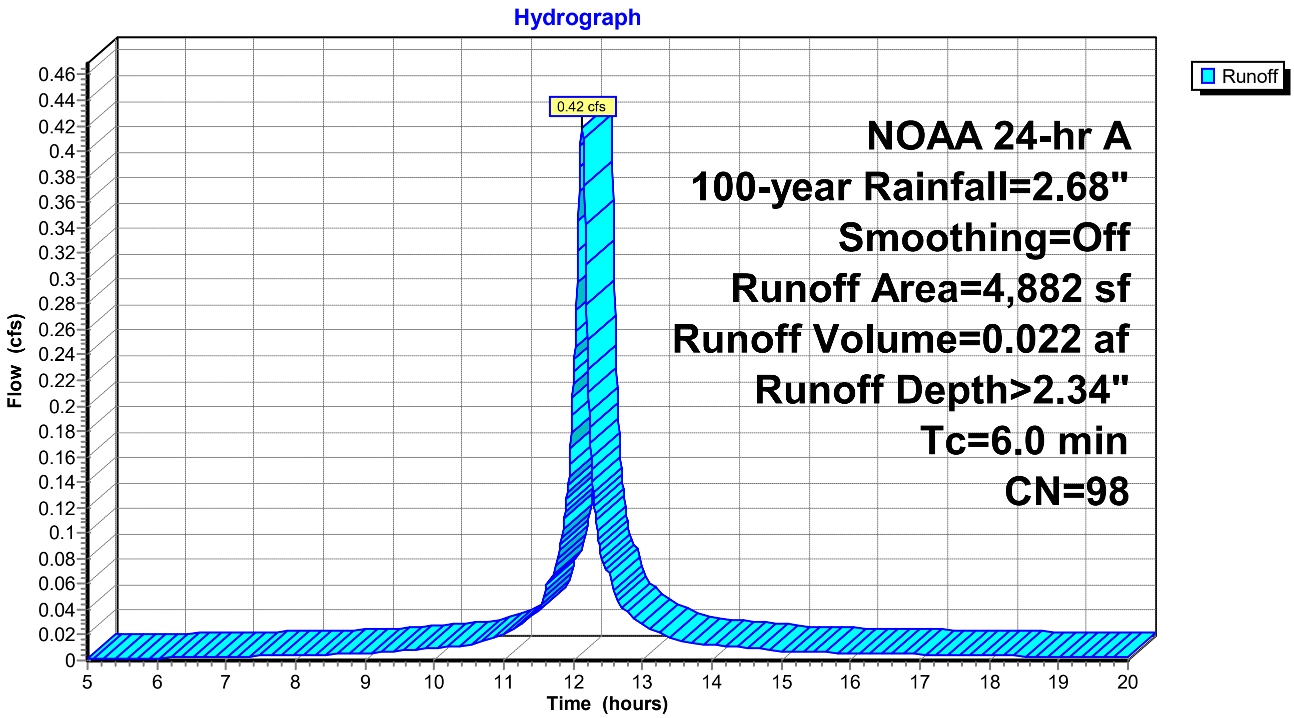
Runoff = 0.42 cfs @ 12.13 hrs, Volume= 0.022 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	4,882	98	
	4,882		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 8S: F



