STORMWATER CALCULATIONS

Prepared For:

Scott Martin Construction, Inc.

PO Box 5850

Salem, OR 97304

Project Location:

Riverbend Road Site Phase II

2499 Wallace Rd NW

Salem, OR 97304

Prepared By:

REGISTERED PROFESSIONAL ENGINEER
88632PE
OREGON
NOV. 10, 2020
ANDREW A. TWEET
RENEWS: 6/30/2022

Westech Engineering, Inc.
3841 Fairview Industrial Drive SE, Suite 100
Salem, OR 97302
(503) 585-2474 FAX: (503) 585-3986

J.O. 3168.3000.0

July 2021

TABLE OF CONTENTS

LIST OF TABLES

Table **Page**

APPENDICES

PROJECT OVERVIEW & DESCRIPTION

1.1 SIZE & LOCATION OF PROJECT

The proposed project is the second phase of the Riverbend Road Site project and sits on 8-acres located at 2499 Wallace Rd NW in Salem, Oregon. Refer to the Civil Drawings for a site map of the project area.

1.2 BRIEF DESCRIPTION OF PROJECT SCOPE AND PROPOSED IMPROVEMENTS

The project scope is to develop the property for multi-family housing consisting of 177 units, indoor recreation facilities, and an office. The project includes site preparation, construction of the facilities, and associated improvements.

1.3 DESCRIPTION OF SIZE OF WATERSHED DRAINING TO THE SITE

A drainage area of 6.84 acres from the site drains to the proposed stormwater facilities including an existing roadway at the southeast corner that connects to the first phase of the project. Some perimeter landscaping will not drain to the proposed stormwater facilities. Refer to the Developed Basin Map in Appendix A and the Civil Drawings for more details.

1.4 DESCRIPTION OF THE EXISTING SITE CONDITIONS, CONSTRAINTS, TREES & NATIVE VEGETATION, SENSITIVE AREAS & WATERWAYS

The existing site contains several rural residential lots predominately covered in grass with a few shrubs and trees located throughout. No other existing sensitive areas, waterways, etc. exist on-site.

1.5 SUMMARY OF GREEN STORMWATER INFRASTRUCTURE

Per Appendix 4E of the City of Salem (COS) Design Standards, a large project will be considered to have met the maximum extent feasible (MEF) requirement when the stormwater runoff from the total amount of new plus replaced impervious surfaces flows into an area set aside for GSI that is at least 10% of the total area of the new plus replaced impervious surfaces or at least 80% of all impervious area must be treated by GSI. The design implements GSI for the entire site and therefore meets MEF for GSI. See the Civil Drawings for more details.

1.6 REGULATORY PERMITS REQUIRED

A 1200-C permit from DEQ will be required since more than one acre is disturbed by the project. City of Salem permits are required. No other permits are required for this project.

1.7 100-YEAR EMERGENCY STORM ESCAPE ROUTES

Please refer to the Developed Basin Map in Appendix A for emergency overflow routes.

2.1 DEPTH TO GROUNDWATER

The proposed stormwater design does not implement infiltration facilities and therefore depth to groundwater is not applicable. However, groundwater is predicted to be as shallow as 10-12 feet below ground surface per the Geotechnical Report in Appendix A. No groundwater was encountered during testing with test pits dug up to 21 feet below ground surface.

2.2 MAXIMUM INFILTRATION AND VEGETATIVE TREATMENT

Measured infiltration rates onsite were zero or nearly zero per the Geotechnical Report. Therefore infiltration facilities are not included in the stormwater design. The proposed stormwater design will treat and detain the entire site with vegetated swales and a dry detention basin. Since stormwater from the entire site will be treated via GSI facilities, GSI has been implemented to the maximum extent feasible.

2.3 SOIL INFORMATION

The pre-developed project site contains approximately 50% hydrologic soil group C soils and 50% group C/D soils. Per the NRCS Soil Report, a C/D classification indicates a D rating for natural soil conditions. Soils for 50% of the pre-developed and developed site are assumed D-rated. Refer to the NRCS Soils Report in Appendix B for more details.

2.4 HAZARDOUS MATERIAL

The owner is not aware of any hazardous material contamination onsite.

3.1 METHODS & SOFTWARE USED

HydroCAD modeling software was used to size the stormwater facilities. The Santa Barbara Unit Hydrograph Type 1A storm was used to model the required design storms. Per the City of Salem (COS) Design Standards the design storms used were the 1.38 inch, 24-hour (water quality storm), half the 2-year, 24-hour, the 10-year, 24-hour, the 25-year, 24-hour, and the 100-year, 24-hour storm events.

Table 1 | City of Salem 24-hour Design Storms

	24-Hour Rainfall Depths for Salem, OR						
Recurrence Interval, Years	WQ	2	5	10	25	50	100
24-Hour Depths, Inches	1.38	2.2	2.7	3.2	3.6	4.1	4.4

Source: City of Salem Administrative Rules Chapter 109 – Division 004 Appendix D

3.2 CURVE NUMBER AND TIME OF CONCENTRATION CALCULATIONS

Per the COS Design Standards, the pre-developed site is considered to be covered in a combination of woods and good-grass, which corresponds to a pre-developed curve number of 72 and 79 for hydrologic soil group C and D-rated soils, respectively.

The developed impervious areas were assigned a curve number of 98 which corresponds to paved/parking and roof areas. The pervious areas were assigned curve numbers of 74 and 80 which corresponds to open space with C and D-rated soils, respectively, per the City of Salem Design Standards.

Time of concentration (Tc) for the pre-developed conditions was calculated using sheet and shallow concentrated flow equations to be 40.8 minutes. See the Pre-Developed Basin Map in Appendix A for the flow path used and refer to the HydroCAD Summaries in Appendix C for calculations. A minimum time of concentration of 5 minutes is applied to the developed basins due to the minimum time-step used by the HydroCAD modeling software.

3.3 Treatment & Flow Control Sizing Calculations

The project site was analyzed as one basin for pre-developed conditions and two basins for developed stormwater runoff calculations. General basin characteristics of pre-developed and developed conditions are listed in Table 2 below. For more detail refer to the Basin Maps in Appendix A and the Civil Drawings.

Table 2 | General Basin Characteristics

Basin ID	Source (Roof/ Road/ Other)	Impervious Area (ac)	Pervious Area (ac)	Design Storms (cfs)				CN[1]
				½ 2 Yr	10 Yr	25 Yr	100 Yr	
Predeveloped Basin								
PD[2]	Native	-	6.84	0.14	0.89	1.20	1.88	76
Developed Basins								
Basin 1	Paved/ Roof/ Landscape	1.60	0.40	0.37	1.29	1.48	1.85	94
Basin 2	Road/ Landscape	3.08	1.76	0.71	2.72	3.15	4.04	90

[1] Area-weighted curve number (CN).
[2] PD = pre-developed site conditions (i.e., pre-developed release rates)

Two vegetated swales at the bottom of a dry detention basin are proposed to treat and detain the runoff from the site. Swale 1 will treat Basin 1, and Swale 2 will treat Basin 2. Both swales are part of the same detention basin. The detention basin has a lower section to detain the half 2-year storm event.

Table 3 compares the designed and allowable swale parameters during the water quality and conveyance storms for Swale 1 and Swale 2. Per the Design Standards, a Manning's "n" of 0.25 was used to design treatment of the water quality storm. Refer to the Civil Drawings and HydroCAD calculations for more details.

Table 3 | Vegetated Swale Design

COS Design Standards		Swale 1	Swale 2
Criteria	Allowable		
Manning's n – Water Quality	0.25	0.25	0.25
Maximum Water Quality Flow Depth (ft)	0.33	0.30	0.33
Maximum Water Quality Flow Velocity (fps)	0.90	0.17	0.21
Min hydraulic Residence Time (min)	9	9.8	9.1
Manning's n – Conveyance (100-yr)	0.03	0.03	0.03
Max. Conveyance Flow Depth (ft)	1.0	0.19	0.23
Max. Conveyance Flow Velocity (fps)	3.0	1.12	1.39
Min Length (ft)	100	100	115
Side Slope (ft:ft)	3:1	3:1	3:1
Longitudinal Slope (%)	-	0.5	0.6
Bottom Width (ft)	-	8	12

The design meets or exceeds all the allowed values in Section 4.4 of the COS Design Standards. The calculations in Table 3 above assumes free discharge from the swales which may not be the case during larger storms due to flow-control requirements. As shown in Table 4 below, the proposed flow-control design results in a water surface elevation of 146.59 feet reached in the water quality storm. This is 0.09-feet (1-inch) above the downstream finish grade of both swales (146.5-feet) and therefore meets the 4-inch maximum depth for vegetated swales in the water quality storm per the COS Design Standards.

Stormwater release from the detention basin facility is controlled by a flow-control manhole. See Table 4 below for a summary of facility outlet sizing and release rates. The 100-year storm is released by a 12-inch overflow riser in the flow-control manhole. An emergency overflow weir is provided along the east edge of the detention basin. Refer to the Developed Basin Map in Appendix A and the Civil Drawings for more details.

Table 4 | Facility Outlet Sizing and Release Rates

Outlet ID/ Storm Event	Orifice Size (in)	Orifice Elevation (ft)	Release Rate (cfs)	Allowed Release (cfs)	Peak WSE[1] (ft)	Emergency Overflow Elevation (ft)
½ - 2 Year	1.9	144.20	0.14	0.14	146.42	149.0
WQ Event	-	-	0.26	-	146.59	149.0
10 Year	5.0	146.50	0.84	0.89	147.52	149.0
25 Year	8.0[2]	147.60	0.98	1.20	147.71	149.0
100 Year	12.0[3]	147.80	1.76	1.88	147.92	149.0

[1] WSE = water surface elevation
[2] 25-year storm controlled by 8.0-inch wide weir cut from top of riser tee.
[3] 100-year storm controlled by 12-inch internal overflow riser.

3.4 Conveyance Capacity Calculations

The stormwater facilities were designed to convey the developed 100-year, 24-hour storm, which has a peak flow of 1.76 cfs released from the flow control manhole. Stormwater runoff is conveyed by 12-inch pipe from the flow-control structure to a 12-inch public storm drain in Wallace Rd. See the Civil Drawings for more detail. The 12-inch pipe has a full-flow capacity of 7.99 cfs using a slope of 5.0% and Manning’s n of 0.013, which exceeds the peak release rate from the stormwater facilities.

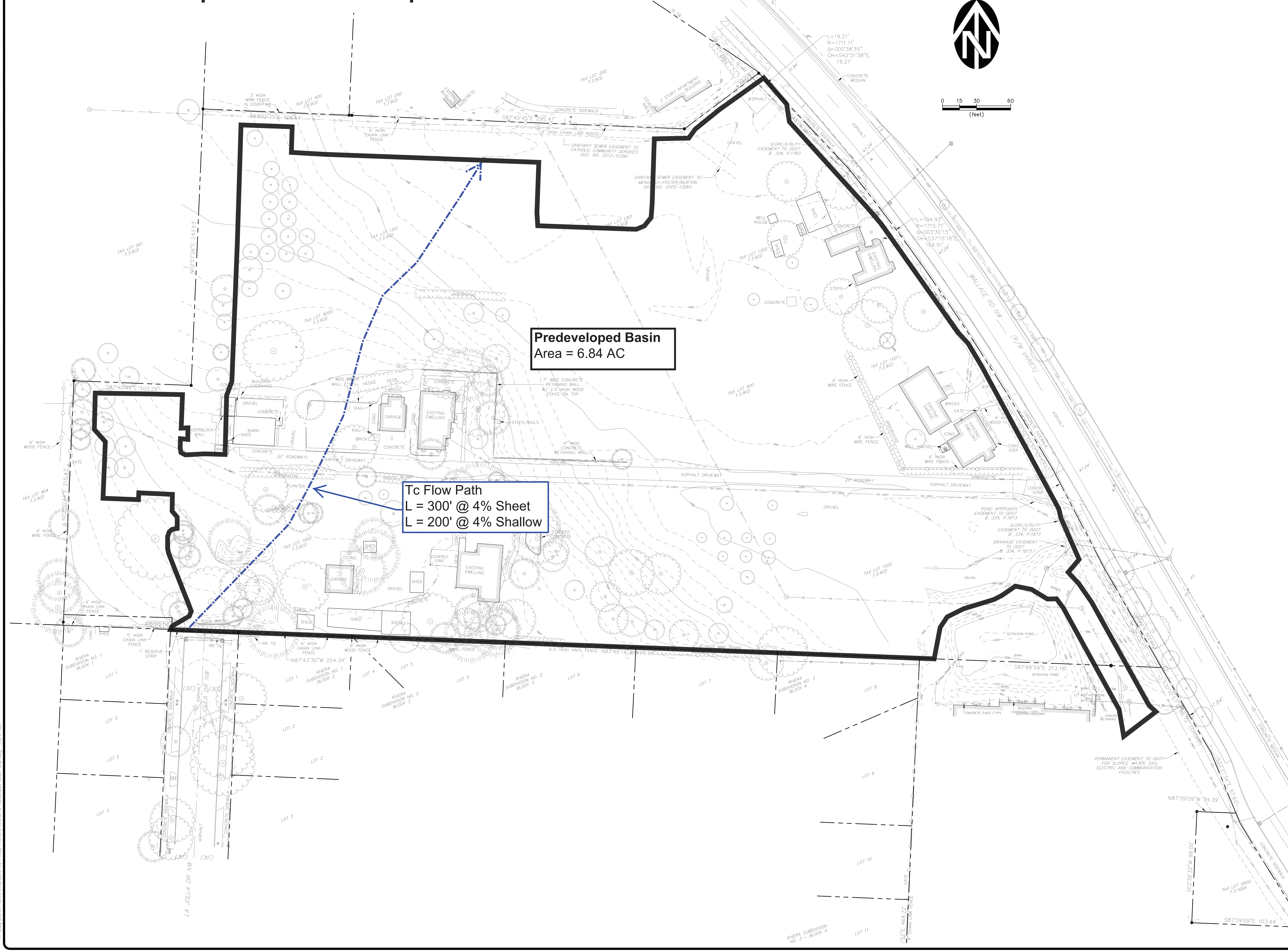
3.5 Summary

The stormwater system has been designed to release half the 2-year, 24-hour, the 10-year, 24-hour, 25-year, 24-hour, and the 100-year, 24-hour storm events at rates less than their respective pre-developed storm. The proposed design also treats the water quality storm. Therefore, the project meets the flow control and treatment requirements as set forth in Administrative Rule 109 Division 004 - Stormwater System.

APPENDIX A

BASIN MAPS

Predeveloped Basin Map



NO.	DATE	DESCRIPTION	BY
1		REVISIONS	

SCALE
HORIZ:
VERT:
DSN. SAW
DRN. AR
CKD. SAW
DATE: JAN 2021

REVIEW
REGISTERED PROFESSIONAL ENGINEER
OREGON
STEVEN A. WHI...
RENEWS: 6/30/2022

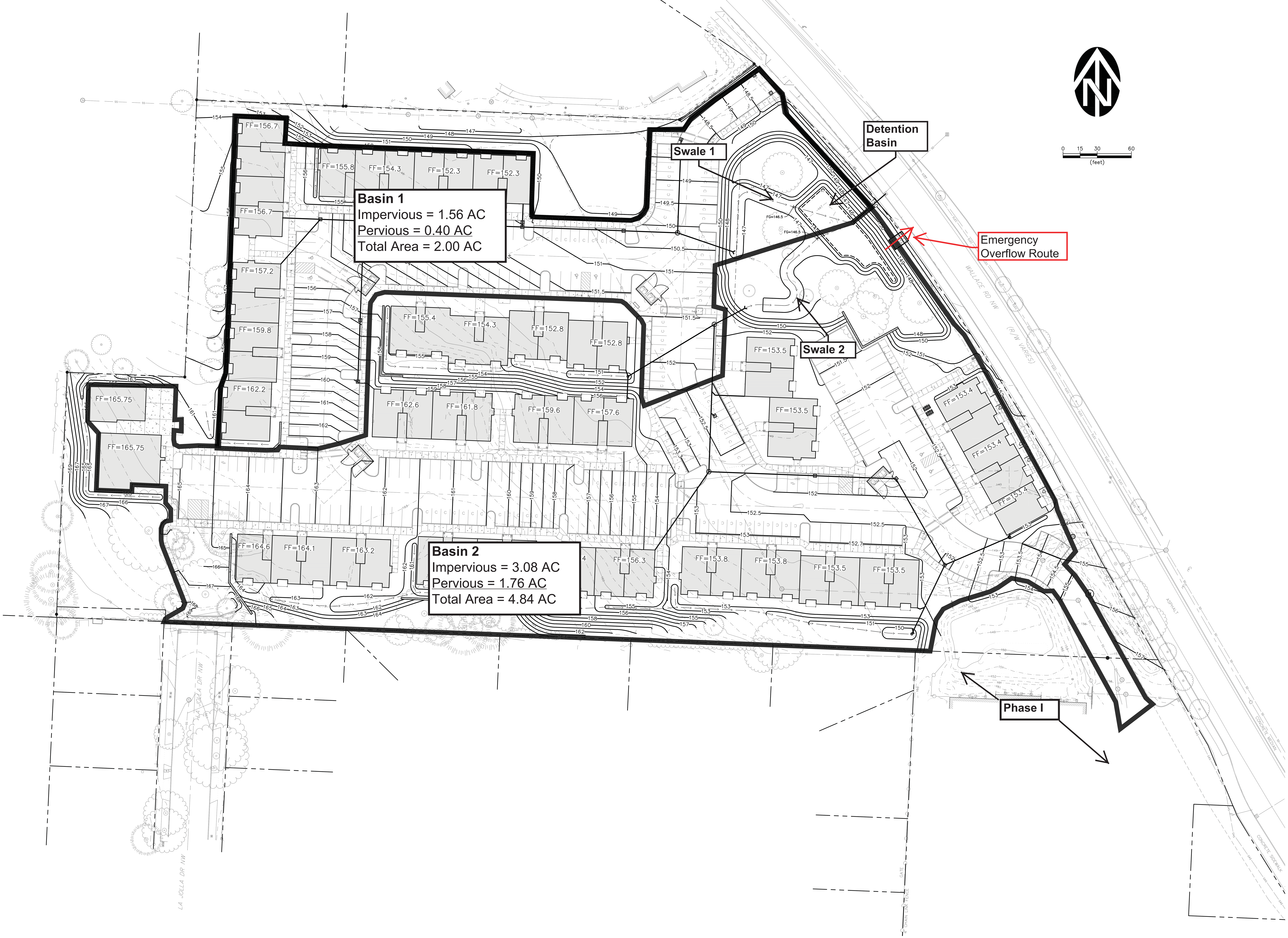
WESTECH ENGINEERING, INC.
CONSULTING ENGINEERS AND PLANNERS
3841 Fairview Industrial Dr. S.E., Suite 100, Salem, OR 97302
Phone: (503) 585-2474 Fax: (503) 585-3986
E-mail: westech@westech-eng.com

SCOTT MARTIN
RIVERBEND ROAD SITE PHASE II
PRE-DEVELOPED STORMWATER BASIN MAP

DRAWING

JOB NUMBER
3048.0000.0

Developed Basin Map



WESTECH ENGINEERING, INC.
CONSULTING ENGINEERS AND PLANNERS
3841 Fairview Industrial Dr. S.E., Suite 100, Salem, OR 97302
Phone: (503) 585-2474 Fax: (503) 585-3986
E-mail: westech@westech-eng.com

REVIEW

NO.	DATE	DESCRIPTION	BY
		REVISIONS	

SCALE
HORIZ:
VERT:

DSN. SAW
DRN. AR
CKD. SAW
DATE: JAN 2021

SCOTT MARTIN
RIVERBEND ROAD SITE PHASE II

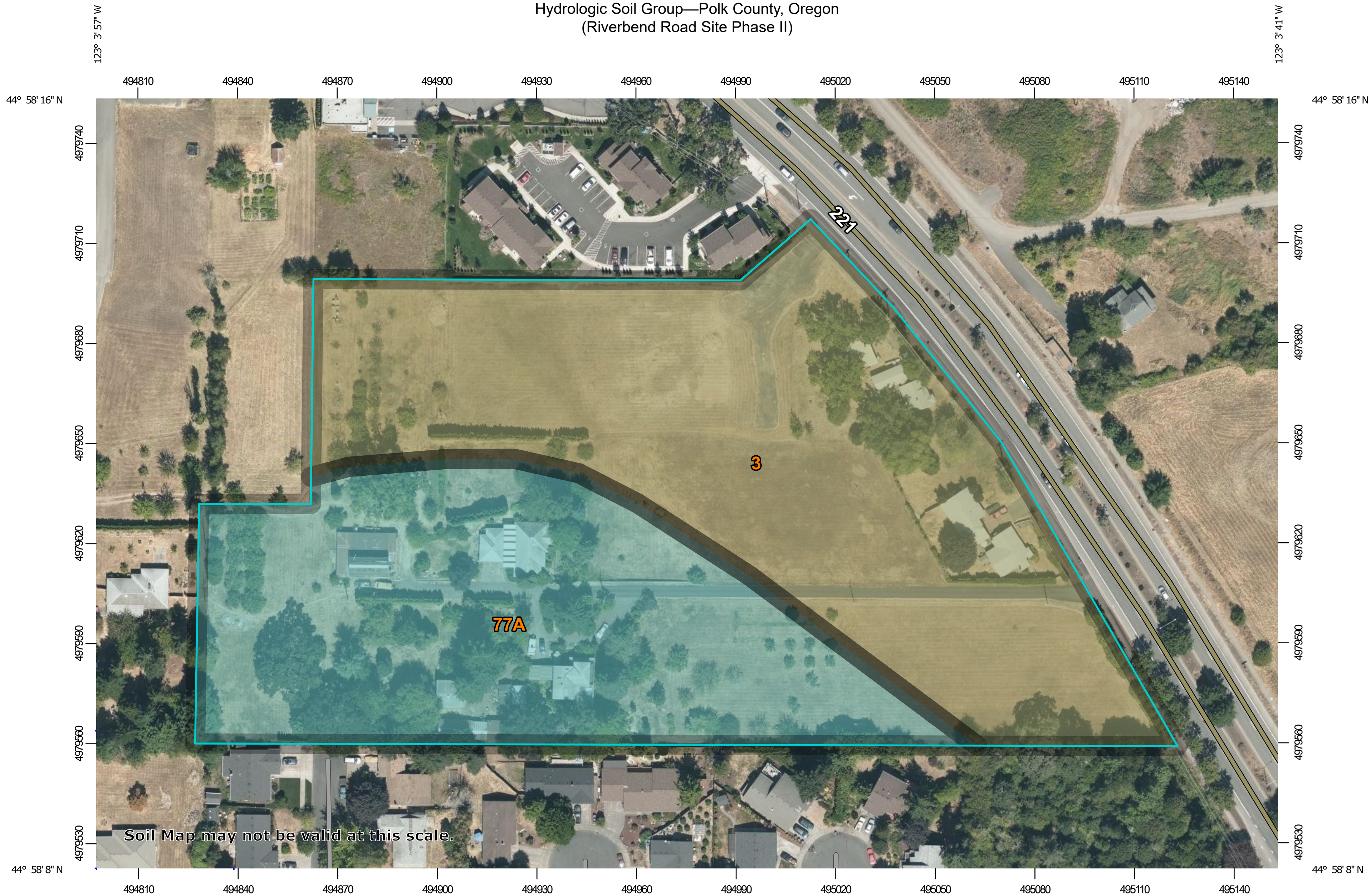
DEVELOPED STORMWATER BASIN MAP

DRAWING

JOB NUMBER
3048.0000.0

APPENDIX B

NRCS SOIL REPORT



Map Scale: 1:1,630 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 20, 40, 80, 120

Feet: 0, 50, 100, 200, 300

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

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 1:20,000.