Summary for Subcatchment 9S: G

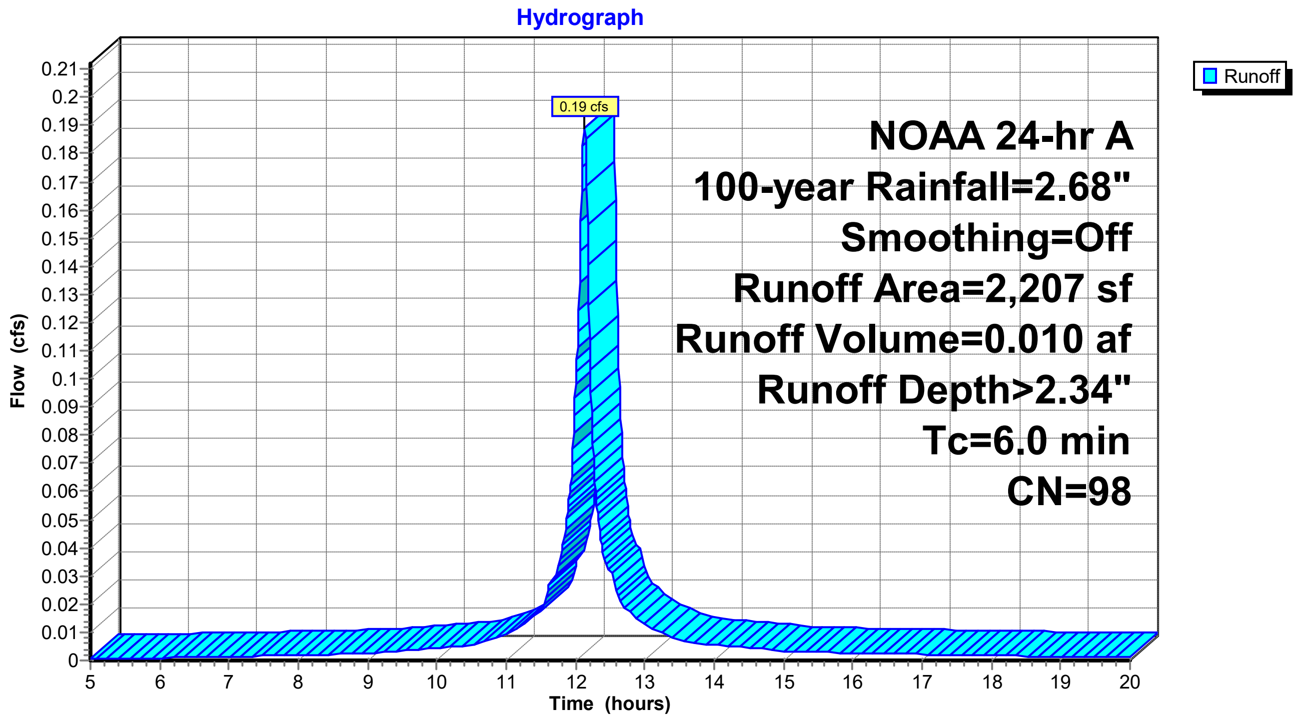
Runoff = 0.19 cfs @ 12.13 hrs, Volume= 0.010 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	2,207	98	
	2,207		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 9S: G



Summary for Subcatchment 10S: H

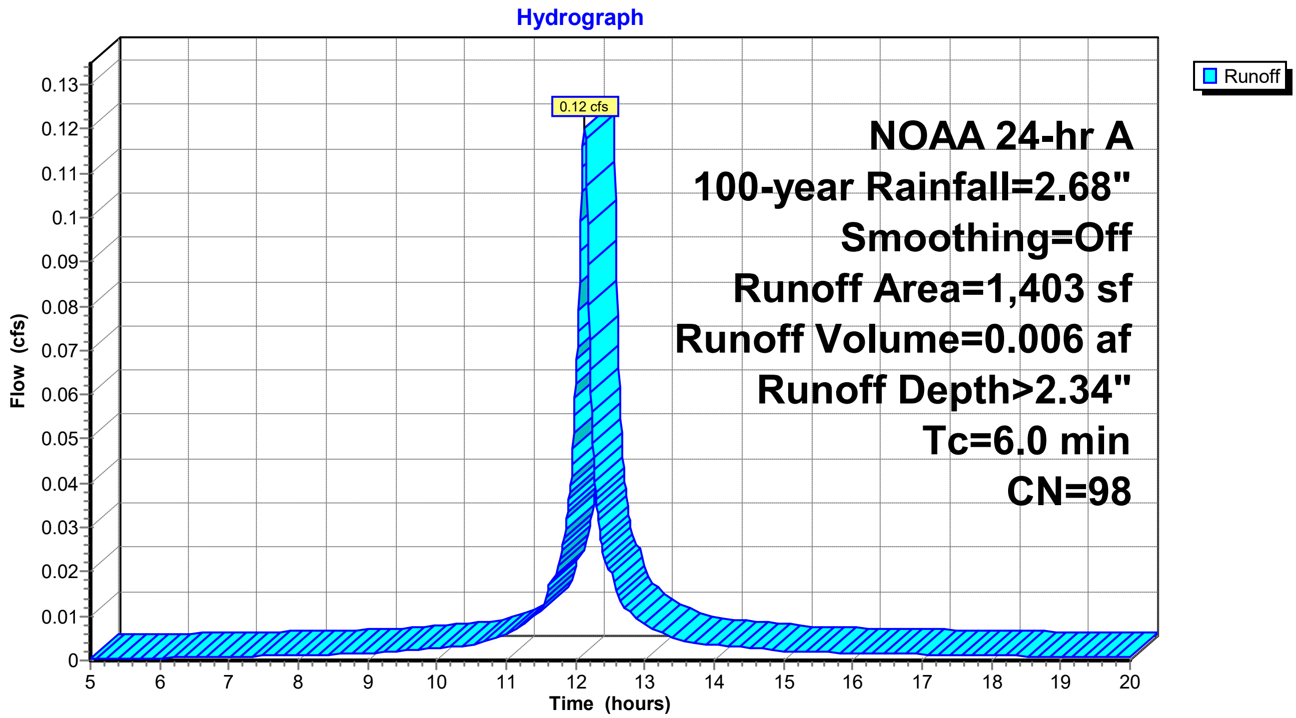
Runoff = 0.12 cfs @ 12.13 hrs, Volume= 0.006 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	1,403	98	
	1,403		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 10S: H