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: Polk County, Oregon
Survey Area Data: Version 18, Jun 11, 2020

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

Date(s) aerial images were photographed: Aug 1, 2018—Aug 31, 2018

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
3	Amity silt loam	C/D	4.7	56.6%
77A	Woodburn silt loam, 0 to 3 percent slopes	C	3.6	43.4%
Totals for Area of Interest			**8.3**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

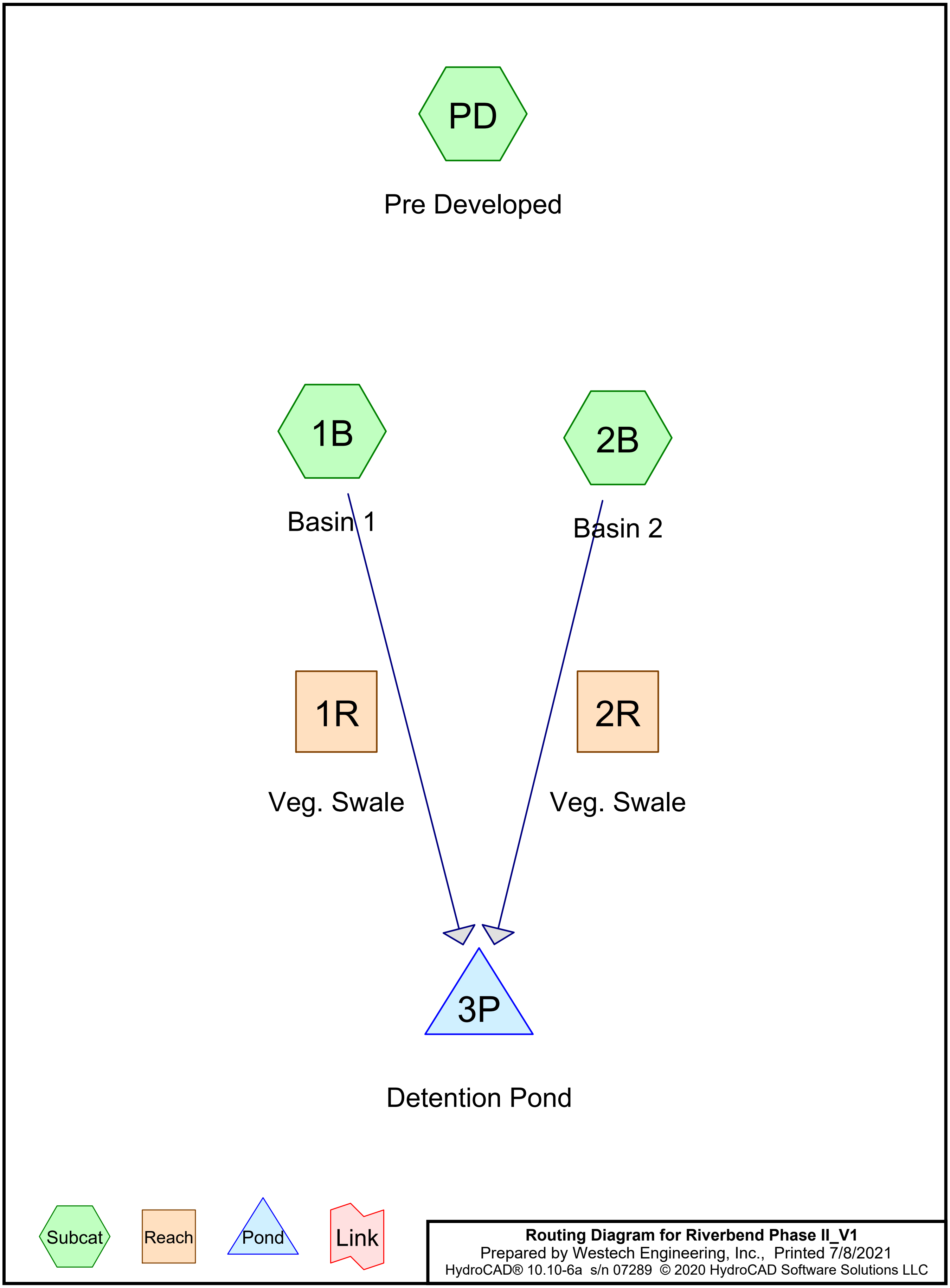
Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

APPENDIX C

HYDROCAD SUMMARIES



Routing Diagram for Riverbend Phase II_V1
Prepared by Westech Engineering, Inc., Printed 7/8/2021
HydroCAD® 10.10-6a s/n 07289 © 2020 HydroCAD Software Solutions LLC

Summary for Subcatchment PD: Pre Developed

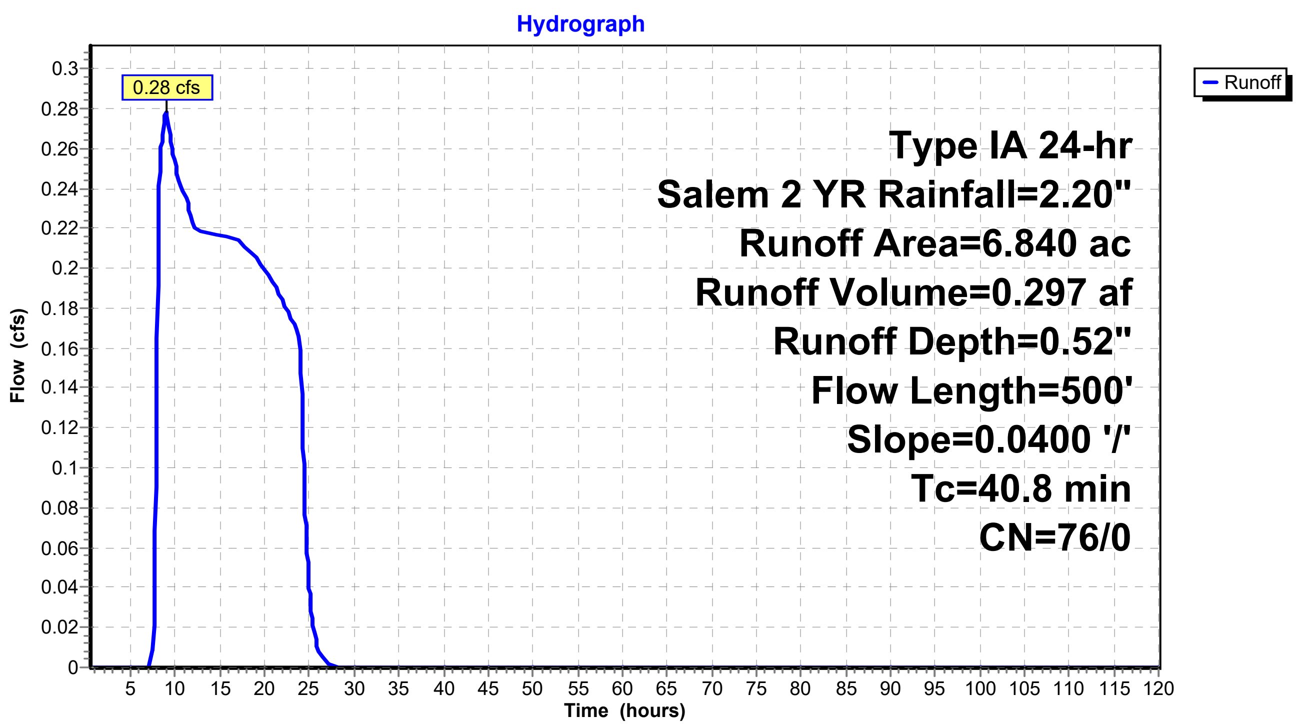
Runoff = 0.28 cfs @ 8.93 hrs, Volume= 0.297 af, Depth= 0.52"
Routed to nonexistent node 25L

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 2 YR Rainfall=2.20"

	Area (ac)	CN	Description
*	3.420	72	Salem Pre-Developed, HSG C
*	3.420	79	Salem Pre-Developed, HSG D
	6.840	76	Weighted Average
	6.840		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
37.5	300	0.0400	0.13		**Sheet Flow, Predeveloped** n= 0.300 P2= 2.20"
3.3	200	0.0400	1.00		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
40.8	500	Total			

Subcatchment PD: Pre Developed



Summary for Subcatchment PD: Pre Developed

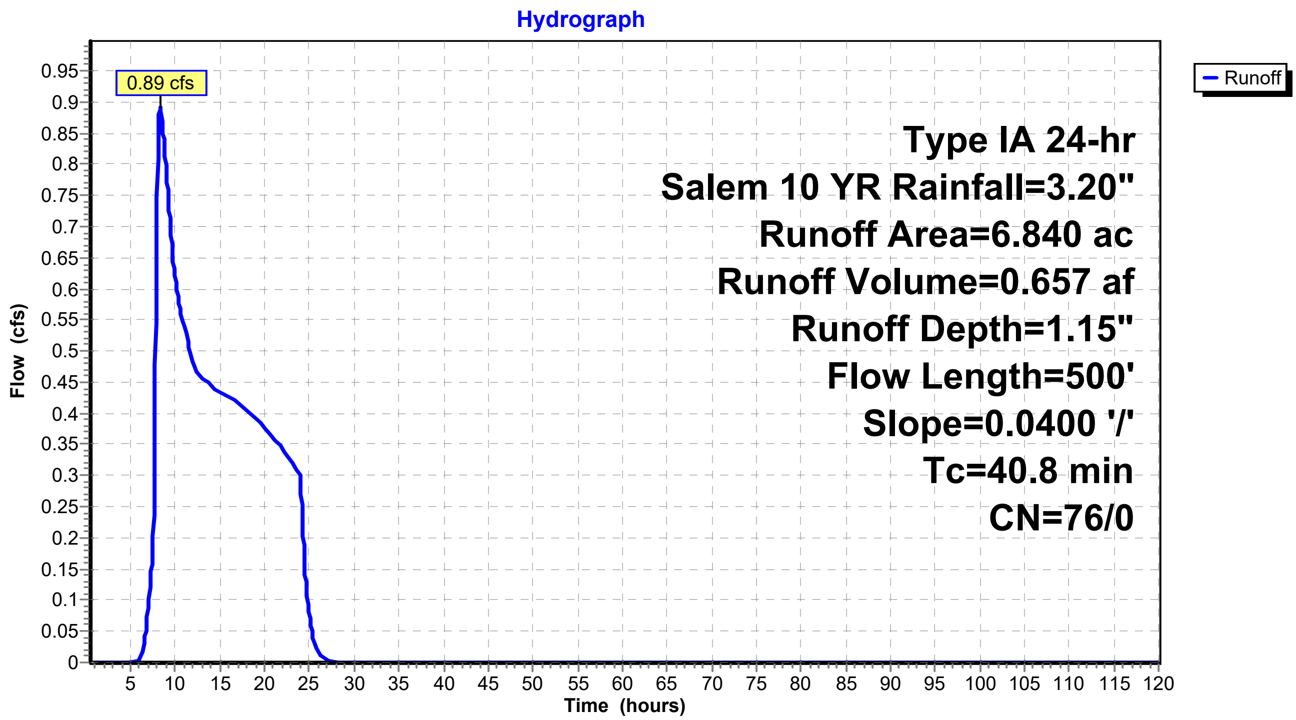
Runoff = 0.89 cfs @ 8.26 hrs, Volume= 0.657 af, Depth= 1.15"
Routed to nonexistent node 25L

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 10 YR Rainfall=3.20"

	Area (ac)	CN	Description
*	3.420	72	Salem Pre-Developed, HSG C
*	3.420	79	Salem Pre-Developed, HSG D
	6.840	76	Weighted Average
	6.840		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
37.5	300	0.0400	0.13		**Sheet Flow, Predeveloped** n= 0.300 P2= 2.20"
3.3	200	0.0400	1.00		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
40.8	500	Total			

Subcatchment PD: Pre Developed



Summary for Subcatchment PD: Pre Developed

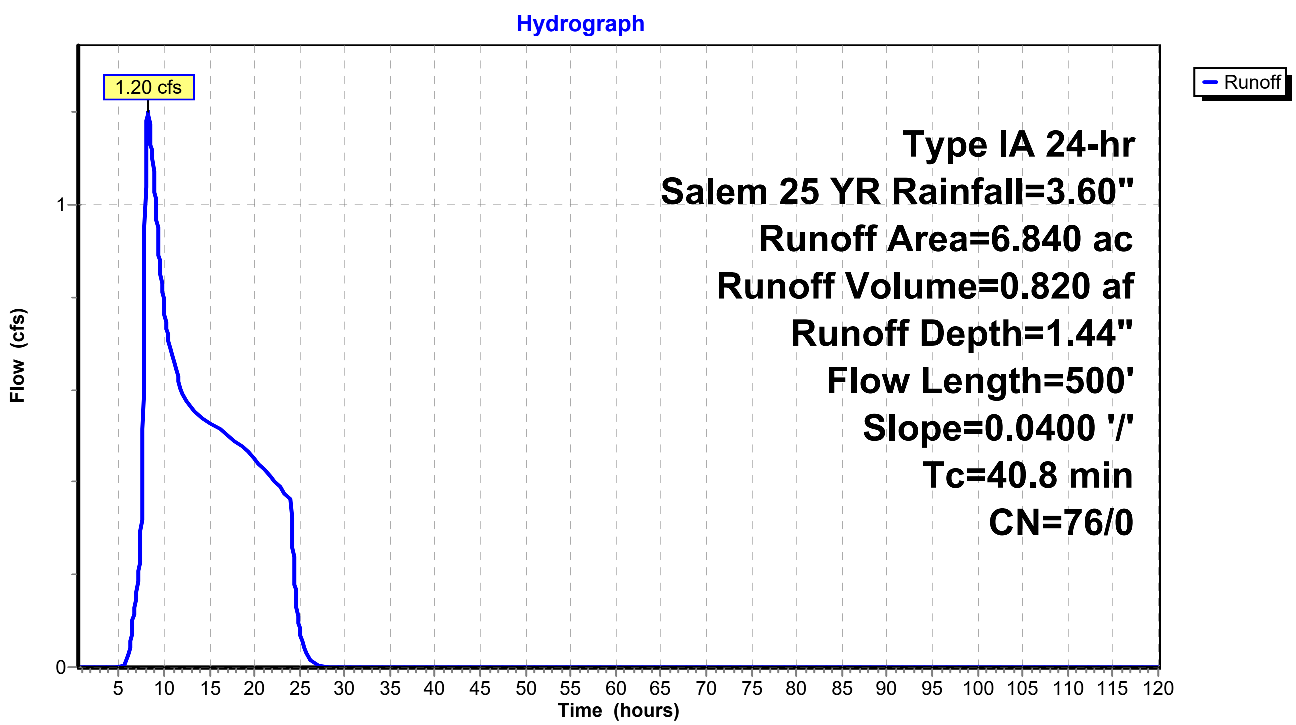
Runoff = 1.20 cfs @ 8.22 hrs, Volume= 0.820 af, Depth= 1.44"
Routed to nonexistent node 25L

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 25 YR Rainfall=3.60"

	Area (ac)	CN	Description
*	3.420	72	Salem Pre-Developed, HSG C
*	3.420	79	Salem Pre-Developed, HSG D
	6.840	76	Weighted Average
	6.840		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
37.5	300	0.0400	0.13		**Sheet Flow, Predeveloped** n= 0.300 P2= 2.20"
3.3	200	0.0400	1.00		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
40.8	500	Total			

Subcatchment PD: Pre Developed



Summary for Subcatchment PD: Pre Developed

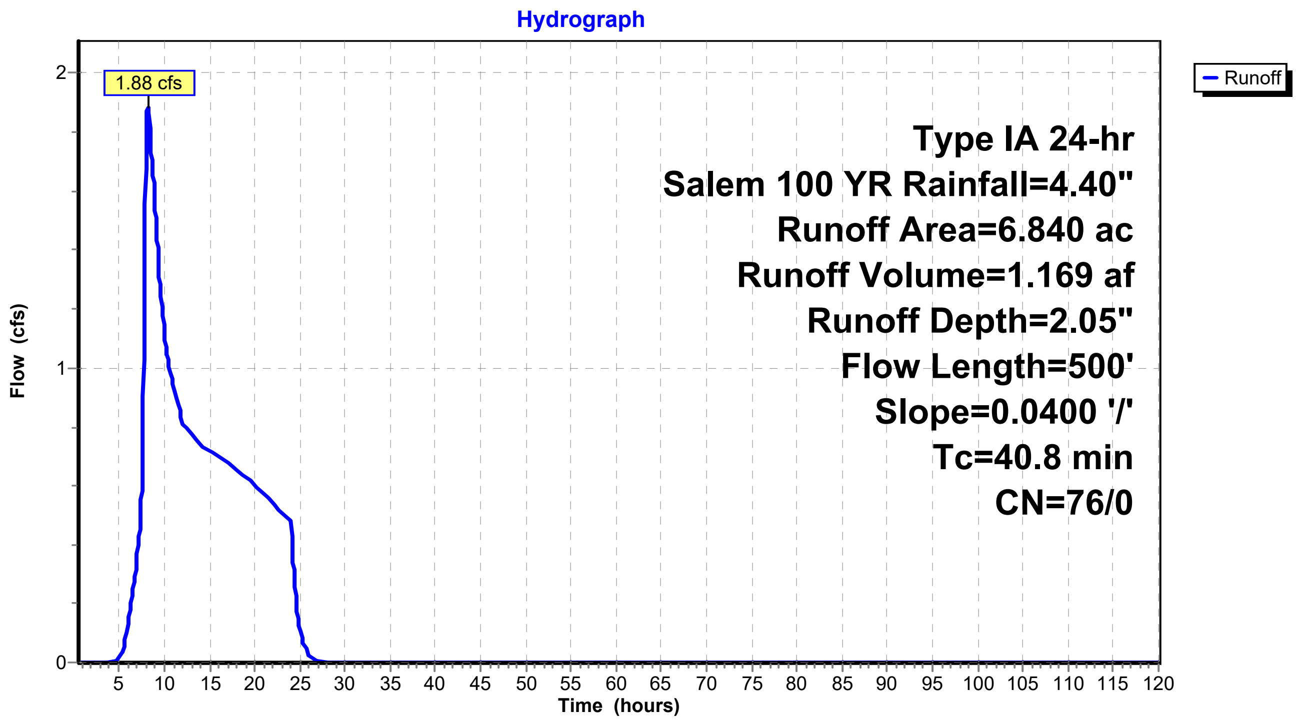
Runoff = 1.88 cfs @ 8.18 hrs, Volume= 1.169 af, Depth= 2.05"
Routed to nonexistent node 25L

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 100 YR Rainfall=4.40"

	Area (ac)	CN	Description
*	3.420	72	Salem Pre-Developed, HSG C
*	3.420	79	Salem Pre-Developed, HSG D
	6.840	76	Weighted Average
	6.840		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
37.5	300	0.0400	0.13		**Sheet Flow, Predeveloped** n= 0.300 P2= 2.20"
3.3	200	0.0400	1.00		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
40.8	500	Total			

Subcatchment PD: Pre Developed



Summary for Subcatchment 1B: Basin 1

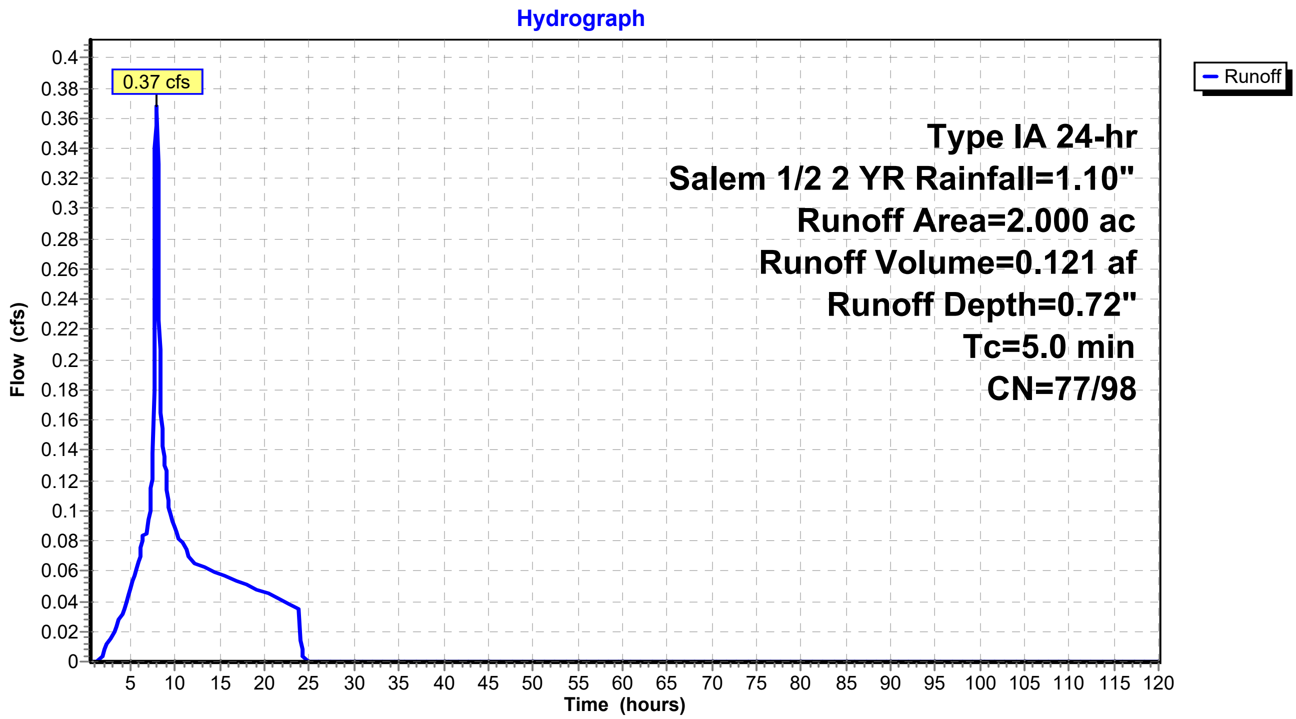
Runoff = 0.37 cfs @ 7.92 hrs, Volume= 0.121 af, Depth= 0.72"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 1/2 2 YR Rainfall=1.10"

	Area (ac)	CN	Description
*	1.600	98	Paved parking & roof, HSG D
*	0.200	74	Open Space, HSG C
*	0.200	80	Open Space, HSG D
	2.000	94	Weighted Average
	0.400		20.00% Pervious Area
	1.600		80.00% Impervious Area

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

Subcatchment 1B: Basin 1



Summary for Subcatchment 2B: Basin 2

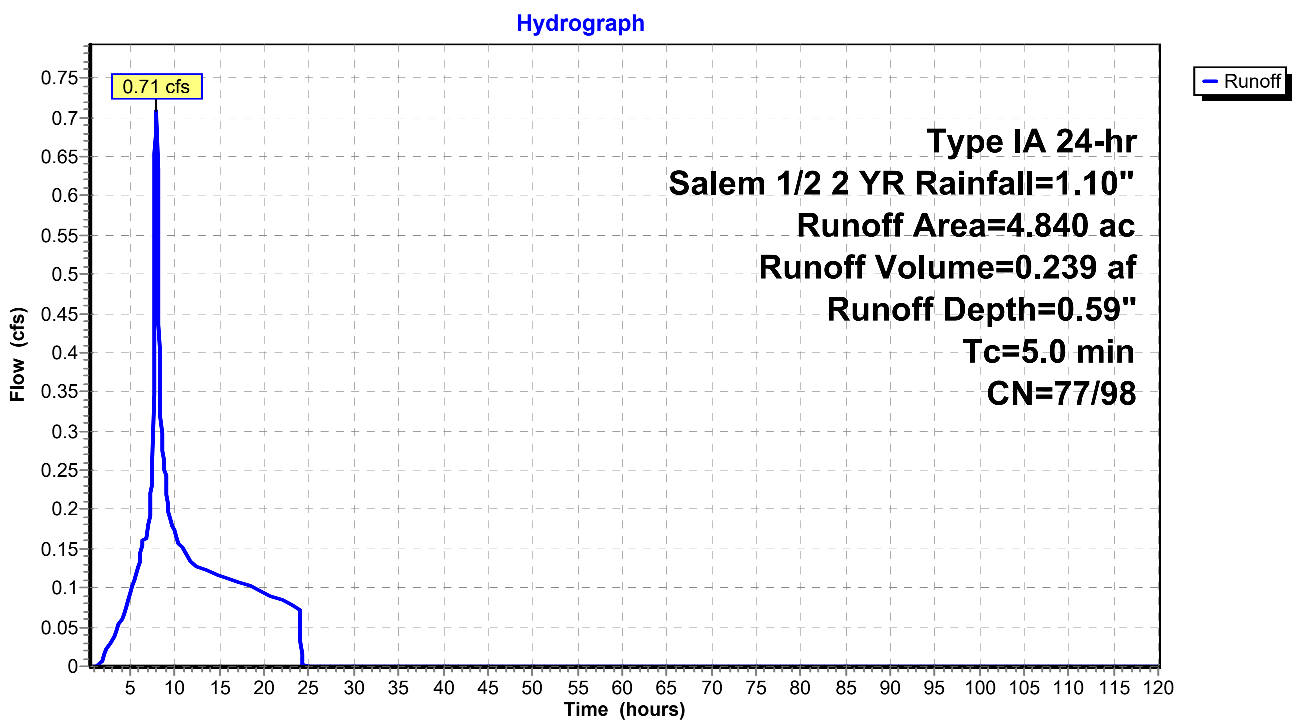
Runoff = 0.71 cfs @ 7.92 hrs, Volume= 0.239 af, Depth= 0.59"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 1/2 2 YR Rainfall=1.10"

	Area (ac)	CN	Description
*	3.080	98	Paved parking & roof, HSG D
*	0.880	74	Open Space, HSG C
*	0.880	80	Open Space, HSG D
	4.840	90	Weighted Average
	1.760		36.36% Pervious Area
	3.080		63.64% Impervious Area

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

Subcatchment 2B: Basin 2



Summary for Pond 3P: Detention Basin

Inflow Area = 6.840 ac, 68.42% Impervious, Inflow Depth = 0.63" for Salem 1/2 2 YR event
Inflow = 1.08 cfs @ 7.92 hrs, Volume= 0.359 af
Outflow = 0.14 cfs @ 19.77 hrs, Volume= 0.359 af, Atten= 87%, Lag= 711.1 min
Primary = 0.14 cfs @ 19.77 hrs, Volume= 0.359 af
Routed to nonexistent node 23L

Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Peak Elev= 146.42' @ 19.77 hrs Surf.Area= 7,403 sf Storage= 6,590 cf

Plug-Flow detention time= 567.8 min calculated for 0.359 af (100% of inflow)
Center-of-Mass det. time= 568.0 min (1,289.8 - 721.8)

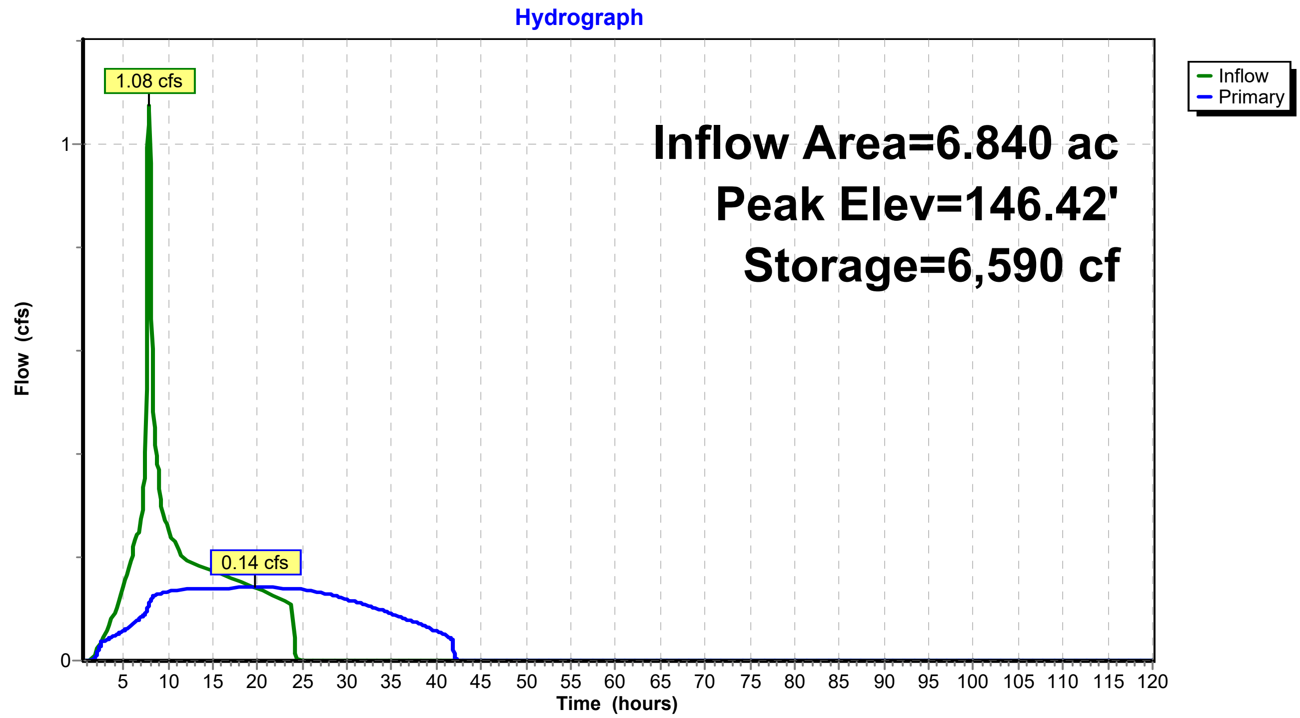
Volume	Invert	Avail.Storage	Storage Description
#1	144.30'	72,067 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
144.30	0	0	0
144.50	2,120	212	212
145.00	2,560	1,170	1,382
146.00	3,360	2,960	4,342
147.00	13,040	8,200	12,542
148.00	19,230	16,135	28,677
149.00	21,930	20,580	49,257
150.00	23,690	22,810	72,067

Device	Routing	Invert	Outlet Devices
#1	Primary	144.20'	**1.9" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#2	Primary	146.50'	**5.0" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#3	Primary	147.60'	**8.0" W x 3.0" H Vert. Weir Cut** C= 0.600 Limited to weir flow at low heads
#4	Primary	147.80'	**12.0" Horiz. O/F Riser** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.14 cfs @ 19.77 hrs HW=146.42' (Free Discharge)
—**1=Orifice** (Orifice Controls 0.14 cfs @ 7.17 fps)
—**2=Orifice** (Controls 0.00 cfs)
—**3=Weir Cut** (Controls 0.00 cfs)
—**4=O/F Riser** (Controls 0.00 cfs)

Pond 3P: Detention Basin



Summary for Subcatchment 1B: Basin 1

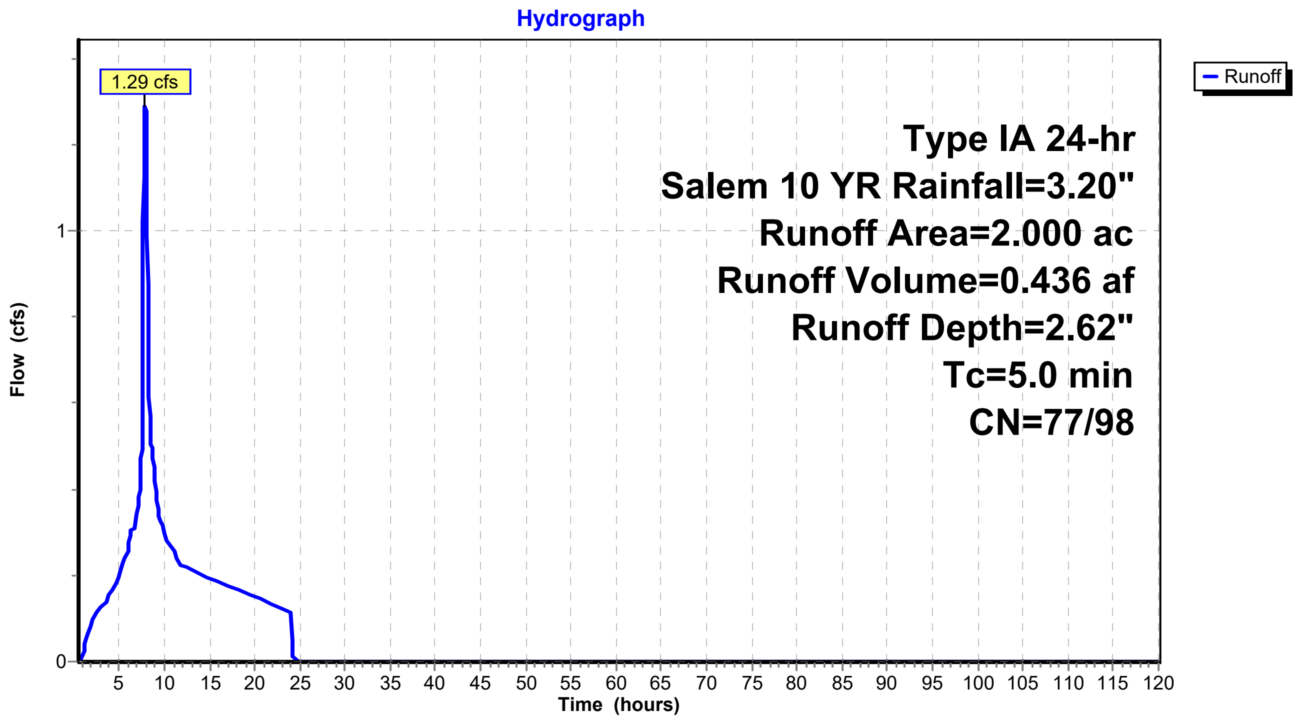
Runoff = 1.29 cfs @ 7.91 hrs, Volume= 0.436 af, Depth= 2.62"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 10 YR Rainfall=3.20"

	Area (ac)	CN	Description
*	1.600	98	Paved parking & roof, HSG D
*	0.200	74	Open Space, HSG C
*	0.200	80	Open Space, HSG D
	2.000	94	Weighted Average
	0.400		20.00% Pervious Area
	1.600		80.00% Impervious Area

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

Subcatchment 1B: Basin 1



Summary for Subcatchment 2B: Basin 2

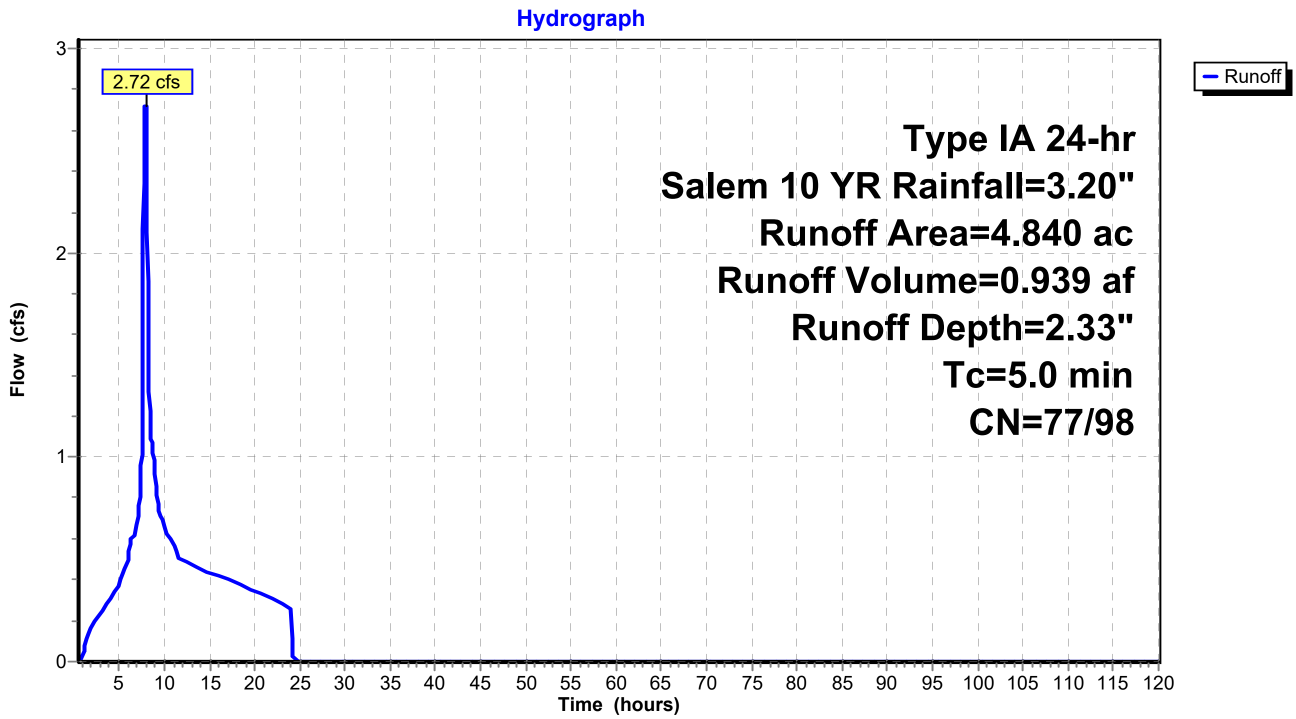
Runoff = 2.72 cfs @ 7.92 hrs, Volume= 0.939 af, Depth= 2.33"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 10 YR Rainfall=3.20"

	Area (ac)	CN	Description
*	3.080	98	Paved parking & roof, HSG D
*	0.880	74	Open Space, HSG C
*	0.880	80	Open Space, HSG D
	4.840	90	Weighted Average
	1.760		36.36% Pervious Area
	3.080		63.64% Impervious Area

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

Subcatchment 2B: Basin 2



Summary for Pond 3P: Detention Basin

Inflow Area = 6.840 ac, 68.42% Impervious, Inflow Depth = 2.41" for Salem 10 YR event
Inflow = 4.01 cfs @ 7.92 hrs, Volume= 1.375 af
Outflow = 0.84 cfs @ 11.00 hrs, Volume= 1.375 af, Atten= 79%, Lag= 185.0 min
Primary = 0.84 cfs @ 11.00 hrs, Volume= 1.375 af
Routed to nonexistent node 23L

Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Peak Elev= 147.52' @ 11.00 hrs Surf.Area= 16,290 sf Storage= 20,241 cf

Plug-Flow detention time= 393.8 min calculated for 1.375 af (100% of inflow)
Center-of-Mass det. time= 393.8 min (1,088.2 - 694.4)

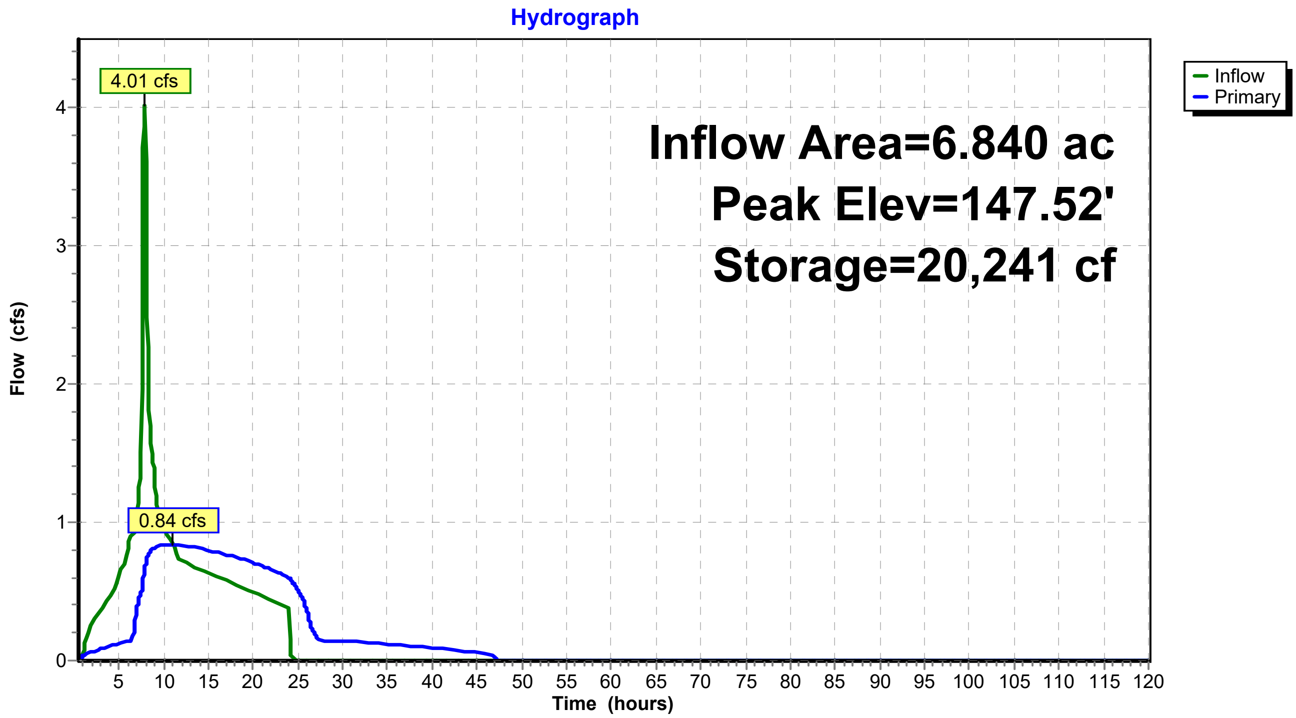
Volume	Invert	Avail.Storage	Storage Description
#1	144.30'	72,067 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
144.30	0	0	0
144.50	2,120	212	212
145.00	2,560	1,170	1,382
146.00	3,360	2,960	4,342
147.00	13,040	8,200	12,542
148.00	19,230	16,135	28,677
149.00	21,930	20,580	49,257
150.00	23,690	22,810	72,067