Summary for Subcatchment 11S: I

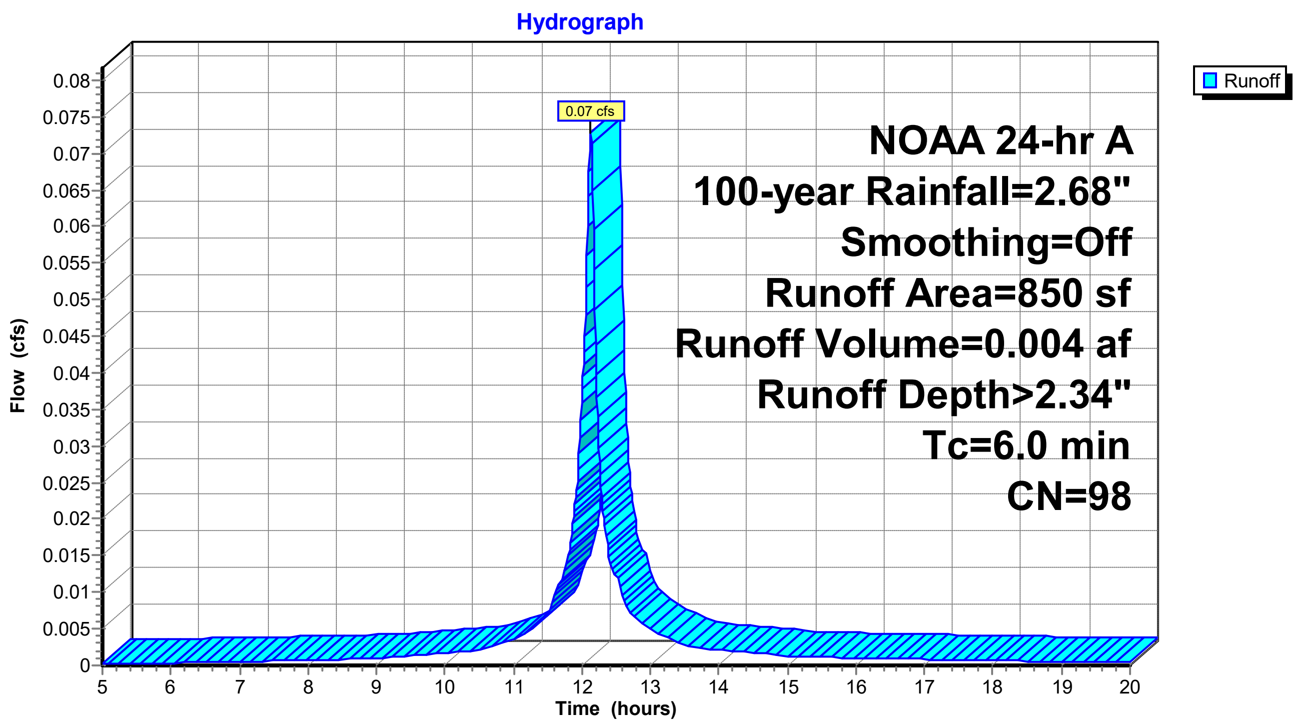
Runoff = 0.07 cfs @ 12.13 hrs, Volume= 0.004 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	850	98	
	850		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 11S: I