Device	Routing	Invert	Outlet Devices
#1	Primary	144.20'	**1.9" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#2	Primary	146.50'	**5.0" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#3	Primary	147.60'	**8.0" W x 3.0" H Vert. Weir Cut** C= 0.600 Limited to weir flow at low heads
#4	Primary	147.80'	**12.0" Horiz. O/F Riser** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.84 cfs @ 11.00 hrs HW=147.52' (Free Discharge)
├─**1=Orifice** (Orifice Controls 0.17 cfs @ 8.78 fps)
├─**2=Orifice** (Orifice Controls 0.66 cfs @ 4.87 fps)
├─**3=Weir Cut** (Controls 0.00 cfs)
└─**4=O/F Riser** (Controls 0.00 cfs)

Pond 3P: Detention Basin



Summary for Subcatchment 1B: Basin 1

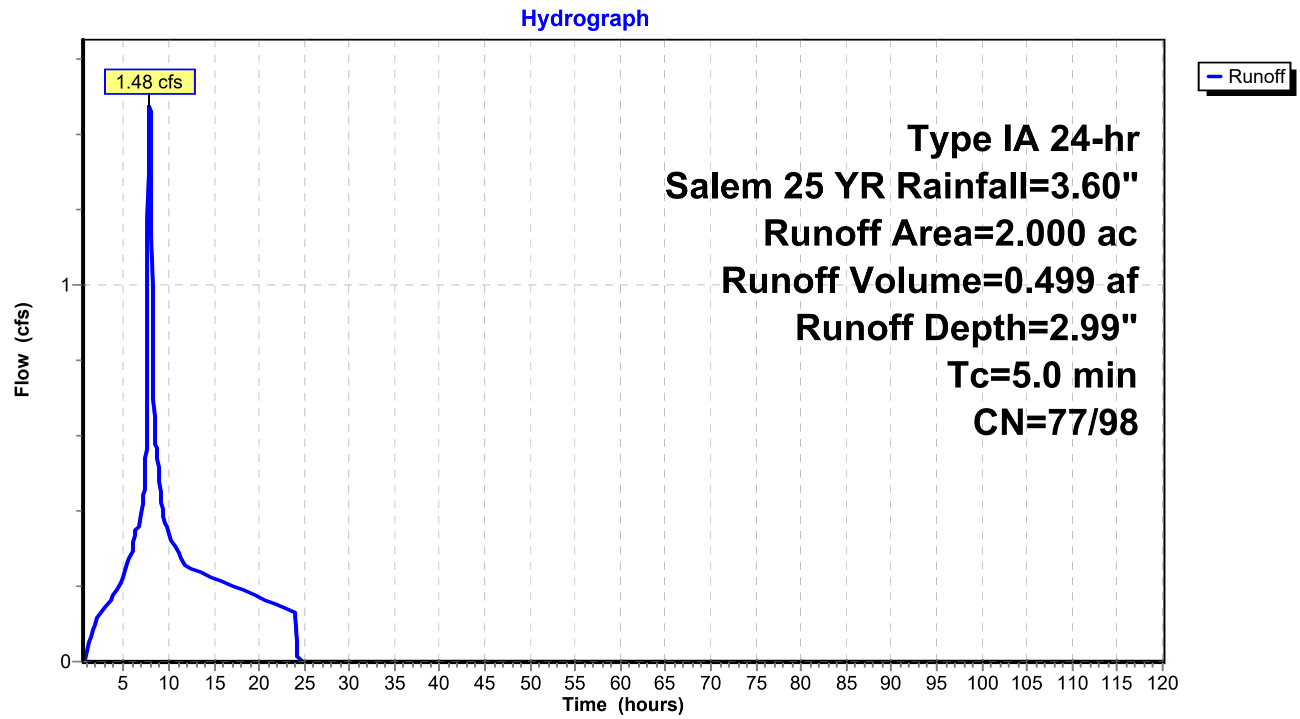
Runoff = 1.48 cfs @ 7.91 hrs, Volume= 0.499 af, Depth= 2.99"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 25 YR Rainfall=3.60"

	Area (ac)	CN	Description
*	1.600	98	Paved parking & roof, HSG D
*	0.200	74	Open Space, HSG C
*	0.200	80	Open Space, HSG D
	2.000	94	Weighted Average
	0.400		20.00% Pervious Area
	1.600		80.00% Impervious Area

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

Subcatchment 1B: Basin 1



Summary for Subcatchment 2B: Basin 2

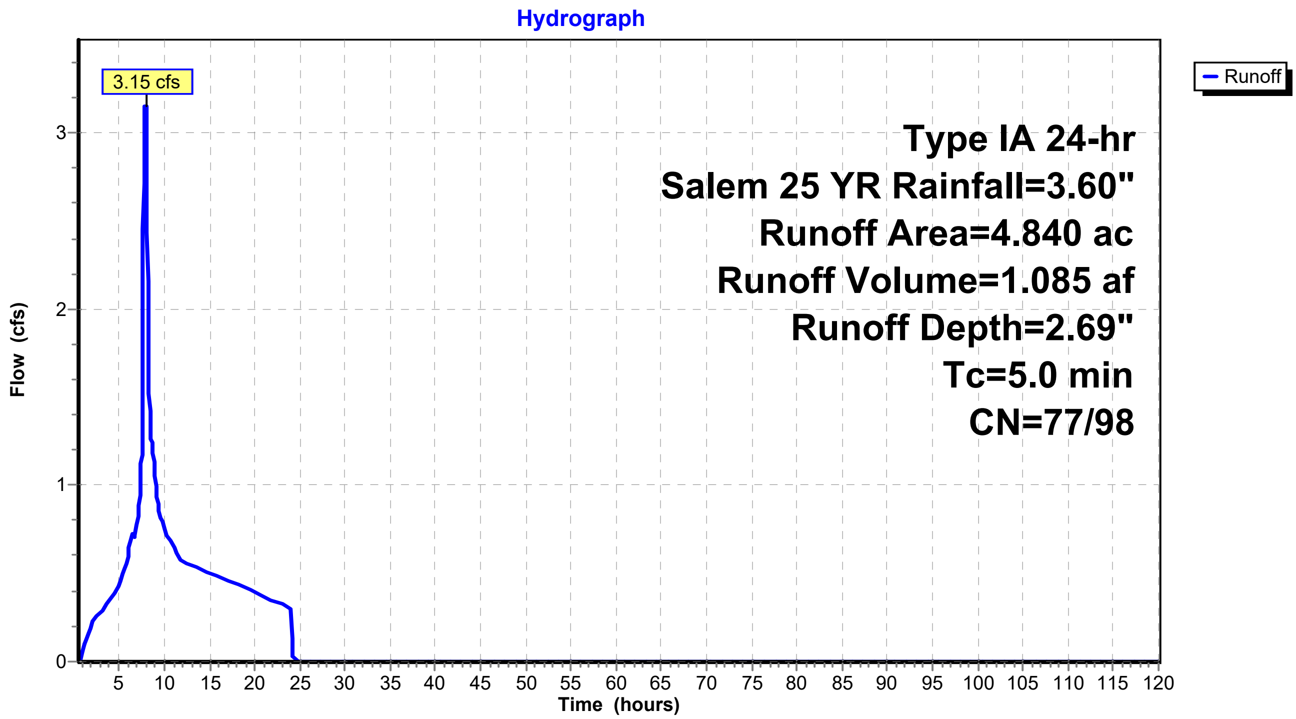
Runoff = 3.15 cfs @ 7.92 hrs, Volume= 1.085 af, Depth= 2.69"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 25 YR Rainfall=3.60"

	Area (ac)	CN	Description
*	3.080	98	Paved parking & roof, HSG D
*	0.880	74	Open Space, HSG C
*	0.880	80	Open Space, HSG D
	4.840	90	Weighted Average
	1.760		36.36% Pervious Area
	3.080		63.64% Impervious Area

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

Subcatchment 2B: Basin 2



Summary for Pond 3P: Detention Basin

Inflow Area = 6.840 ac, 68.42% Impervious, Inflow Depth = 2.78" for Salem 25 YR event
Inflow = 4.63 cfs @ 7.92 hrs, Volume= 1.584 af
Outflow = 0.98 cfs @ 10.86 hrs, Volume= 1.584 af, Atten= 79%, Lag= 176.6 min
Primary = 0.98 cfs @ 10.86 hrs, Volume= 1.584 af
Routed to nonexistent node 23L

Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Peak Elev= 147.71' @ 10.86 hrs Surf.Area= 17,452 sf Storage= 23,409 cf

Plug-Flow detention time= 403.3 min calculated for 1.584 af (100% of inflow)
Center-of-Mass det. time= 403.3 min (1,095.0 - 691.6)

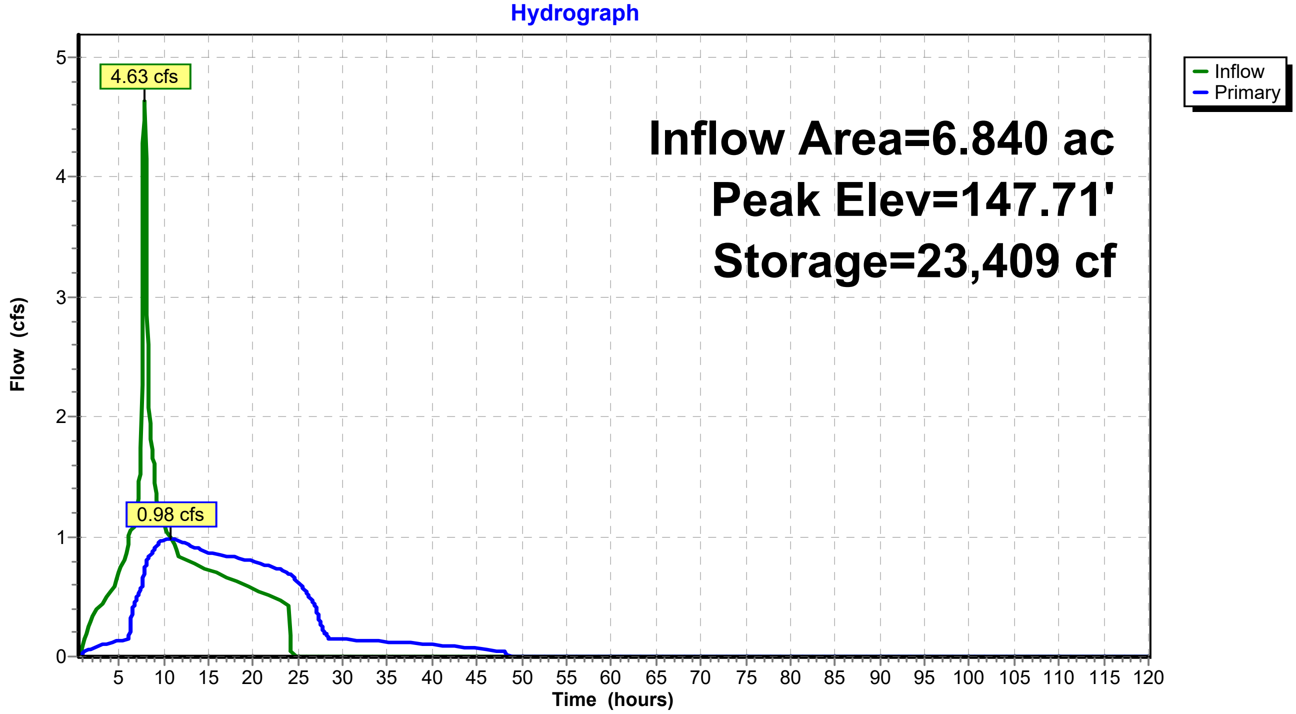
Volume	Invert	Avail.Storage	Storage Description
#1	144.30'	72,067 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
144.30	0	0	0
144.50	2,120	212	212
145.00	2,560	1,170	1,382
146.00	3,360	2,960	4,342
147.00	13,040	8,200	12,542
148.00	19,230	16,135	28,677
149.00	21,930	20,580	49,257
150.00	23,690	22,810	72,067

Device	Routing	Invert	Outlet Devices
#1	Primary	144.20'	**1.9" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#2	Primary	146.50'	**5.0" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#3	Primary	147.60'	**8.0" W x 3.0" H Vert. Weir Cut** C= 0.600 Limited to weir flow at low heads
#4	Primary	147.80'	**12.0" Horiz. O/F Riser** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.98 cfs @ 10.86 hrs HW=147.71' (Free Discharge)
—**1=Orifice** (Orifice Controls 0.18 cfs @ 9.02 fps)
—**2=Orifice** (Orifice Controls 0.72 cfs @ 5.30 fps)
—**3=Weir Cut** (Orifice Controls 0.08 cfs @ 1.08 fps)
—**4=O/F Riser** (Controls 0.00 cfs)

Pond 3P: Detention Basin



Summary for Subcatchment 1B: Basin 1

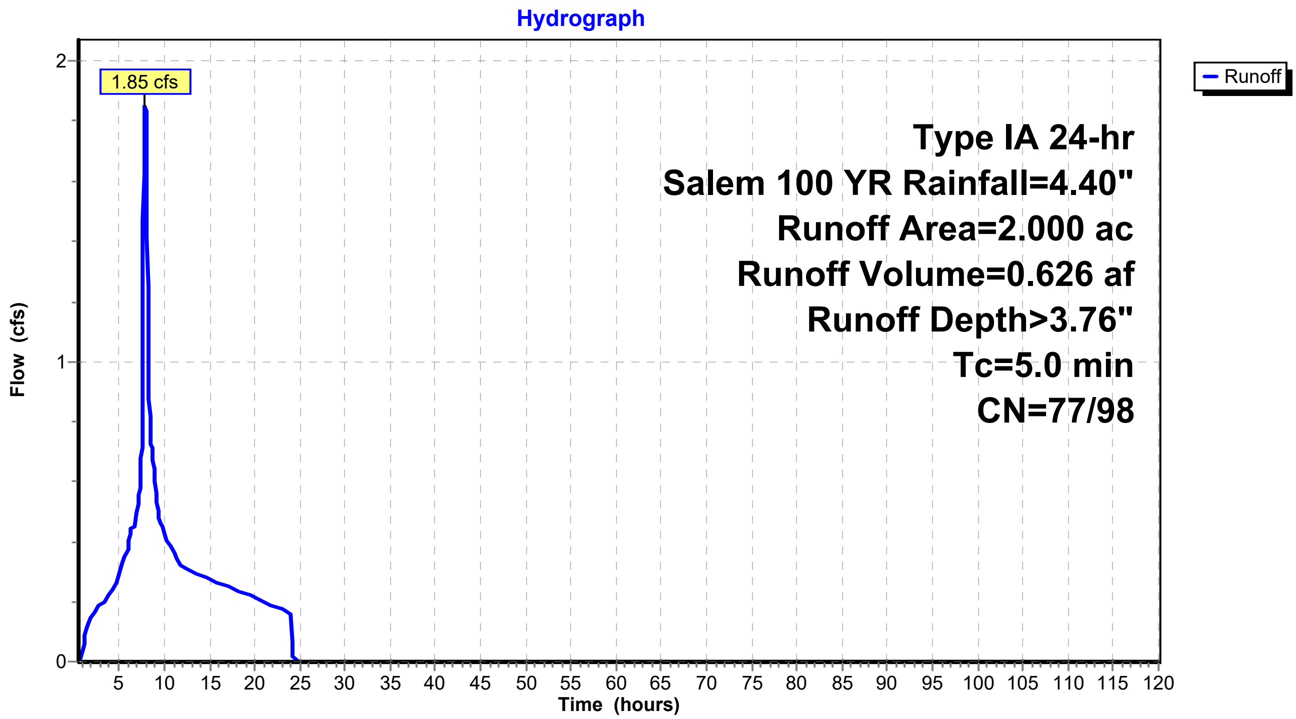
Runoff = 1.85 cfs @ 7.91 hrs, Volume= 0.626 af, Depth> 3.76"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 100 YR Rainfall=4.40"

	Area (ac)	CN	Description
*	1.600	98	Paved parking & roof, HSG D
*	0.200	74	Open Space, HSG C
*	0.200	80	Open Space, HSG D
	2.000	94	Weighted Average
	0.400		20.00% Pervious Area
	1.600		80.00% Impervious Area

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

Subcatchment 1B: Basin 1



Summary for Subcatchment 2B: Basin 2

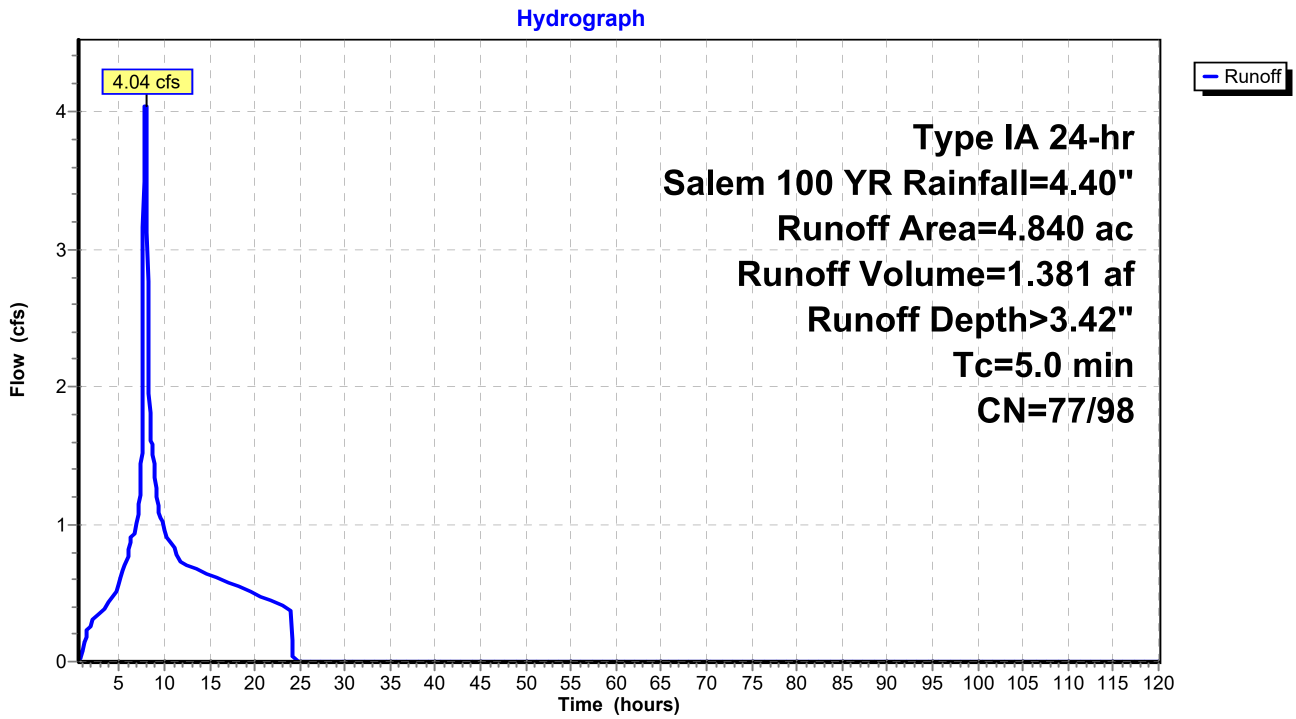
Runoff = 4.04 cfs @ 7.92 hrs, Volume= 1.381 af, Depth> 3.42"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem 100 YR Rainfall=4.40"

	Area (ac)	CN	Description
*	3.080	98	Paved parking & roof, HSG D
*	0.880	74	Open Space, HSG C
*	0.880	80	Open Space, HSG D
	4.840	90	Weighted Average
	1.760		36.36% Pervious Area
	3.080		63.64% Impervious Area

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

Subcatchment 2B: Basin 2



Summary for Pond 3P: Detention Basin

Inflow Area = 6.840 ac, 68.42% Impervious, Inflow Depth > 3.52" for Salem 100 YR event
Inflow = 5.89 cfs @ 7.92 hrs, Volume= 2.007 af
Outflow = 1.76 cfs @ 9.14 hrs, Volume= 2.007 af, Atten= 70%, Lag= 73.2 min
Primary = 1.76 cfs @ 9.14 hrs, Volume= 2.007 af
Routed to nonexistent node 23L

Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Peak Elev= 147.92' @ 9.14 hrs Surf.Area= 18,749 sf Storage= 27,201 cf

Plug-Flow detention time= 376.5 min calculated for 2.007 af (100% of inflow)
Center-of-Mass det. time= 376.4 min (1,063.3 - 686.9)

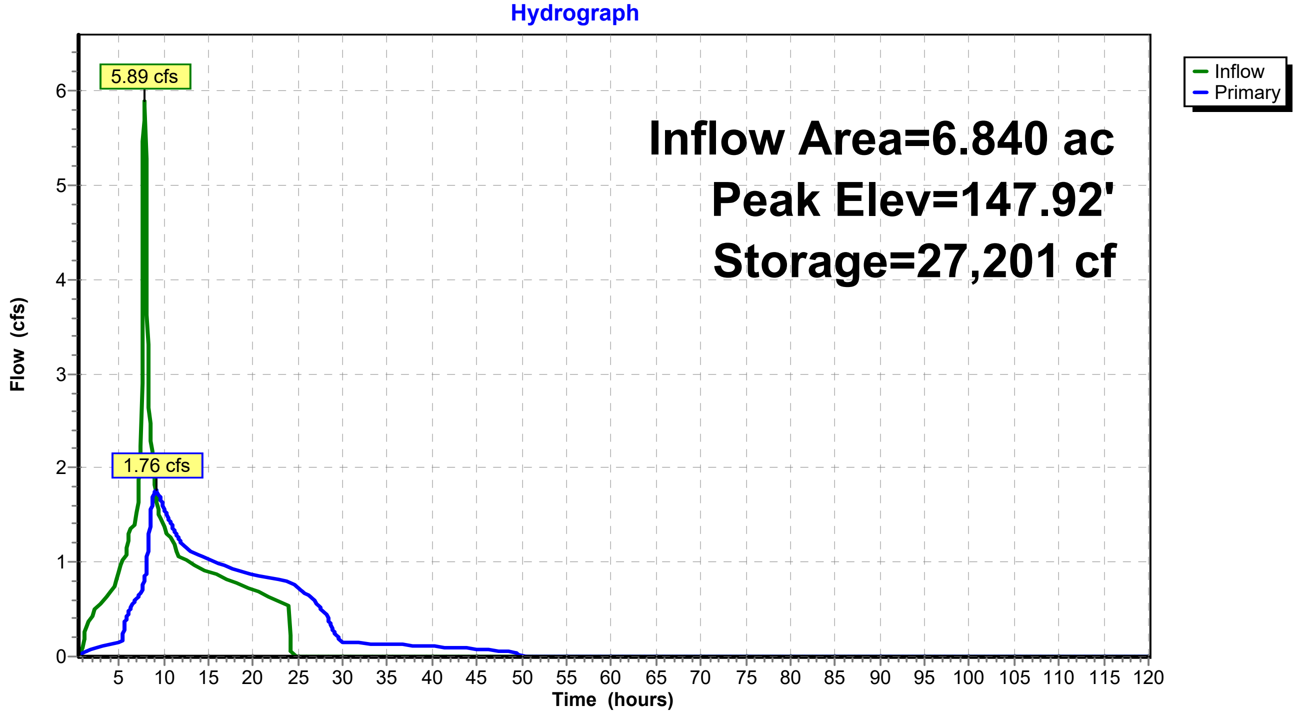
Volume	Invert	Avail.Storage	Storage Description
#1	144.30'	72,067 cf	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
144.30	0	0	0
144.50	2,120	212	212
145.00	2,560	1,170	1,382
146.00	3,360	2,960	4,342
147.00	13,040	8,200	12,542
148.00	19,230	16,135	28,677
149.00	21,930	20,580	49,257
150.00	23,690	22,810	72,067

Device	Routing	Invert	Outlet Devices
#1	Primary	144.20'	**1.9" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#2	Primary	146.50'	**5.0" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#3	Primary	147.60'	**8.0" W x 3.0" H Vert. Weir Cut** C= 0.600 Limited to weir flow at low heads
#4	Primary	147.80'	**12.0" Horiz. O/F Riser** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=1.75 cfs @ 9.14 hrs HW=147.92' (Free Discharge)
1=Orifice (Orifice Controls 0.18 cfs @ 9.29 fps)
2=Orifice (Orifice Controls 0.78 cfs @ 5.74 fps)
3=Weir Cut (Orifice Controls 0.35 cfs @ 2.10 fps)
4=O/F Riser (Weir Controls 0.44 cfs @ 1.14 fps)

Pond 3P: Detention Basin



Summary for Subcatchment 1B: Basin 1

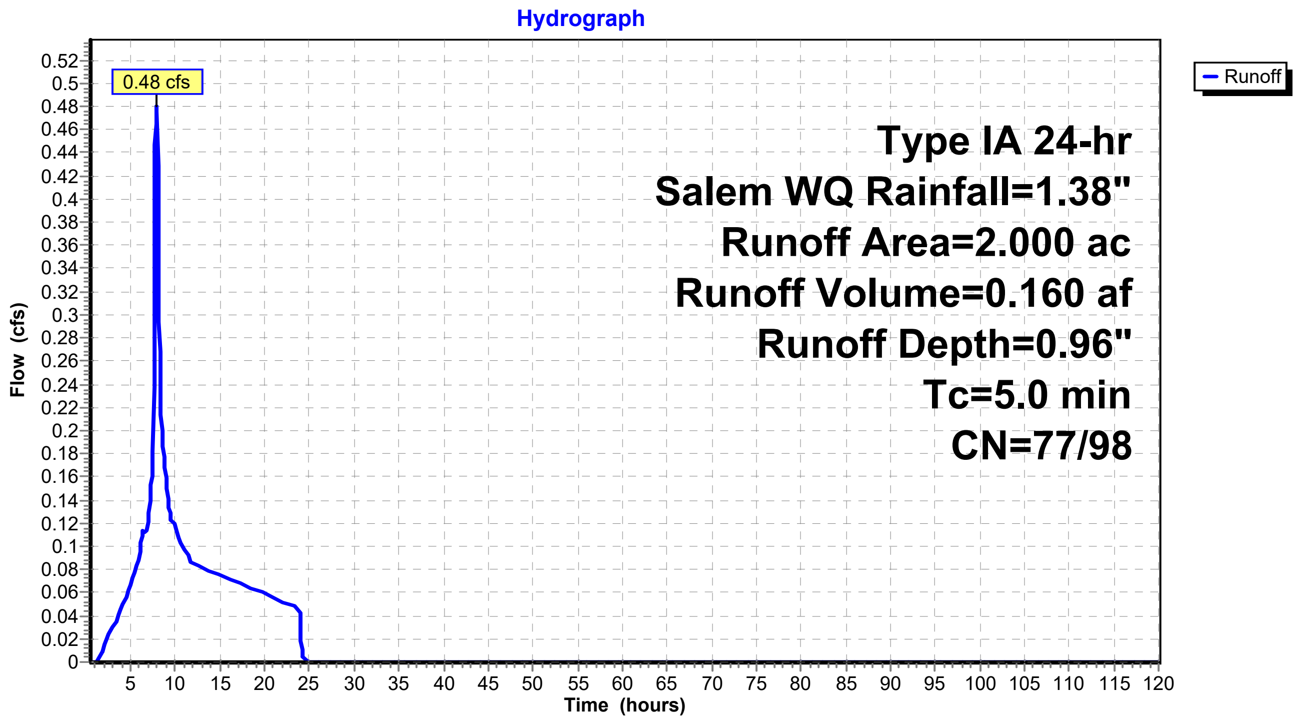
Runoff = 0.48 cfs @ 7.91 hrs, Volume= 0.160 af, Depth= 0.96"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem WQ Rainfall=1.38"

	Area (ac)	CN	Description
*	1.600	98	Paved parking & roof, HSG D
*	0.200	74	Open Space, HSG C
*	0.200	80	Open Space, HSG D
	2.000	94	Weighted Average
	0.400		20.00% Pervious Area
	1.600		80.00% Impervious Area

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

Subcatchment 1B: Basin 1



Summary for Subcatchment 2B: Basin 2

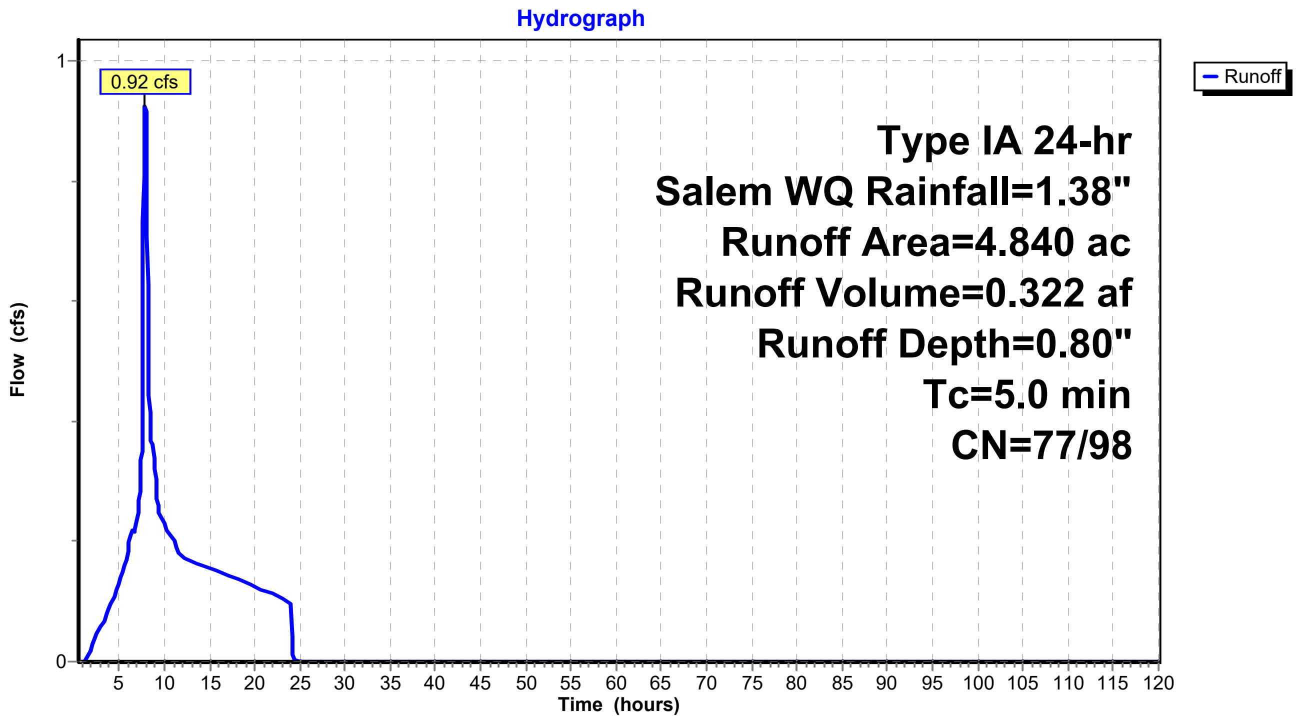
Runoff = 0.92 cfs @ 7.91 hrs, Volume= 0.322 af, Depth= 0.80"
Routed to Pond 3P : Detention Basin

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Type IA 24-hr Salem WQ Rainfall=1.38"

	Area (ac)	CN	Description
*	3.080	98	Paved parking & roof, HSG D
*	0.880	74	Open Space, HSG C
*	0.880	80	Open Space, HSG D
	4.840	90	Weighted Average
	1.760		36.36% Pervious Area
	3.080		63.64% Impervious Area

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

Subcatchment 2B: Basin 2



Summary for Pond 3P: Detention Basin

Inflow Area = 6.840 ac, 68.42% Impervious, Inflow Depth = 0.85" for Salem WQ event
Inflow = 1.40 cfs @ 7.91 hrs, Volume= 0.482 af
Outflow = 0.26 cfs @ 12.62 hrs, Volume= 0.482 af, Atten= 82%, Lag= 282.7 min
Primary = 0.26 cfs @ 12.62 hrs, Volume= 0.482 af
Routed to nonexistent node 23L

Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Peak Elev= 146.59' @ 12.62 hrs Surf.Area= 9,034 sf Storage= 7,975 cf

Plug-Flow detention time= 542.3 min calculated for 0.482 af (100% of inflow)
Center-of-Mass det. time= 542.6 min (1,258.1 - 715.5)

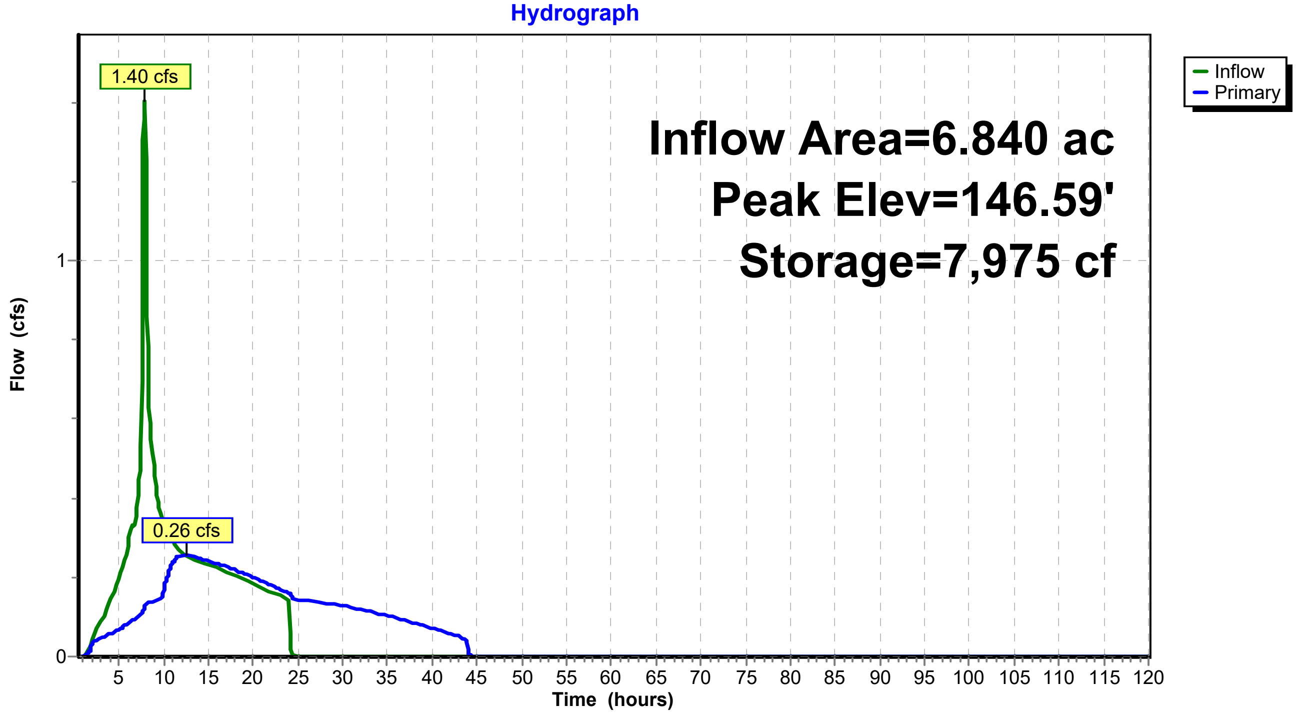
Volume	Invert	Avail.Storage	Storage Description
#1	144.30'	72,067 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

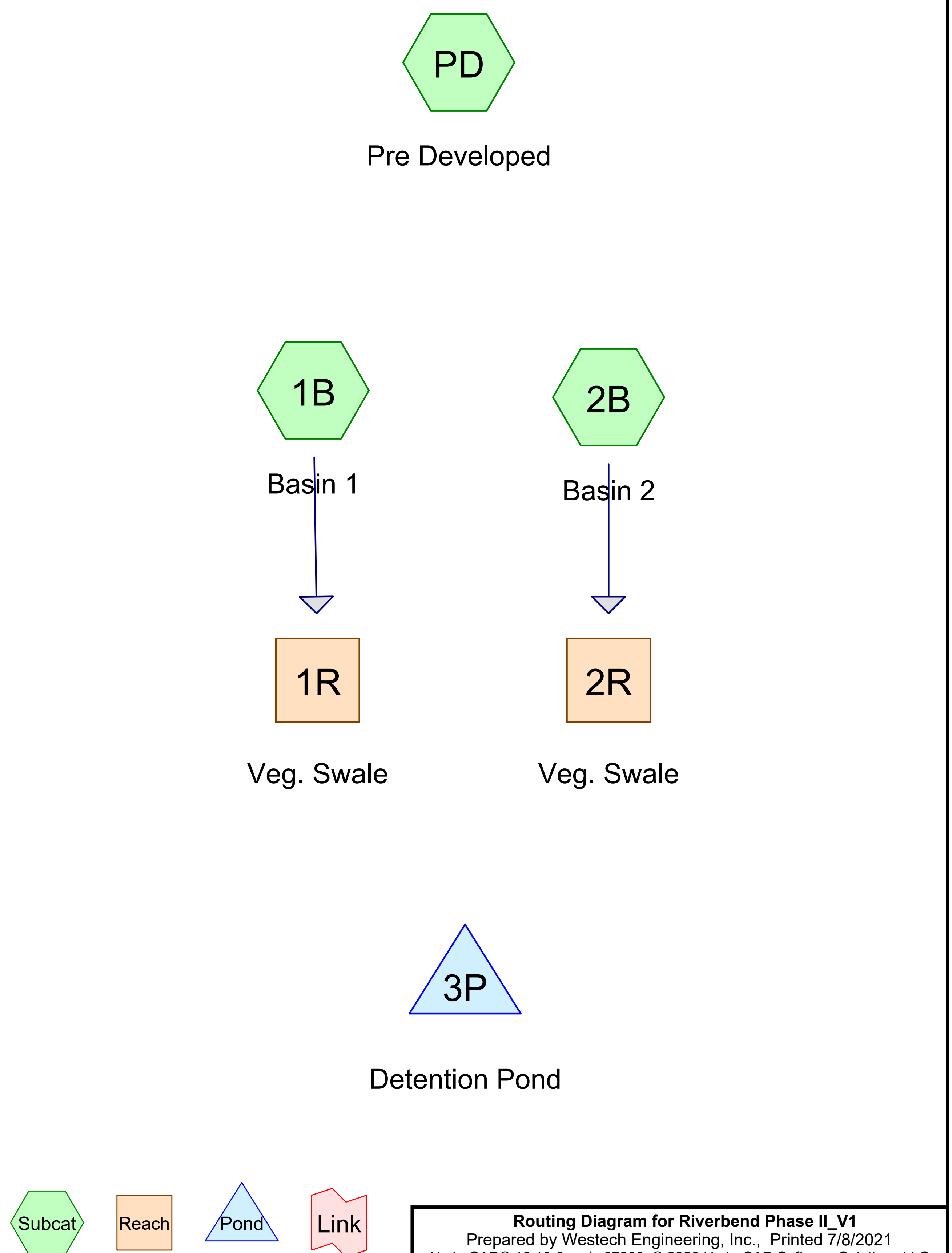
Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
144.30	0	0	0
144.50	2,120	212	212
145.00	2,560	1,170	1,382
146.00	3,360	2,960	4,342
147.00	13,040	8,200	12,542
148.00	19,230	16,135	28,677
149.00	21,930	20,580	49,257
150.00	23,690	22,810	72,067

Device	Routing	Invert	Outlet Devices
#1	Primary	144.20'	**1.9" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#2	Primary	146.50'	**5.0" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#3	Primary	147.60'	**8.0" W x 3.0" H Vert. Weir Cut** C= 0.600 Limited to weir flow at low heads
#4	Primary	147.80'	**12.0" Horiz. O/F Riser** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.25 cfs @ 12.62 hrs HW=146.59' (Free Discharge)
- **1=Orifice** (Orifice Controls 0.15 cfs @ 7.44 fps)
- **2=Orifice** (Weir Controls 0.11 cfs @ 0.96 fps)
- **3=Weir Cut** (Controls 0.00 cfs)
- **4=O/F Riser** (Controls 0.00 cfs)

Pond 3P: Detention Basin





Routing Diagram for Riverbend Phase II_V1
Prepared by Westech Engineering, Inc., Printed 7/8/2021
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Summary for Reach 1R: Veg. Swale

Inflow Area = 2.000 ac, 80.00% Impervious, Inflow Depth = 0.96" for Salem WQ event
Inflow = 0.48 cfs @ 7.91 hrs, Volume= 0.160 af
Outflow = 0.46 cfs @ 8.00 hrs, Volume= 0.160 af, Atten= 3%, Lag= 5.6 min

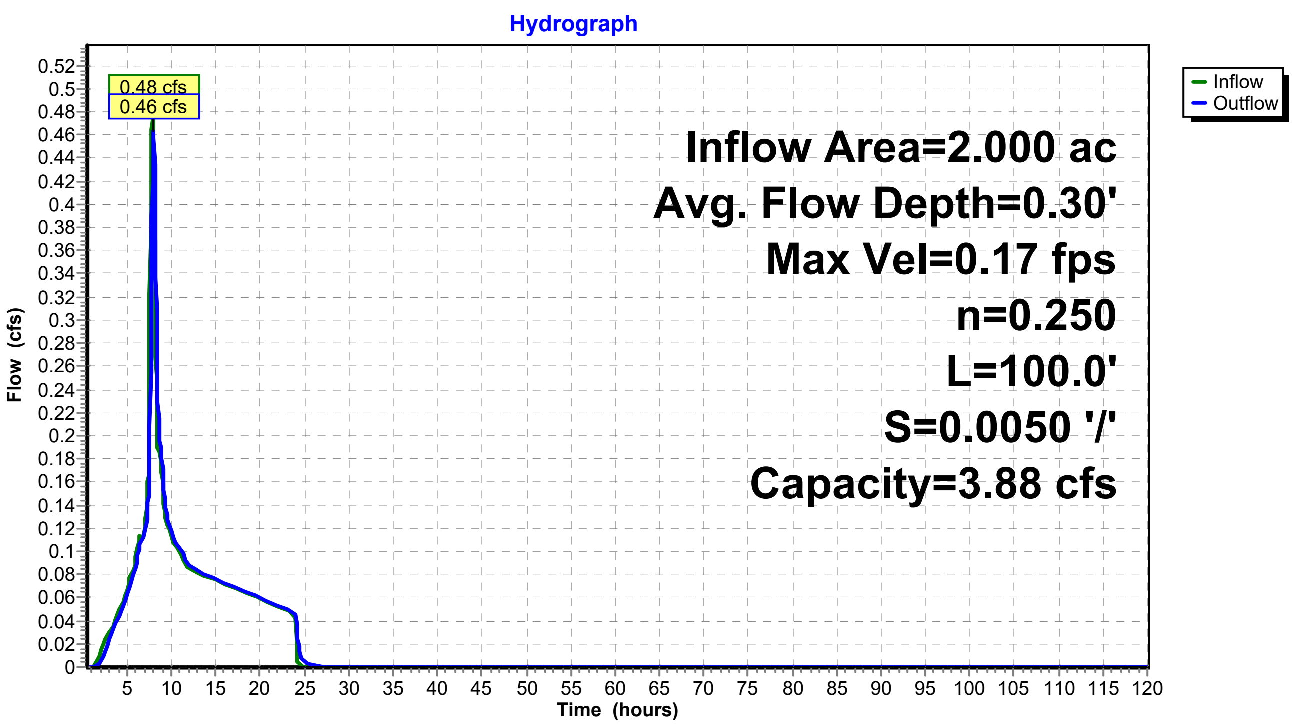
Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.17 fps, Min. Travel Time= 9.5 min
Avg. Velocity = 0.07 fps, Avg. Travel Time= 25.1 min