Summary for Subcatchment 12S: J

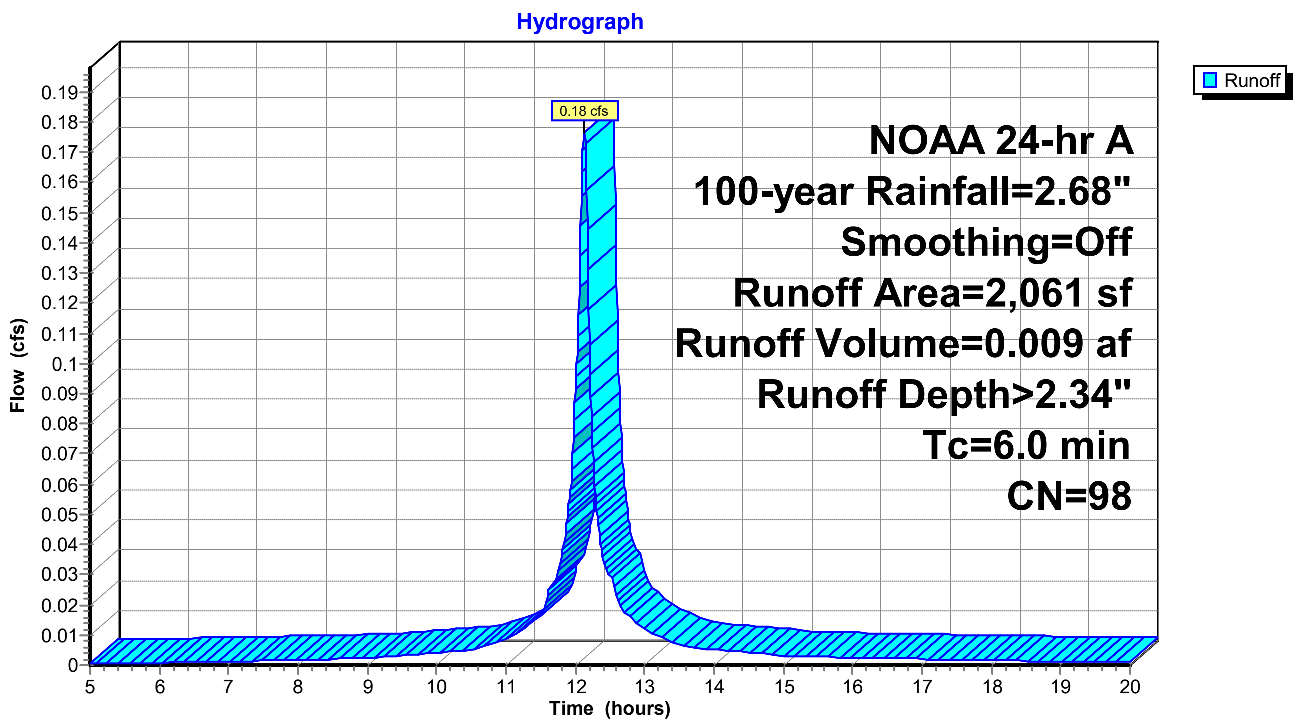
Runoff = 0.18 cfs @ 12.13 hrs, Volume= 0.009 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	2,061	98	
	2,061		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 12S: J



Summary for Subcatchment 13S: K

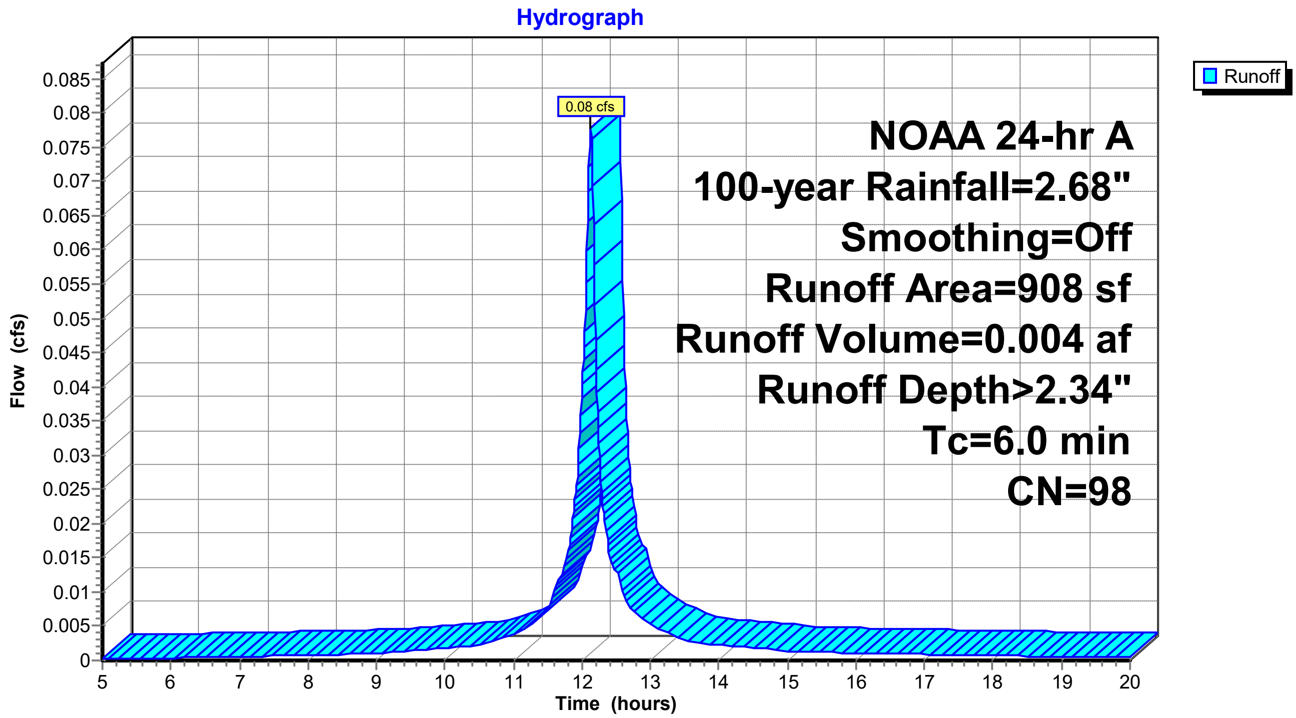
Runoff = 0.08 cfs @ 12.13 hrs, Volume= 0.004 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	908	98	
	908		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 13S: K