Peak Storage= 265 cf @ 8.00 hrs
Average Depth at Peak Storage= 0.30' , Surface Width= 9.79'
Bank-Full Depth= 1.00' Flow Area= 11.0 sf, Capacity= 3.88 cfs

8.00' x 1.00' deep channel, n= 0.250
Side Slope Z-value= 3.0 '/' Top Width= 14.00'
Length= 100.0' Slope= 0.0050 '/'
Inlet Invert= 147.00', Outlet Invert= 146.50'

‡

Reach 1R: Veg. Swale



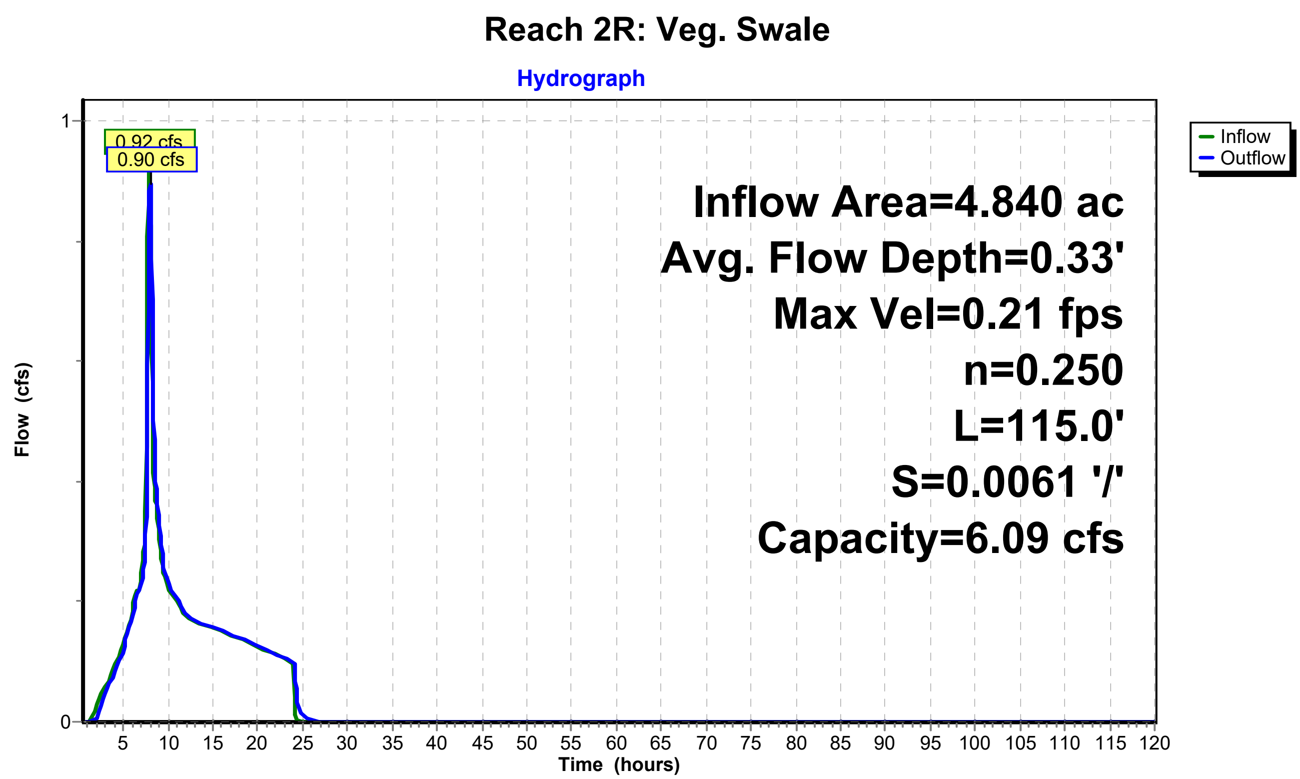
Summary for Reach 2R: Veg. Swale

Inflow Area = 4.840 ac, 63.64% Impervious, Inflow Depth = 0.80" for Salem WQ event
Inflow = 0.92 cfs @ 7.91 hrs, Volume= 0.322 af
Outflow = 0.90 cfs @ 8.00 hrs, Volume= 0.322 af, Atten= 3%, Lag= 5.3 min

Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.21 fps, Min. Travel Time= 9.1 min
Avg. Velocity = 0.08 fps, Avg. Travel Time= 24.8 min

Peak Storage= 492 cf @ 8.00 hrs
Average Depth at Peak Storage= 0.33' , Surface Width= 13.98'
Bank-Full Depth= 1.00' Flow Area= 15.0 sf, Capacity= 6.09 cfs

12.00' x 1.00' deep channel, n= 0.250
Side Slope Z-value= 3.0 '/' Top Width= 18.00'
Length= 115.0' Slope= 0.0061 '/'
Inlet Invert= 147.20', Outlet Invert= 146.50'



Summary for Reach 1R: Veg. Swale

Inflow Area = 2.000 ac, 80.00% Impervious, Inflow Depth > 3.76" for Salem 100 YR event
Inflow = 1.85 cfs @ 7.91 hrs, Volume= 0.626 af
Outflow = 1.85 cfs @ 7.92 hrs, Volume= 0.626 af, Atten= 0%, Lag= 0.9 min

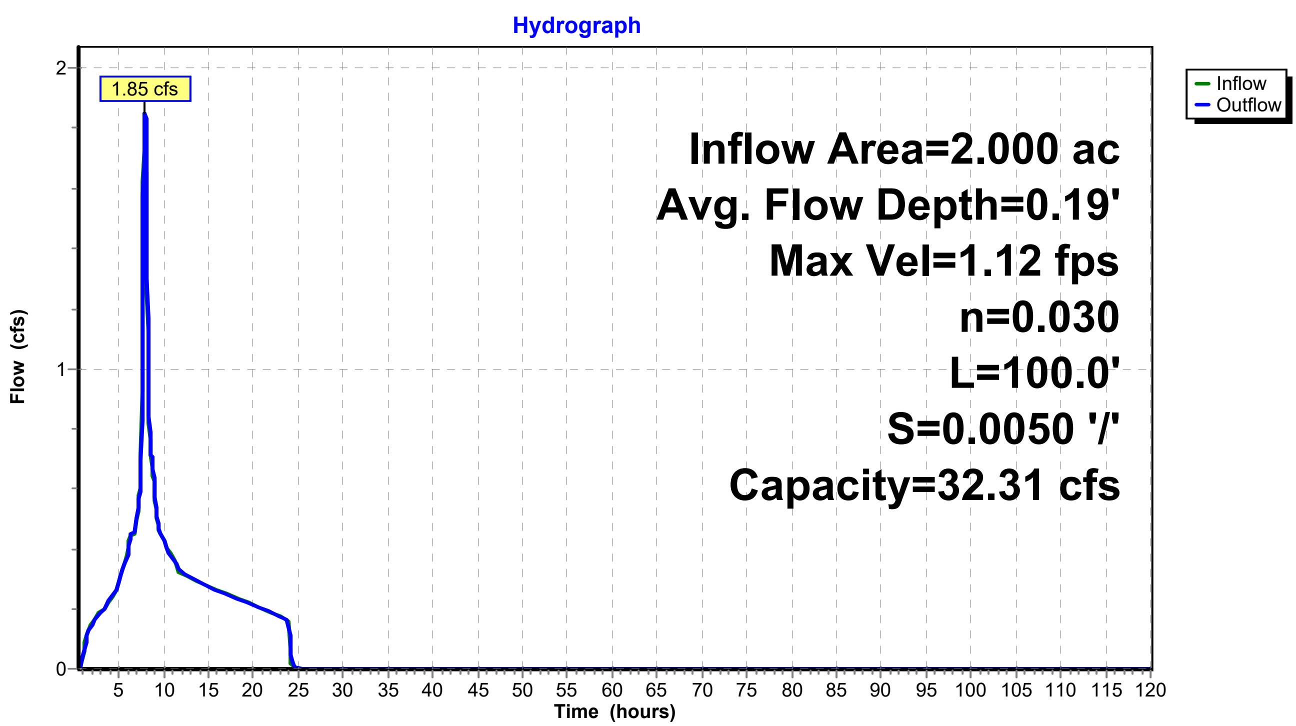
Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Max. Velocity= 1.12 fps, Min. Travel Time= 1.5 min
Avg. Velocity = 0.53 fps, Avg. Travel Time= 3.2 min

Peak Storage= 166 cf @ 7.92 hrs
Average Depth at Peak Storage= 0.19' , Surface Width= 9.16'
Bank-Full Depth= 1.00' Flow Area= 11.0 sf, Capacity= 32.31 cfs

8.00' x 1.00' deep channel, n= 0.030
Side Slope Z-value= 3.0 '/' Top Width= 14.00'
Length= 100.0' Slope= 0.0050 '/'
Inlet Invert= 147.00', Outlet Invert= 146.50'

‡

Reach 1R: Veg. Swale



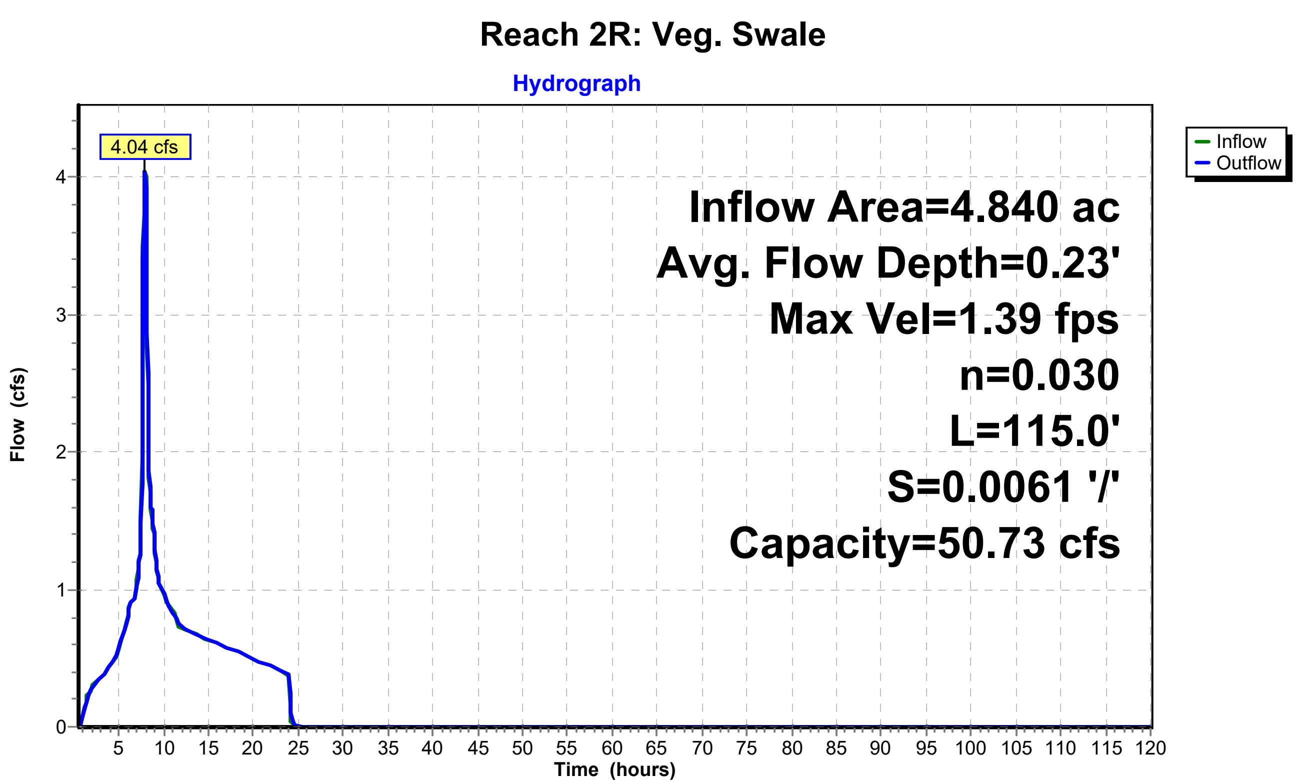
Summary for Reach 2R: Veg. Swale

Inflow Area = 4.840 ac, 63.64% Impervious, Inflow Depth > 3.42" for Salem 100 YR event
Inflow = 4.04 cfs @ 7.92 hrs, Volume= 1.381 af
Outflow = 4.04 cfs @ 7.93 hrs, Volume= 1.381 af, Atten= 0%, Lag= 0.9 min

Routing by Stor-Ind method, Time Span= 0.50-120.00 hrs, dt= 0.05 hrs
Max. Velocity= 1.39 fps, Min. Travel Time= 1.4 min
Avg. Velocity = 0.65 fps, Avg. Travel Time= 3.0 min

Peak Storage= 334 cf @ 7.93 hrs
Average Depth at Peak Storage= 0.23' , Surface Width= 13.37'
Bank-Full Depth= 1.00' Flow Area= 15.0 sf, Capacity= 50.73 cfs

12.00' x 1.00' deep channel, n= 0.030
Side Slope Z-value= 3.0 '/' Top Width= 18.00'
Length= 115.0' Slope= 0.0061 '/'
Inlet Invert= 147.20', Outlet Invert= 146.50'



APPENDIX D

GEOTECHNICAL REPORT

Geotechnical Engineering Report

Riverbend Phase 2
Wallace Road NW
Salem, Oregon

for
Scott Martin Construction, Inc.

December 23, 2020

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Report

Riverbend Phase 2
Wallace Road NW
Salem, Oregon

for
Scott Martin Construction, Inc.

December 23, 2020

GeoEngineers

333 High Street NE, Suite 102
Salem, Oregon 97301
971.304.3078

Geotechnical Engineering Report

Riverbend Phase 2
Wallace Road NW
Salem, Oregon

File No. 23198-001-00

December 23, 2020

Prepared for:

Scott Martin Construction, Inc.
2600 Michigan City Rd NW
Salem, Oregon 97304

Attention: Scott Martin

Prepared by:

GeoEngineers, Inc.
333 High Street NE, Suite 102
Salem, Oregon 97301
971.304.3078

CERTIFIED
OREGON
JOHN L. LAWES
No. E1853
ENGINEERING GEOLOGIST

Benjamin J. Hoffman, PE
Senior Engineer

John L. Lawes, CEG
Staff Geologist

REGISTERED PROFESSIONAL
ENGINEER
60333
OREGON
FEBRUARY 08, 2000
JULIO C VELA

12/23/2020

Julio C. Vela, PhD, PE, GE
Principal

EXPIRES: 06/30/22

BJH:JLL:JCV:cje

GEOENGINEERS

Table of Contents

LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION

GeoEngineers, Inc. (GeoEngineers), is pleased to submit this geotechnical engineering report for the proposed Phase 2 of the Riverbend Neighborhood Center and Apartment Development located along the west side of Wallace Road NW at approximately 2501 Wallace Road NW in Salem, Oregon. Phase 2 of the Riverbend development is located adjacent to and north of Phase 1 of the same development constructed by Scott Martin Construction, Inc. Our understanding of the project is based on the preliminary master plan for proposed development provided to us by Steve Ward with Westech Engineering, Inc., as well as our previous work performed on Phase 1. The location of the site relative to the surrounding area is shown in the Vicinity Map, Figure 1.

Based on the information provided to us, we understand that the development will include neighborhood use (clubhouse type) and commercial structures, multi-family residential apartment structures, and associated parking areas, drive aisles and sidewalks. Two commercial structures are identified on the preliminary plan, consisting of a 7,600 square-foot (sf) retail building and a 7,600 sf live-work over retail building. Typical apartment structures are noted as three-story, wood-framed apartment buildings consistent with those of Phase 1 development. Paved areas include 283 parking stalls between the commercial and the residential structures and the drive aisles. We have also assumed that maximum cuts and fills will be less than 5 feet each and that on-site retaining walls will be less than 8 feet in height.

Our structural design recommendations are based on the following:

- For commercial buildings, we assumed that maximum column and wall loads will be on the order of 60 kips per column and 4 kips per lineal foot (klf) respectively, and that floor loads for slabs on grade will be 100 pounds per square foot (psf) or less.
- For apartments, we assumed typical light wood-frame structural loads, including column loads less than 30 kips, wall loads of 3 klf or less, and floor slab loads of 100 psf or less.

2.0 SCOPE OF SERVICES

The purpose of our services was to evaluate soil and groundwater conditions as a basis for developing geotechnical engineering design recommendations as well as a Geological Assessment in general accordance with the requirements of the City of Salem (City) Revised Code Chapter 810.030 (a) for the site.

Our proposed scope of services included the following:

1. Reviewed existing available subsurface soil and groundwater information, geologic maps and other available geotechnical engineering related information pertinent to the site.
2. Coordinated and managed the field investigation, including public utility notification and scheduling of subcontractors and GeoEngineers' field staff.
3. Explored subsurface soil and groundwater conditions at the site by drilling a total of 12 borings. Ten borings (B-1 and B-2, B-4, and B-6 through B-12) advanced within or near proposed apartment and commercial building footprints, each extending to a depth of 16½ to 21½ feet below ground surface (bgs), and two borings (B-3 and B-5) advanced in proposed paved and parking areas, extending to a

depth of 6½ feet bgs. Exploration locations are shown in the Site Plan, Figure 2. Logs of each exploration are provided in Appendix A.

4. Obtained samples at representative intervals from the explorations, observed groundwater conditions and maintained detailed logs in general accordance with ASTM International (ASTM) Standard Practices Test Method D 2488. Qualified staff from our office observed and documented field activities.
5. Performed two infiltration tests (IT-1 and IT-2) at select locations at the project site as shown in Figure 2. Infiltration testing was conducted as required by Division 004 of the *City of Salem Department of Public Works Administrative Rules Design Standards* (COSDS).
6. Performed laboratory tests on selected soil samples obtained from the explorations to evaluate pertinent engineering characteristics. Laboratory test results are included in the exploration logs in Appendix A.
7. Performed a general geologic assessment of slopes at the site relative to existing stability and impact on proposed site development consistent with the requirements of Salem Revised Code (SRC) Chapter 810.030 (a).
8. Provided a geotechnical evaluation of the site and provided project-specific design recommendations in this geotechnical report that address the following geotechnical components:
    a. A general description of site topography, geology and subsurface conditions.
    b. An opinion as to the existing general stability of site slopes and conclusions regarding the effect of the existing geologic conditions on potential fill slope construction as performed in general accordance with our recommendations.
    c. An opinion as to the adequacy of the proposed development from a geotechnical engineering standpoint.
    d. Recommendations for site preparation measures, including disposition of undocumented fill and unsuitable native soils, recommendations for temporary cut slopes and constraints for wet weather construction.
    e. Provide estimates of groundwater level and management recommendations.
    f. Recommendations for temporary excavation and temporary excavation protection, such as excavation sheeting and bracing.
    g. Recommendations for earthwork construction, including use of on-site and imported structural fill, and fill placement and compaction requirements.
    h. Recommendations for use in designing conventional retaining walls up to 8 feet tall, including backfill and drainage requirements, and static and seismic lateral earth pressures.
    i. Recommendations for shallow foundations to support the proposed structures, including minimum width and embedment, design soil bearing pressures, settlement estimates (total and differential), coefficient of friction and passive earth pressures for sliding resistance.
    j. Recommendations for supporting on-grade slabs, including base rock, capillary break, and modulus of subgrade reaction.
    k. Summary of infiltration testing and discussion of suitability of on-site infiltration facilities based on subsurface conditions.

l. Seismic design parameters, including soil site class evaluation in accordance with the current version of the International Building Code (IBC).

m. Recommendations for constructing asphaltic concrete (AC) pavements on-site, including subgrade, drainage, base rock, and pavement section.

Our geotechnical work has been directly supervised by a professional engineer licensed in the state of Oregon.

3.0 SITE CONDITIONS

3.1. Site Geology

The project site is located within the western edge of the Willamette Basin physiographic province in the lowermost foothills of the Eola Hills. The project site is located on a raised terrace west of Greene Creek, a second-order tributary of the Willamette River.

The “Geologic Map of the Rickreall and Salem West Quadrangles, Oregon” (Bela 1981) shows the site mantled by higher Pleistocene terrace deposits. This alluvial sediment is described as “semiconsolidated… sand, silt, and clay.” The topography of the site and our field investigation suggests that the near-surface site geology is generally consistent with published geologic mapping.

3.2. Surface Conditions

The proposed new development is located on an approximate 7-acre property consisting of a group of several residential properties with existing house and barn and related structures, driveways, fencing, trees, and open grass fields. The property gently slopes from the west at approximate Elevation 180 feet North American Vertical Datum of 1988 (NAVD88) to the east at Wallace Road NW at approximate Elevation 160 feet NAVD88.

3.3. Slope Conditions

A general geologic assessment of slopes at the site relative to existing stability and impact on proposed site development consistent with the requirements of SRC, Chapter 810.030 (a) was performed. We performed a visual geologic reconnaissance on November 5, 2020, to observe existing slope conditions. Site topography generally slopes down to the east, while the ground surface is typically undulatory to gently sloping, with maximum gradients typically less than 5H:1V (horizontal to vertical) to as low as 30H:1V or flatter.

The interior site slopes appear planar to convex and regular. We did not observe indications of large, deeply seated, recent or active slope instability such as concave, steeply inclined bare-soil scarps, bulging or hummocky topography, anomalous drainage features or vegetation.

Light Detection and Ranging (LiDAR) landslide hazard mapping has not been completed for the West Salem area. The Oregon State Landslide Information Layer (SLIDO) (DOGAMI 2017) shows a portion of the southernmost edge and northern corner of the site within the “moderate – landslide possible” hazard zone. The SLIDO legend does not provide a mapped source of this supposed hazard and our observations do not support the SLIDO classification.