Summary for Subcatchment 14S: L

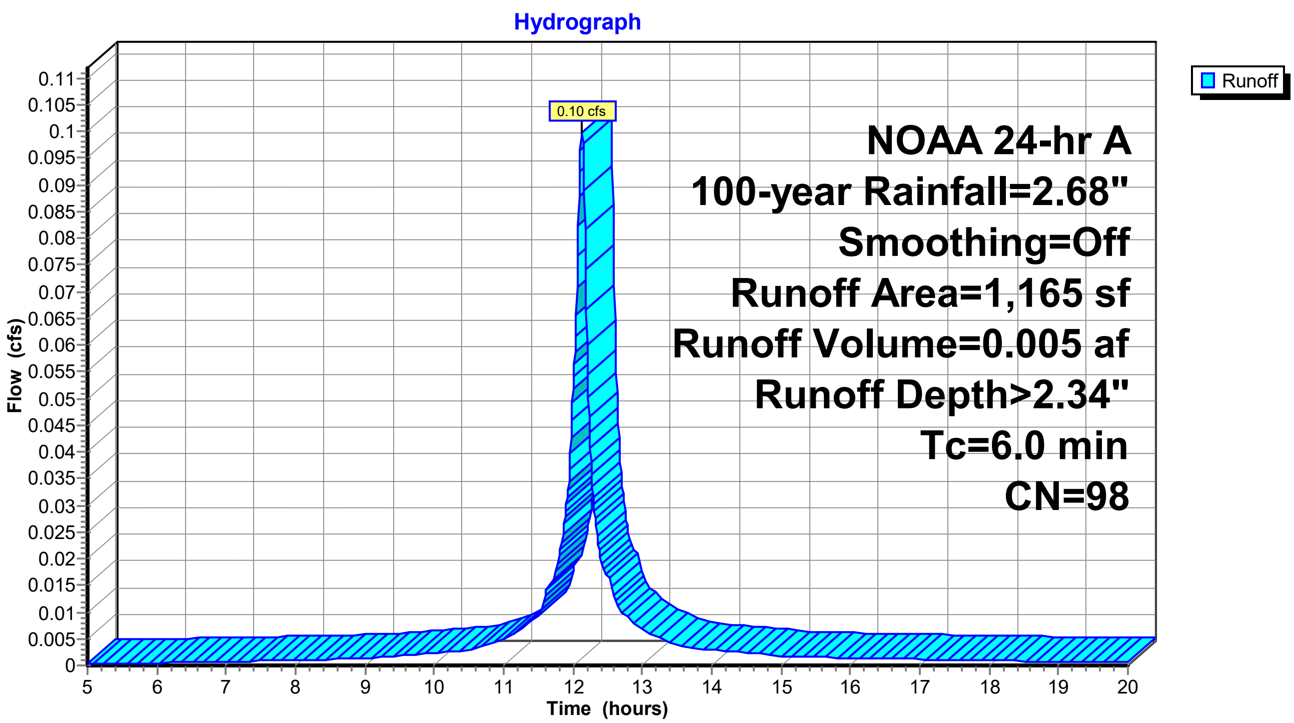
Runoff = 0.10 cfs @ 12.13 hrs, Volume= 0.005 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	1,165	98	
	1,165		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 14S: L