3.4. Subsurface Conditions

We completed field explorations at the project site on November 5 and 6, 2020. Our explorations included 12 drilled borings (B-1 through B-12) to depths of between 6½ to 21½ feet bgs, two infiltration tests (IT-1 and IT-2) at depths of 2½ and 3 feet bgs, respectively, and two dynamic cone penetrometer (DCP) readings (DCP-1 and DCP-2) to depths ranging from 24 to 61 inches bgs. A member of our professional staff maintained detailed logs of the soils encountered, test results collected, and gathered representative soil samples. A summary of our exploration methods as well as the boring logs, DCP test logs and infiltration test logs can be found in Appendix A. Laboratory test results are also provided in the boring logs and described in Appendix A. The approximate locations of the explorations are shown in Figure 2.

The upper approximately 20 feet of the subsurface primarily consists of gray and brown silt to sandy silt that was observed to be generally medium stiff to very stiff with sand content decreasing with depth. Drilling was terminated within the medium stiff to very stiff silt to sandy silt layer in all but three of the borings (B-2, B-9, and B-10). Borings B-2, B-9, and B-10 were each drilled to a depth of 21½ feet and are located in the eastern half of the project site where a coarser grained soil layer was encountered below a depth of approximately 20 feet. Near the bottom of each of the three borings a gray poorly graded sand and gravel layer that was observed to be dense to very dense was encountered. All three borings were terminated within the dense to very dense poorly graded sand and gravel layer at a depth of 21½ feet. The reader is referred to the exploration logs for more detailed information about the soils encountered in the explorations.

3.5. Groundwater

Groundwater was not encountered during drilling; however, due to increasing moisture content and condition of soil samples, it could be inferred that groundwater is present at or just below the maximum depth drilled of 21½ feet bgs. Explorations were conducted at the end of the dry weather cycle. Groundwater elevations will be shallower after prolonged periods of wet weather and may be as shallow as 10 to 12 feet bgs.

Also, groundwater may be present at relatively shallow depths in a perched or capillary condition during wet times of the year or during extended periods of wet weather. Groundwater conditions at the site are expected to vary seasonally due to rainfall events and other factors not observed in our explorations.

4.0 CONCLUSIONS

4.1. General

Based on our explorations, testing and analyses, it is our opinion that the site is suitable for the proposed project from a geotechnical standpoint, provided the recommendations in this report are included in design and construction. We offer the following summary of conclusions regarding geotechnical design at the site.

- Groundwater was not encountered in the upper 21½ feet bgs during drilling. However, based on our experience and our observations, perched groundwater may be present during periods of persistent rainfall and may be as shallow as 10 to 12 feet bgs near the end of a wet cycle of weather. Perched groundwater conditions may require dewatering of trenches or deeper excavations by use of sumps during construction and site grading.

- Surface conditions at the site consist primarily of several residential properties with existing house and barn and related structures, driveways, fencing, trees, and open grass fields; therefore, stripping and demolition will be required in all proposed development areas. We anticipate a stripping depth of approximately 6 inches bgs to remove the grass roots. Grubbing and deeper excavations up to several feet will be required to remove the root zones of shrubs and trees. Existing structures should be demolished and removed from the site and existing utilities should be abandoned in-place or removed from the site.
- Minimal (near zero) rates of infiltration were measured during the testing period. The water level in IT-1, left to infiltrate overnight, declined approximately 0.75 inch in 15 hours, or approximately 0.05 inch/hour (in/hr). In general, soils with infiltration rates less than 2 in/hr are not well suited as the sole means of stormwater disposal for sites.
- Typical infiltration facilities require at least 5 feet of separation between the base of the facility and the seasonal high groundwater level. Groundwater was not encountered at depths of at least 21½ feet bgs.
- On-site near surface soils generally consist of silt. The silty soil will become significantly disturbed when trafficked during earthwork, particularly when construction traffic over the site occurs during periods of wet weather or when the moisture content of the soil is more than a few percentage points above optimum. Wet weather construction practices will be required over exposed native soils unless earthwork occurs during the dry summer months (typically mid-July to mid-September).
- Proposed structures can be satisfactorily supported on continuous and isolated shallow foundations supported on medium stiff to very stiff native soils or on structural fill that extends to native soil.
- For proposed commercial structures, our foundation recommendations are based on maximum anticipated loads of 60 kips or less for columns, 4 klf or less for walls, and floor loads of 100 psf or less. Based on these design loads, we estimate total settlement to be less than 1 inch. If larger structural loads are anticipated, we should review and reassess the estimated settlement.
- For proposed apartment structures, our foundation recommendations are based on maximum anticipated loads of 30 kips or less for columns, 3 klf or less for walls, and floor loads of 100 psf or less. Based on these design loads, we estimate total settlement to be less than 1 inch. If larger structural loads are anticipated, we should review and reassess the estimated settlement.
- Fill material encountered at subgrade elevation should be evaluated by GeoEngineers during construction. Soft fill or fill with significant debris or unsuitable material should be removed to native stiff or firmer material and replaced with compacted structural fill.
- Slabs-on-grade will be satisfactorily supported on medium stiff to very stiff native soils with a minimum 6-inch-thick layer of compacted crushed rock base overlying approved subgrade or on structural fill over medium stiff to stiff native soils.
- Standard pavement sections prepared as described in this report will suitably support the estimated traffic loads provided the site subgrade is prepared as recommended.

5.0 INFILTRATION TESTING

5.1. General

As is typical for development projects in the Salem area, we conducted infiltration tests on site to assist in evaluation of the site for potential stormwater infiltration design. We conducted two infiltration tests, at depths of 3 and 2½ feet bgs; one (IT-1) in the northeast portion of the site near boring B-5, the other (IT-2) near the south property boundary about 250 feet west of the edge of the Wallace Road right-of-way. This is a typical depth for consideration of stormwater disposal.

Testing was conducted using the encased falling head procedure consistent with the method outlined in "Division 004" of the COSDS. A 2- to 3-inch-thick layer of pea gravel was placed in the pipes prior to adding water to diminish disturbance from flowing water at the base of the pipe interior. The test areas were pre-soaked over a 4-hour period by repeated addition of water into the pipe when necessary. Based on observations, a good seal was present between the base of the pipe and the underlying soil.

After the saturation period, both pipes were filled with clean water to at least 1 foot above the bottom of the pipe placed in the boring. The water level was then monitored over three, 1-hour iterations. In the case where water levels fall during the time-measured testing, infiltration rates diminish as a result of less head from the water column in the test.

Both test locations showed a minimal measurable decline in head during the saturation or the three, 1-hour test periods. Therefore IT-1 was left to infiltrate overnight; approximately 15 hours later the water level was 0.75 inches lower than the final test reading, yielding an infiltration rate of roughly 0.05 in/hr.

Field test results are summarized in Table 1.

TABLE 1. INFILTRATION RESULTS

Infiltration Test No.	Location	Depth (feet)	USCS Material Type	Field Measured Infiltration Rate[1] (inches/hour)
IT-1	East-central area of site (near B-5)	3	ML	0.05 (see above)
IT-2	Southeast area of site	2.5	ML	0.0[2]

Notes:

[1] Appropriate factors should be applied to the field-measured infiltration rate, based on the design methodology and specific system.

[2] Measured rate not perceptible over 3-hour test period. Likely similar to rate noted for IT-1 in long-term observation.

USCS = Unified Soil Classification System

Infiltration rates shown in Table 1 are field-measured rates and represent a relatively short-term measured rate. It is likely that IT-2 has some amount of infiltration, it is just not measurable in the test time window. Further, for matters of facility design, factors of safety have not been applied for the type of infiltration system being considered, or for variability that may be present in the on-site soil. In our opinion, and consistent with the state of the practice, correction factors should be applied to this measured rate to reflect the small area of testing and the number of tests conducted. Correction factors to minimal amounts would result in even lower rates for design, effectively resulting in a zero rate of infiltration.

Actual depths, lateral extent, and estimated infiltration rates can vary from the values presented above. Field testing/confirmation during construction is often required in large or long systems or other situations where soil conditions may vary within the area where the system is constructed. The results of this field-testing during construction may yield a higher rate of infiltration for the overall area, but would require modifying the system design for higher rates.

Even in the best of circumstances. the infiltration flow rate of a focused stormwater system typically diminishes over time as suspended solids and precipitates in the stormwater slowly clog the void spaces between soil particles or cake on the infiltration surface. The serviceable life of a stormwater system can be extended by pre-filtering or with on-going accessible maintenance. Eventually, most systems will fail and will need to be replaced or have media regenerated or replaced. We recommend that infiltration systems include an overflow that is connected to a suitable discharge point. Also, infiltration systems can cause localized high groundwater levels and should not be located near basement walls, retaining walls, or other embedded structures unless these are specifically designed to account for resulting hydrostatic pressure. Infiltration locations should not be located on or adjacent to sloping ground, unless it is approved by the project geotechnical engineer of record, and should not be infiltrated at a location that allows for flow to travel laterally toward a slope face, such as a mounded water condition or too near the slope face.

5.2. Suitability of Infiltration System

Successful design and implementation of stormwater infiltration systems, and whether a system is suitable for a development depend on several site-specific factors. Stormwater infiltration systems are generally best suited for sites having sandy or gravelly soil with saturated hydraulic conductivities greater than 2 in/hr. Sites with silty or clayey soil, including sites with fine sand, silty sand such as at the upper portions of this site, or gravel with a high percentage of silt or clay in the matrix are generally not well suited for stormwater infiltration. Soil that has higher fine-grained matrices is susceptible to volumetric change and softening during wetting and drying cycles. Fine-grained soil also has large variations in the magnitude of infiltration rates because of bedding and stratification that occurs during deposition and often has thin layers of less permeable or impermeable soil within a larger layer.

As a result of fine-grained soil conditions and relatively low measured infiltration rates, we recommend infiltration of stormwater not be used in the upper soils, or at the very least not be used as the sole method of stormwater management at this site unless those design factors can be otherwise accounted for by increasing infiltration area or coupling with other methods of stormwater disposal. At a minimum, an overflow method should be provided for the overall system.

6.0 EARTHWORK RECOMMENDATIONS

6.1. Site Preparation

In general, site preparation and earthwork for site development will include demolition of existing residential and farm structures, excavation for removal of existing tree and tree root removal, stripping and grubbing, grading the site and excavating for utilities and foundations, and may also include removal or relocation of existing site utilities where present beneath proposed buildings.

6.1.1. Demolition

Existing structures, or remnants of structures or other debris should be demolished and removed from the site. If present, existing utilities that will be abandoned on site should be identified prior to project construction. Abandoned utility lines beneath proposed structural areas should be completely removed or filled with grout if abandoned and left in-place in order to reduce potential settlement or caving in the future. Materials generated during demolition of existing utilities should be transported off site for disposal.

Existing voids and new depressions created due to removal of existing utilities, or other subsurface elements, should be cleaned of loose soil or debris down to firm soil and backfilled with compacted structural fill. Disturbance to a greater depth should be expected if site preparation and earthwork are conducted during periods of wet weather.

6.1.2. Stripping and Grubbing

Based on our observations at the site, we estimate that the depth of stripping of on-site organics in wild grass-covered areas will be on the order of about 6 inches. Greater stripping depths may be required to remove localized zones of loose or organic soil, and in areas where moderate to heavy vegetation may be present, or surface disturbance has occurred. In addition, if present in areas of proposed development, the primary root systems of trees should be completely removed. Stripped material should be transported off site for disposal or processed and used as fill in landscaping areas.

Where encountered, trees and their root balls should be grubbed to the depth of the roots, which could exceed 3 feet bgs. Depending on the methods used to remove the preceding material, considerable disturbance and loosening of the subgrade could occur. We recommend that disturbed soil be removed to expose stiff native soil. The resulting excavations should be backfilled with structural fill.

6.2. Subgrade Preparation and Evaluation

Upon completion of site preparation activities, exposed subgrades that are to receive fill should be compacted in-place prior to fill placement due to the presence of a tilled zone that extends to depths of approximately 12 inches bgs in open areas of the project site. If site grading extends to below these depths, and to the native in-place (non-tilled) soils, compaction of in-place subgrade is not required.

Exposed subgrades should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tired construction equipment, where space allows, to identify soft, loose or unsuitable areas. Probing may be used for evaluating smaller areas or where proof-rolling is not practical. Proof-rolling and probing should be conducted prior to placing fill, and should be performed by a representative of GeoEngineers who will evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. If soft or loose zones are identified during proof-rolling or probing, these areas should be excavated to the extent indicated by our representative and replaced with structural fill.

As discussed in Section 4.1 of this report, the native fine-grained, silty soil can be sensitive to small changes in moisture content and will be difficult, if not impossible, to compact adequately during wet weather. While tilling and compacting the subgrade is the economical method for subgrade improvement, it will likely only be possible during extended dry periods and following moisture conditioning of the soil.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe.

Observations, probing, and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing should be removed and replaced with compacted structural fill.

6.3. Subgrade Protection and Wet Weather Considerations

The upper fine-grained soils at the site are highly susceptible to moisture. Wet weather construction practices will be necessary if work is performed during periods of wet weather. If site grading will occur during wet weather conditions, it will be necessary to use track-mounted equipment, load material into trucks supported on gravel work pads and employ other methods to reduce ground disturbance. The contractor should be responsible to protect the subgrade during construction, reflective of their proposed means and methods and time of year.

Earthwork planning should include considerations for minimizing subgrade disturbance. The following recommendations can be implemented if wet weather construction is considered:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance such as haul roads and rocked staging areas.
- When on-site fine-grained soils are wet of optimum, they are easily disturbed and will not provide adequate support for construction traffic or the proposed development. The use of granular haul roads and staging areas will be necessary for support of construction traffic. Generally, a 12- to 16-inch-thick mat of imported granular base rock aggregate material is sufficient for light staging areas for building pad and light staging activities but is not expected to be adequate to support repeated heavy equipment or truck traffic. The granular mat for haul roads and areas with repeated heavy construction traffic should be increased to between 18 and 24 inches. The actual thickness of haul roads and staging areas should be based on the contractor's approach to site development, and the amount and type of construction traffic.
- During periods of wet weather, concrete should be placed as soon as practical after preparation of the footing excavations. Foundation bearing surfaces should not be exposed to standing water. If water collects in the excavation, it should be removed before placing structural fill or reinforcing steel.

Subgrade protection for foundations consisting of a lean concrete mat may be necessary if footing excavations are exposed to extended wet weather conditions.

- The base rock (Aggregate Base and Aggregate Subbase) thicknesses described in Section 9.0 of this report is intended to support post-construction design traffic loads. The design base rock thicknesses will likely not support repeated heavy construction traffic during site construction, or during pavement construction. A thicker base rock section, as described above for haul roads, will likely be required to support construction traffic.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations, probing and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing should be removed and replaced with compacted structural fill.

6.4. Cement Treated Subgrade Design

These recommendations are included as a potential alternative to the use of imported granular material for wet weather structural fill provided areas being graded or developed make the cement treating process a feasible option.

An experienced contractor may be able to amend the on-site soil with portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content and amendment quantities. Specific recommendations, based on exposed site conditions, for soil amending can be provided if necessary. However, for preliminary planning purposes, it may be assumed that a minimum of 5 percent cement (by dry weight, assuming a unit weight of 100 pounds per cubic foot [pcf]) will be sufficient for subgrade and general fill amendment. Treatment depths of 12 to 16 inches for roadway subgrades are typical (assuming a seven-day unconfined compressive strength of at least 80 pounds per square inch [psi]), though they may be adjusted in the field depending on site conditions. Soil amending should be conducted in accordance with the specifications provided in Oregon Structural Specialty Code 00344 (Treated Subgrade).

Portland cement-amended soil is hard and has low permeability; therefore, this soil does not drain well nor is it suitable for planting. Future landscape areas should not be cement amended, if practical, or accommodations should be planned for drainage and planting. Cement amendment should not be used if runoff during construction cannot be directed or drained away from areas that would be negatively affected by runoff from the amended surface, including adjacent building foundations, low-lying wet areas or active waterways, and area drainage paths.

We recommend a target strength for cement-amended soils of 80 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. However, for preliminary design purposes, 4 to 5 percent cement by weight of dry soil can generally be used when the soil moisture content does not exceed approximately 25 percent. If the soil moisture content is in the range of 25 to 35 percent, 5 to 7 percent by weight of dry soil is recommended. The amount of cement added to the soil may need to be adjusted based on field observations and performance.

When used for construction of pavement, staging, or haul road subgrades, the amended surface should be protected from abrasion by placing a minimum 4-inch thickness of crushed rock. To prevent strength loss during curing, cement-amended soil should be allowed to cure for a minimum of four days prior to placing the crushed rock. The crushed rock may typically become contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas such that the minimum thickness of free-draining base at the surface is 4 inches.

It is not possible to amend soil during heavy or continuous rainfall. Work should be completed during suitable conditions.

6.5. Excavation

Based on the materials encountered in our subsurface exploration, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations.

The earthwork contractor should be responsible for reviewing this report, including the boring logs, providing their own assessments, and providing equipment and methods needed to excavate the site soils while protecting subgrades.

6.6. Dewatering

As discussed in Section 3.5 of this report, groundwater was not encountered during drilling in the upper 21½ feet at the site. We do not anticipate excavations to extend below this depth. However, if excavations do extend into saturated/wet soils they should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. Deeper excavations may require more intensive or filtered dewatering or use of well points. Deeper excavations that extend below groundwater into sandier soils (encountered below depths of 15 feet bgs) may be difficult to dewater with conventional sumps because inflow of water may promote a “running soils” condition into excavations, where sandy material flows in with seeping groundwater. For deep excavations or where running soils are encountered, dewatering from well points would likely be required to maintain an open and workable trench.

In addition to groundwater seepage and upward confining flow, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

6.7. Trench Cuts and Trench Shoring

All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. Site soils within expected excavation depths typically range from medium stiff to stiff silt. In our opinion, native soils are generally OSHA Type B, provided there is no seepage and excavations occur during periods of dry weather. Excavations deeper than 4 feet should be shored or laid back at an inclination of 1H:1V for Type B soils. Flatter slopes may be necessary if workers are required to enter. Excavations made to construct footings or other structural elements should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

Shoring for trenches less than 6 feet deep that are above the effects of groundwater should be possible with a conventional box system. Slight to moderate sloughing should be expected outside the box. Shoring deeper than 6 feet or below the groundwater table should be designed by a registered engineer before installation. Further, the shoring design engineer should be provided with a copy of this report.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods and sequencing of the construction operations and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

6.8. Erosion Control

Erosion control plans are required on construction projects located within Marion County in accordance with Oregon Administrative Rules (OAR) 340-41-006 and 340-41-455 and City regulations. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds and granular haul roads.

6.9. Structural Fill and Backfill

6.9.1. General

Structural areas include areas beneath foundations, floor slabs, pavements, and any other areas intended to support structures or within the influence zone of structures, should generally meet the criteria for structural fill presented below. All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches (3-inch maximum particle size in building footprints) and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

6.9.2. On-Site Soils

On-site near-surface soil consists of native silt to sandy silt. On-site soils can be used as structural fill, provided the material meets the above requirements, although due to moisture sensitivity, this material will likely be unsuitable as structural fill during most of the year. If the soil is too wet to achieve satisfactory compaction, moisture conditioning by drying back the material will be required. If the material cannot be properly moisture conditioned, we recommend using imported material for structural fill.

An experienced geotechnical engineer from GeoEngineers should determine the suitability of on-site soil encountered during earthwork activities for reuse as structural fill.

6.9.3. Imported Select Structural Fill

Select imported granular material may be used as structural fill. The imported material should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine sizes (approximately 25 to 65 percent passing the U.S. No. 4 sieve). It should have less than 5 percent passing the U.S. No. 200 sieve and have a minimum of 75 percent fractured particles according to American Association of State Highway and Transportation Officials (AASHTO) TP-61.

6.9.4. Aggregate Base

Aggregate base material located under floor slabs and pavements and crushed rock used in footing overexcavations should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1 inch and have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls), and meet the gradation requirements in Table 2. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

TABLE 2. RECOMMENDED GRADATION FOR AGGREGATE BASE

Sieve Size	Percent Passing (by weight)
1 inch	100
½ inch	50 to 65
No. 4	40 to 60
No. 40	5 to 15
No. 200	0 to 5

6.9.5. Trench Backfill

Backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 5 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone backfill, Imported Select Structural Fill may be used as described above.

6.9.6. Retaining Wall Backfill

Fill placed to provide a drainage zone behind retaining walls should meet the general requirements above and consist of free-draining sand and gravel or crushed rock with a maximum particle size of ¾ inch and less than 3 percent passing the U.S. No. 200 sieve.

6.10. Fill Placement and Compaction

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM Test Method D 1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Fill material that is not near the optimum moisture content should be moisture conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts, and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 3, below. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 18 inches.

TABLE 3. COMPACTION CRITERIA

Fill Type	Compaction Requirements Percent Maximum Dry Density Determined by ASTM Test Method D 1557 at ± 3% of Optimum Moisture		
	0 to 2 Feet Below Subgrade	> 2 Feet Below Subgrade	Pipe Zone
Fine-grained soils (non-expansive)	92	92	----
Imported Granular, maximum particle size < 1¼ inch	95	95	----
Imported Granular, maximum particle size 1¼ inch to 4 inches (3-inch maximum under building footprints)	n/a (proof-roll)	n/a (proof-roll)	----
Retaining Wall Backfill*	92	92	----
Nonstructural Zones	90	90	90
Trench Backfill	95	90	90

Note:

* Measures should be taken to prevent overcompaction of the backfill behind retaining walls. We recommend placing the zone of backfill located within 5 feet of the wall in lifts not exceeding about 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor and a jumping jack.

A representative from GeoEngineers should evaluate compaction of each lift of fill. Compaction should be evaluated by compaction testing unless other methods are proposed for oversized materials and are approved by GeoEngineers during construction. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as proof-rolling.