Summary for Subcatchment 15S: M

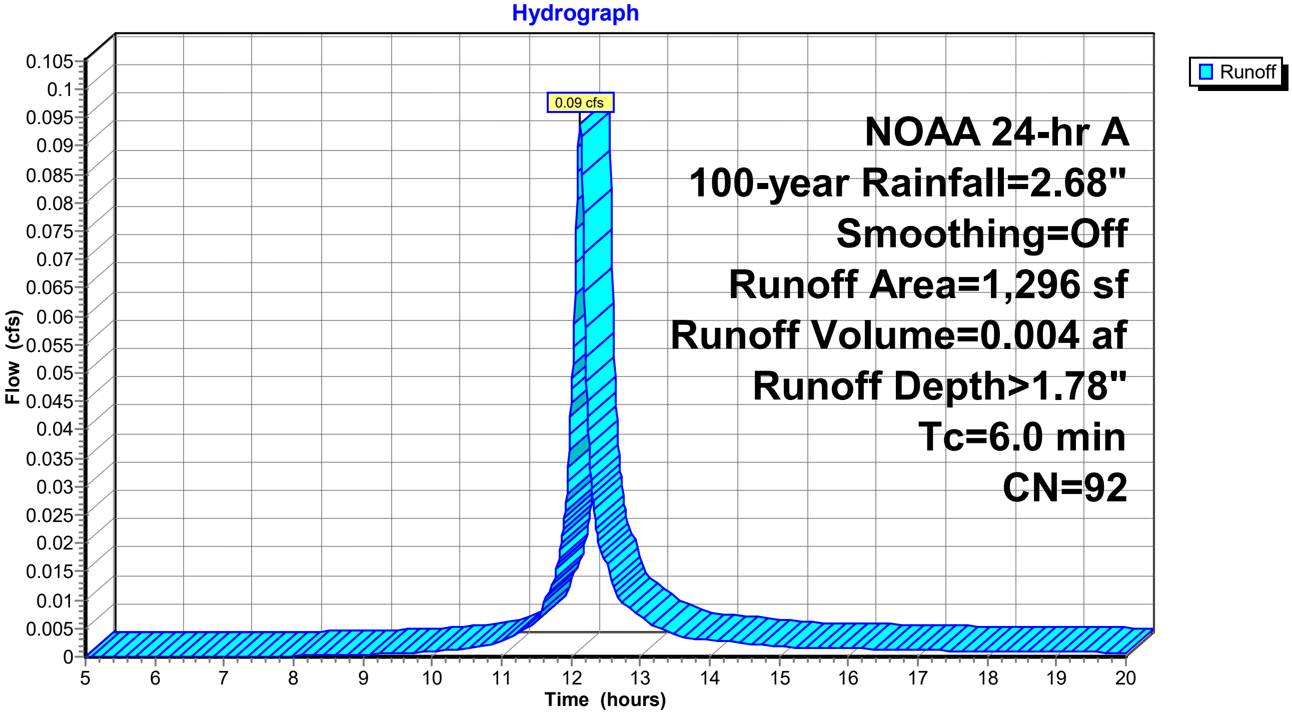
Runoff = 0.09 cfs @ 12.13 hrs, Volume= 0.004 af, Depth> 1.78"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	624	98	
*	672	86	
	1,296	92	Weighted Average
	672		51.85% Pervious Area
	624		48.15% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 15S: M



Summary for Subcatchment 16S: N

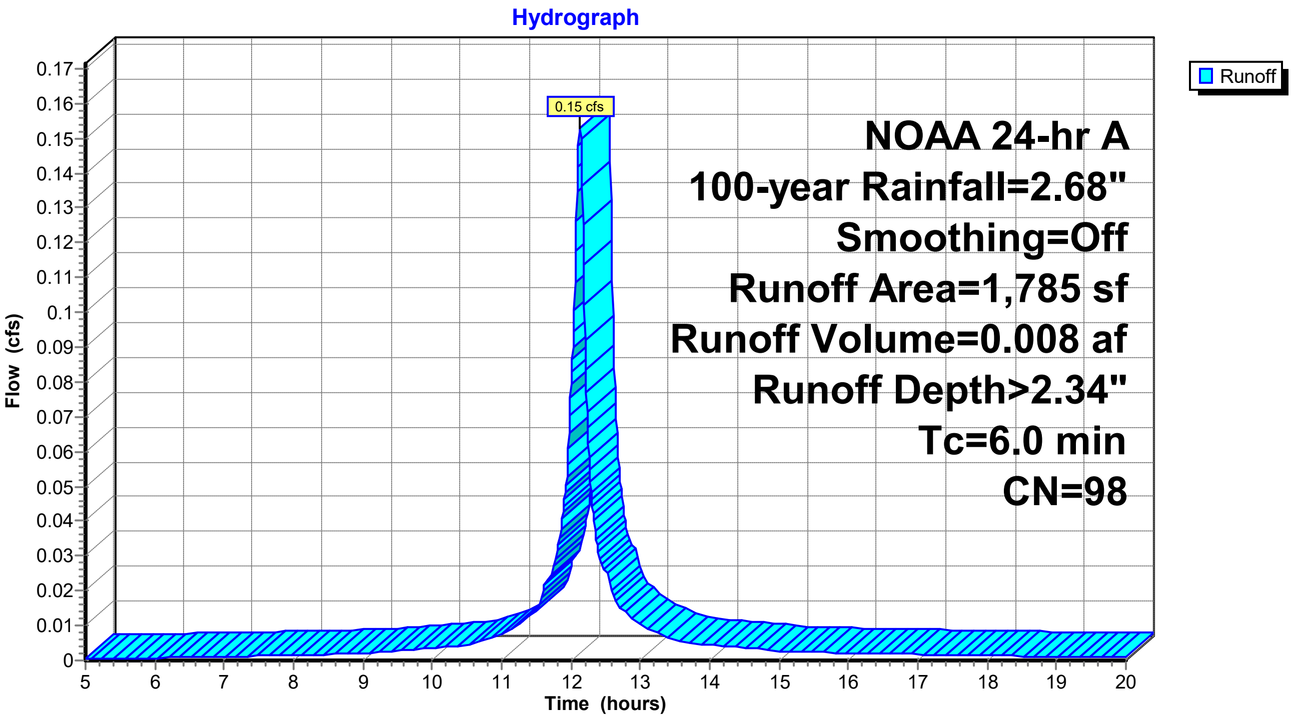
Runoff = 0.15 cfs @ 12.13 hrs, Volume= 0.008 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	1,785	98	
	1,785		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 16S: N



Summary for Subcatchment 17S: O

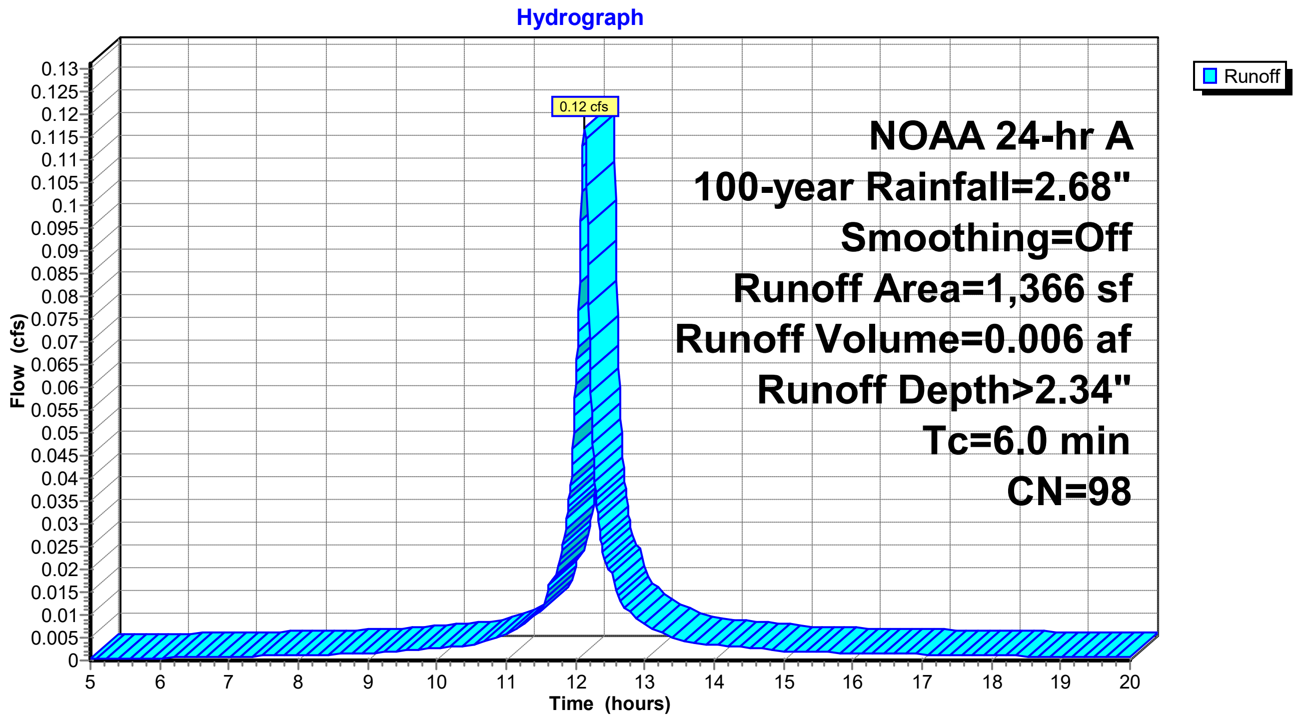
Runoff = 0.12 cfs @ 12.13 hrs, Volume= 0.006 af, Depth> 2.34"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.01 hrs
NOAA 24-hr A 100-year Rainfall=2.68", Smoothing=Off

	Area (sf)	CN	Description
*	1,366	98	
	1,366		100.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 17S: O



Appendix I: Storm Sewer Manning's Spreadsheet

Excel Engineering Project No. 230193300 ` Project Name MCW FIESTA PARK

Pipe Data				Pipe Capacity (100-yr)				
Pipe ID	Diameter (FT)	Slope (FT/FT)	Manning's n	Basin ID	Total Flow (cfs)	Total Flow (gpm)	Full Flow Capacity (cfs)	Full Flow Capacity (gpm)
A	0.5	0.0030	0.012	A	0.06	27	0.33	150
B	0.5	0.0100	0.012	C	0.04	18	0.61	274
C	0.67	0.0030	0.012	A,B,C	0.35	157	0.72	323
D	0.5	0.0030	0.012	E	0.11	49	0.33	150
E	0.83	0.0030	0.012	A,B,C,D,E	0.78	350	1.30	585
F	0.5	0.0030	0.012	O	0.12	54	0.33	150
G	1	0.0030	0.012	A,B,C,D,E,F,O	1.32	592	2.12	951
H	1	0.0030	0.012	A,B,C,D,E,F,G,O	1.51	678	2.12	951
I	0.5	0.0030	0.012	H	0.12	54	0.33	150
J	0.5	0.0100	0.012	I	0.07	31	0.61	274
K	0.67	0.0030	0.012	H,I,J	0.37	166	0.72	323
L	0.5	0.0100	0.012	K	0.08	36	0.61	274
M	0.67	0.0030	0.012	H,I,J,K,L	0.55	247	0.72	323
N	0.5	0.0625	0.012	M	0.09	40	1.52	684
O	0.83	0.0030	0.012	H,I,J,K,L,M,N	0.79	355	1.30	585

Full Flow Capacity based off Manning's Equation

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} a$$

Where:
- Q = Full Flow Capacity of Pipe (cfs)
- n = manning's roughness coefficient
- R = hydraulic radius (ft) (D/4)
- s = hydraulic gradient, slope (ft/ft)
- a = flow area (sq. ft.)

Typical Manning's n	
HDPE	0.012
PVC	0.012
Concrete	0.013
CMP	0.024

*Total Flow calculated via TR-55 hydrologic calculations. Reference Storm Pipe Basin Map & TR-55 Calculations

Appendix J: Post Construction Operation and Maintenance Plan

The owner of the property affected shall inspect and maintain the following stormwater management systems frequently, especially after heavy rainfalls, but at least on an annual basis unless otherwise specified.	
STORMWATER FACILITY	**TYPE OF ACTION**
1. Lawn and Landscaped Areas	All lawn areas shall be kept clear of any materials that block the flow of stormwater. Rills and small gullies shall immediately be filled and seeded or have sod placed in them. The lawn shall be kept mowed, tree seedlings shall be removed, and litter shall be removed from landscaped areas.
2. Rip Rap	All rip rap showing signs of erosion or scour shall be repaired, reinforced, and revegetated immediately. Rip rap should be kept clean of vegetation and sediment. All rip rap shall be repaired to the construction plan requirements.
3. Catch Basin/Curb Inlet Grates	The grate openings to these structures must be cleared of any clogging or the blocking of stormwater flow from getting into the stormwater conveyance system of any kind.
4. Detention/Infiltration Basin	Inspections shall occur at minimum every 3 months. Inspections shall include the spreader, overflow spillway, and the condition of vegetation. To maintain vegetation, the first mowing of newly planted seed shall occur once it reaches a height of 10 to 12 inches. Mowing shall reduce the height of plants to 5 to 6 inches. After establishment, if burning cannot be accommodated, mowing shall occur once in the fall after November 1st. Mowing shall reduce the height of plants to 5 to 6 inches. If burning can take place, beginning the second year, burning shall occur in the early spring prior to May 1st, or in late fall after November 1st. Burning shall be done two consecutive years and then up to three years can pass before the next burning. Under no circumstances shall burning occur every other year. If standing water is observed over 50% of the basin floor 3 days after rainfall, the basin is considered clogged. If this ever occurs, remove the top 2 to 3 inches, chisel plow and add topsoil and compost. If deep tilling is used, the basin shall be drained and soils dried to a depth of 8 inches. Replant with turf grass. If clogging again occurs, the basin shall be replanted with prairie style vegetation. During winter conditions, all draw down devices in the pond shall be opened to discourage the infiltration of high levels of chlorides. For enclosed basins, the use of chloride deicers shall be limited in the upland areas of the basin. Trash shall be removed as quickly as possible once observed.
5. Record of Maintenance	The operation and maintenance plan shall remain onsite and be available for inspection when requested. When requested, the

	owner shall make available for inspection all maintenance records to the department or agent for the life of the system.