6.11. Slopes

6.11.1. Permanent Slopes

Permanent cut or fill slopes should not exceed a gradient of 2H:1V. Where access for landscape maintenance is desired, we recommend a maximum gradient of 3H:1V. Fill slopes should be overbuilt by at least 12 inches and trimmed back to the required slope to maintain a firm face.

Slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

6.11.2. Temporary Slopes

All temporary soil cuts associated with site excavations (greater than 4 feet in depth) should be adequately sloped back to prevent sloughing and collapse, in accordance with applicable OSHA and state guidelines.

Temporary cut slopes should not exceed a gradient appropriate for the soil type being excavated. As noted in Section 6.7, medium stiff to very stiff silt soils should be considered OSHA Soil Type B. However, because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated before construction.

The stability and safety of cut slopes depend on a number of factors, including:

- The type and density of the soil.
- The presence and amount of any seepage.
- Depth of cut.
- Proximity and magnitude of the cut to any surcharge loads, such as stockpiled material, traffic loads or structures.
- Duration of the open excavation.
- Care and methods used by the contractor.

We recommend that stability of the temporary slopes used for construction be the responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface. If groundwater seepage is encountered within the excavation slopes, the cut slope inclination may have to be flatter than 1.5H:1V. However, appropriate inclinations will ultimately depend on the actual soil and groundwater seepage conditions exposed in the cuts at the time of construction. It is the responsibility of the contractor to ensure that the excavation is properly sloped or braced for worker protection, in accordance with applicable guidelines. To assist with this effort, we make the following recommendations regarding temporary excavation slopes:

- Protect the slope from erosion with plastic sheeting for the duration of the excavation to minimize surface erosion and raveling.
- Limit the maximum duration of the open excavation to the shortest time period possible.
- Place no surcharge loads (equipment, materials, etc.) within 10 feet of the top of the slope.

More restrictive requirements may apply depending on specific site conditions, which should be continuously assessed by the contractor.

If temporary sloping is not feasible based on site spatial constraints, excavations could be supported by internally braced shoring systems, such as a trench box or other temporary shoring. There are a variety of options available. We recommend that the contractor be responsible for selecting the type of shoring system to apply.

6.11.3. Slope Drainage

If seepage is encountered at the face of permanent or temporary slopes, it will be necessary to flatten the slopes or install a subdrain to collect the water. We should be contacted to evaluate such conditions on a case-by-case basis.

7.0 STRUCTURAL DESIGN RECOMMENDATIONS

7.1. Foundation Support Recommendations

Proposed structures can be satisfactorily founded on continuous strip or isolated column footings supported on firm native soils, or on structural fill placed over native soils. Exterior footings should be established at least 18 inches below the lowest adjacent grade. The recommended minimum footing depth is greater than the anticipated frost depth. Interior footings can be founded a minimum of 12 inches below the top of the floor slab. Isolated column and continuous wall footings should have minimum widths of 24 and 18 inches, respectively. For the proposed commercial structures, we have assumed that the maximum isolated column loads will be on the order of 60 kips, wall loads will be 4 klf or less and floor loads for slabs-on-grade will be 100 psf or less for the proposed development. For the proposed apartment structures, we have assumed that the maximum isolated column loads will be on the order of 30 kips, wall loads will be 3 klf or less and floor loads for slabs-on-grade will be 100 psf or less. If design loads exceed these values, we should be notified as our recommendations may need to be revised.

7.1.1. Foundation Subgrade Preparation

We recommend that prepared subgrades be observed by a member of our firm, who will evaluate the suitability of the subgrade and identify any areas of yielding, which are indicative of soft or loose soil. The exposed subgrade soil should be probed with a ½-inch-diameter steel rod. If soft, yielding or otherwise unsuitable areas are revealed during probing the unsuitable soils should be removed and replaced with structural fill, as needed.

Fill material encountered at subgrade elevation should be evaluated by GeoEngineers during construction. Soft fill or fill with significant debris or unsuitable material should be removed to native medium stiff or stiffer material and replaced with compacted structural fill. The width of the overexcavation should extend beyond the edge of the footing a distance equal to the depth of the overexcavation below the base of the footing.

We recommend loose or disturbed soils be removed before placing reinforcing steel and concrete. Foundation bearing surfaces should not be exposed to standing water. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing reinforcing steel. A thin layer (2 to 3 inches) of crushed rock can be used to provide protection to the subgrade from light foot traffic. Compaction should be performed as described in Section 6.10.

We recommend GeoEngineers observe all foundation excavations before placing concrete forms and reinforcing steel to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

7.1.2. Bearing Capacity – Spread Footings

We recommend conventional footings be proportioned using a maximum allowable bearing pressure of 2,500 psf if supported on medium stiff or stiffer native silt or structural fill bearing on these materials. The recommended bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

7.1.3. Foundation Settlement

Foundations designed and constructed as recommended are expected to experience settlements of less than 1 inch. Differential settlements of up to one half of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

7.1.4. Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressures on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid unit weight of 250 pcf for foundations confined by native medium stiff or stiffer silt and 400 pcf if confined by a minimum of 2 feet of imported granular fill.

We recommend using a friction coefficient of 0.38 for foundations placed on the native medium stiff or stiffer silt, or 0.48 for foundations placed on a minimum 1-foot thickness of compacted crushed rock. The passive earth pressure and friction components may be combined provided the passive component does not exceed two-thirds of the total.

The passive earth pressure value is based on the assumptions that the adjacent grade is level and static groundwater remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures unless the adjacent area is covered with pavement or slab-on-grade. The lateral resistance values include a safety factor of approximately 1.5.

7.2. Drainage Considerations

We recommend the ground surface be sloped away from the buildings at least 2 percent. All downspouts should be tightlined away from the building foundation areas and should also be discharged into a stormwater disposal system. Downspouts should not be connected to footing drains.

Although not required based on expected groundwater depths, if perimeter footing drains are used for below-grade structural elements or crawlspaces, they should be installed at the base of the exterior footings. If used, perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of, and surrounded by, 6 inches of drainage material enclosed in a non-woven geotextile such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drain material. We recommend against using flexible tubing for footing drainpipes. The perimeter drains should be sloped to drain by gravity to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered and placed in flush-mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

If an elevator pit or utility vaults or other subterranean open structural elements are installed below the expected level of groundwater, we recommend foundation drains be installed as described above. Active dewatering or tightline routing of draining water will be required during wet times of the year at these locations in order to provide a removal pathway.

7.3. Floor Slabs

Satisfactory subgrade support for floor slabs supporting up to 100 psf floor loads can be obtained provided the floor slab subgrade is as described in Section 6.2 of this report. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Subgrade support for concrete slabs can be obtained from the medium stiff or stiffer native soils. We recommend that on-grade slabs be underlain by a minimum 6-inch-thick compacted crushed rock base section to reduce the potential for moisture migration into the slab and to provide structural support as noted below. The crushed rock base material should consist of Aggregate Base material as described Section 6.9 of this report. The material should be placed as recommended in Section 6.10.

If dry slabs are required (e.g., where moisture-sensitive adhesives are used to anchor carpet or tile to the slab), a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing. Load-bearing concrete slabs should be designed assuming a modulus of subgrade reaction (k) of 125 psi per inch. We estimate that concrete slabs constructed as recommended will settle less than ½ inch. We recommend that the floor slab subgrade be evaluated by proof-rolling prior to placing concrete.

7.4. Seismic Design

Parameters provided in Table 4 are based on the conditions encountered during our subsurface exploration program and the procedure outlined in the 2018 IBC, which references the 2016 Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers [ASCE] 7-16). Per ASCE 7-16 Section 11.4.8, a ground motion hazard analysis or site-specific response analysis is required to determine the design ground motions for structures on Site Class D sites with S_1 greater than or equal to 0.2g.

For this project, the site is classified as Site Class D with an S_1 value of 0.421g; therefore, the provision of 11.4.8 applies. The parameters listed in Table 4 below may be used to determine the design ground motions if Exception 2 of Section 11.4.8 of ASCE 7-16 is used. Using this exception, the seismic response coefficient (C_s) is determined by Equation (Eq.) (12.8-2) for values of $T \leq 1.5T_S$, and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$, where T represents the fundamental period of the structure and T_S=0.809 sec. If requested, we can complete a site-specific seismic response analysis, which might provide somewhat reduced seismic demands from the parameters in Table 4 and the requirements for using Exception 2 of Section 11.4.8 in ASCE 7-16. The reduced values will likely not be significant enough to warrant the additional cost of further evaluation if designing to 2018 IBC.

TABLE 4. MAPPED 2018 IBC SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value[1,2]
Site Class	D
Mapped Spectral Response Acceleration at Short Period (S_S)	0.841 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.421 g
Site Modified Peak Ground Acceleration (PGA_M)	0.473 g
Site Amplification Factor at 0.2 second period (F_a)	1.164
Site Amplification Factor at 1.0 second period (F_v)	1.879
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.652 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.527 g

Notes:

[1] Parameters developed based on Latitude 44. 9700373° and Longitude -123.0638420° using the ATC Hazards online tool.

[2] These values are only valid if the structural engineer utilizes Exception 2 of Section 11.4.8 (ASCE 7-16).

7.4.1. Liquefaction Potential

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Based on our boring logs at the project site, the groundwater is located at or below the extent of the depth of drilling of 21½ feet bgs, indicating that the soils encountered within our boring logs are not susceptible to liquefaction. Liquefaction is not considered a hazard for the project.

7.5. Retaining Walls

7.5.1. Drainage

Positive drainage is imperative behind retaining structures. This can be accomplished by providing a drainage zone behind the wall consisting of free-draining material and perforated pipes to collect and dispose the water. The drainage material should consist of Aggregate Base having less than 3 percent passing the U.S. No 200 sieve. The wall drainage zone should extend horizontally at least 18 inches from the back of the wall.

A perforated smooth-walled rigid drainpipe, having a minimum diameter of 4 inches, should be placed at the bottom of the drainage zone along the entire length of the wall, with the pipe invert at or below the base of the wall footing. The drainpipes should discharge to a tightline leading to an appropriate collection and disposal system. An adequate number of cleanouts should be incorporated into the design of the drains to provide access for regular maintenance. Roof downspouts, perimeter drains, or other types of drainage systems should not be connected to retaining wall drain systems.

7.5.2. Lateral Earth Pressures

Static and seismic lateral earth pressures were evaluated for use in design of the project. The evaluation assumed that: (1) backfill placed directly against walls will be free draining and meet the requirements of Section 6.9 of this report; (2) the backfill above the wall is level; and (3) that backfill placed within 2 feet of the wall is compacted using hand-operated equipment. The following subsections present recommended lateral earth pressures for use in design of conventional gravity retaining walls as well as braced walls such as the walls required for the below-grade parking floor for the proposed structure.

The recommended static lateral earth pressure coefficients and equivalent fluid pressures (triangular pressure distribution) for use in design of conventional gravity walls are presented in Table 5 for both imported structural fill as well as native on-site soil. Active earth pressure conditions assume that walls are not structurally restrained and are free to rotate. At-rest pressure conditions assume walls are structurally restrained from movement, or are to be designed for no lateral relaxation (rotational movement) as a function of elements that are being supported within a distance of twice the wall height from the top of the wall.

TABLE 5. STATIC EARTH PRESSURES FOR CONVENTIONAL GRAVITY WALLS

State of Stress	Earth Pressure Coefficient		Equivalent Fluid Pressure (psf)	
	Structural Fill	Native Soil	Structural Fill	Native Soil
At-Rest (K_O)	0.41	0.50	53*H	58*H
Active (K_A)	0.24	0.30	31*H	35*H
Passive (K_P)	3.0	2.5	390*H	288*H

Notes:

1. The magnitude of lateral earth pressure at a given height of wall is presented in units of pcf per foot of wall height (H), or psf. The wall height is the distance between the ground surface and the base of the wall. Walls should be designed to resist loads from surcharge and adjacent at-grade structures.
2. For drained earth pressures to be used for design, provisions for adequate drainage behind the wall must be included.
3. Compaction within 2 horizontal feet of the walls should be performed with lightweight, hand-operated equipment so that compaction-induced lateral stresses are limited.

For seismic active earth pressures for conventional gravity walls, use an additional dynamic increment force equal to 11*H psf, where H is the wall height, applied at 0.6*H from the exposed wall base (centroid location of seismic force).

The recommended pressures do not include the effects of surcharges from surface loads. If vehicles will be operated within one-half the height of the wall, a traffic surcharge should be added to the wall pressure. The traffic surcharge can be approximated by the equivalent weight of an additional 2 feet of backfill behind the wall. Additional surcharge loading conditions should also be considered on a case-by-case basis.

Retaining walls founded on native soil or structural fill extending to these materials may be designed using the allowable soil bearing values and lateral resistance values presented above in Section 7.1 of this report. We estimate settlement of retaining structures will be similar to the values previously presented for building foundations.

8.0 OTHER CONSIDERATIONS

8.1. Frost Penetration

The near-surface soils are slightly susceptible to frost heave. However, floor slabs are expected to bear on compacted granular fill and the foundations will be founded below the anticipated depth of frost penetration in the region, which is approximately 12 inches. The recommended exterior and interior footing embedment depths provided above should allow adequate frost protection.

8.2. Expansive Soils

Based on our laboratory test results and experience with similar soils in the area, we do not consider the soils encountered in our borings to be expansive.

9.0 PAVEMENT RECOMMENDATIONS

9.1. Dynamic Cone Penetrometer (DCP) Testing

We conducted DCP testing in general accordance with ASTM D 6951 to estimate the subgrade resilient modulus (M_R) at each test location. We recorded penetration depth of the cone versus hammer blow count and terminated testing when at a depth of approximately 5 feet bgs. The approximate locations of the explorations are presented in Figure 2. We plotted depth of penetration versus blow count and visually assessed portions of the data where slopes were relatively constant using the equation from the Oregon Department of Transportation (ODOT) Pavement Design Guide to estimate the moduli using a conversion coefficient, C_f = 0.35. Table 6 lists our estimate of the subgrade resilient modulus, and Appendix A (Figures A-14 and A-15) provides a summary of the field data.

TABLE 6. ESTIMATED SUBGRADE RESILIENT MODULI BASED ON DCP TESTING

Boring Number	Estimated Resilient Modulus (psi)
DCP-1	5,400
DCP-2	6,200

9.2. Asphalt Concrete (AC) Pavement Sections

Pavement recommendations are provided herein for paved parking and drive areas at the project site. Standards used for pavement design for asphalt pavement design are listed below:

- ODOT Pavement Design Guide (ODOT 2019)
- AASHTO Guide for Design of Pavement Structures (AASHTO 1993)

Our pavement recommendations assume that traffic at the site will consist of occasional truck traffic and passenger cars. We do not have specific information on the frequency and type of vehicles that will use the area; however, we have based our design analysis on traffic consisting of up to eight heavy trucks per day to account for the proposed commercial properties as well as delivery and service-type vehicles and passenger car traffic for pavement sections within drive areas, and passenger car traffic only for pavement sections within parking areas. Our pavement recommendations are based on the following assumptions:

- The on-site soil subgrade below proposed fill placed to raise site grades or below aggregate base sections has been prepared as described in Section 6.0 of this report, and observations indicate that subgrade is in a firm and unyielding condition.
- A resilient modulus of 20,000 psi was estimated for base rock prepared and compacted as recommended.
- A resilient modulus of 5,400 psi was estimated for firm in-place soils or structural fill placed on firm native soils.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviations of 90 percent and 0.49, respectively.
- Structural coefficients of 0.42 and 0.10 for the asphalt and base rock, respectively.

- A 20-year design life.

If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate. The recommended minimum pavement sections are provided in Table 7 below.

TABLE 7. MINIMUM PAVEMENT SECTIONS FOR ON-SITE ROADWAYS AND PARKING AREAS

	Minimum Asphalt Thickness (inches)	Minimum Base Thickness (inches)
Drive Lanes	4.0	5.0
Parking (cars only)	3.0	6.0

The aggregate base course should conform to Section 6.9.4 of this report and be compacted to at least 95 percent of the maximum dry density (MDD) determined in accordance with AASHTO T-180/ASTM Test Method D 1557.

The AC pavement should conform to Section 00745 of the most current edition of the ODOT Standard Specifications for Highway Construction. The Job Mix Formula should meet the requirements for a ½-inch Dense Graded Level 2 Mix. The AC should be PG 64-22 grade meeting the ODOT Standard Specifications for Asphalt Materials. AC pavement should be compacted to 91.0 percent at Maximum Theoretical Unit Weight (Rice Gravity) of AASHTO T-209.

The recommended pavement sections assume that final improvements surrounding the pavement will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the crushed base.

10.0 DESIGN REVIEW AND CONSTRUCTION SERVICES

Recommendations provided in this report are based on the assumptions and design information stated herein. We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GeoEngineers should be retained to review the geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in this report.

Satisfactory construction and earthwork performance depend to a large degree on quality of construction. Sufficient monitoring of the contractor’s activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

In order to continue as geotechnical engineer of record for the project, we recommend that GeoEngineers be retained to observe construction at the site to confirm that subsurface conditions are consistent with the site explorations, and to confirm that the intent of project plans and specifications relating to earthwork, pavement and foundation construction are being met.

11.0 LIMITATIONS

We have prepared this report for the exclusive use of Scott Martin Construction, Inc., and their authorized agents and/or regulatory agencies for the proposed Riverbend Phase 2 Development Project in Salem, Oregon.

This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in the area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix B titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

12.0 REFERENCES

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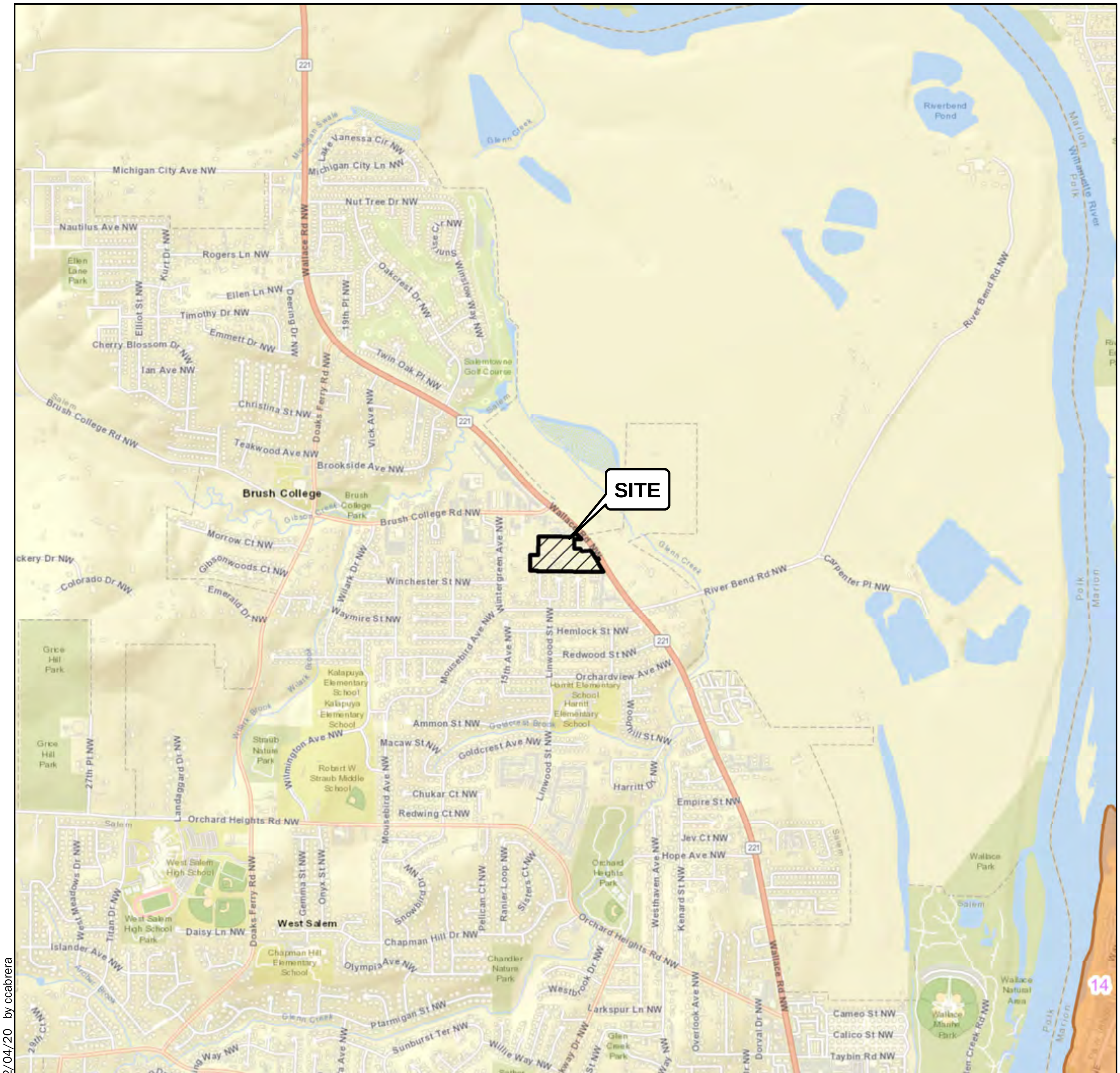
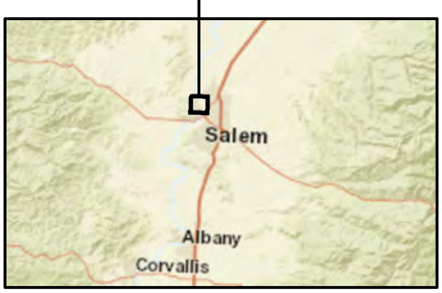
Oregon Department of Transportation (ODOT). 2019. ODOT Pavement Design Guide. Salem, Oregon.

FIGURES





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2,000 0 2,000
Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: ESRI World Street Map

Projection: NAD 1983 UTM Zone 10N

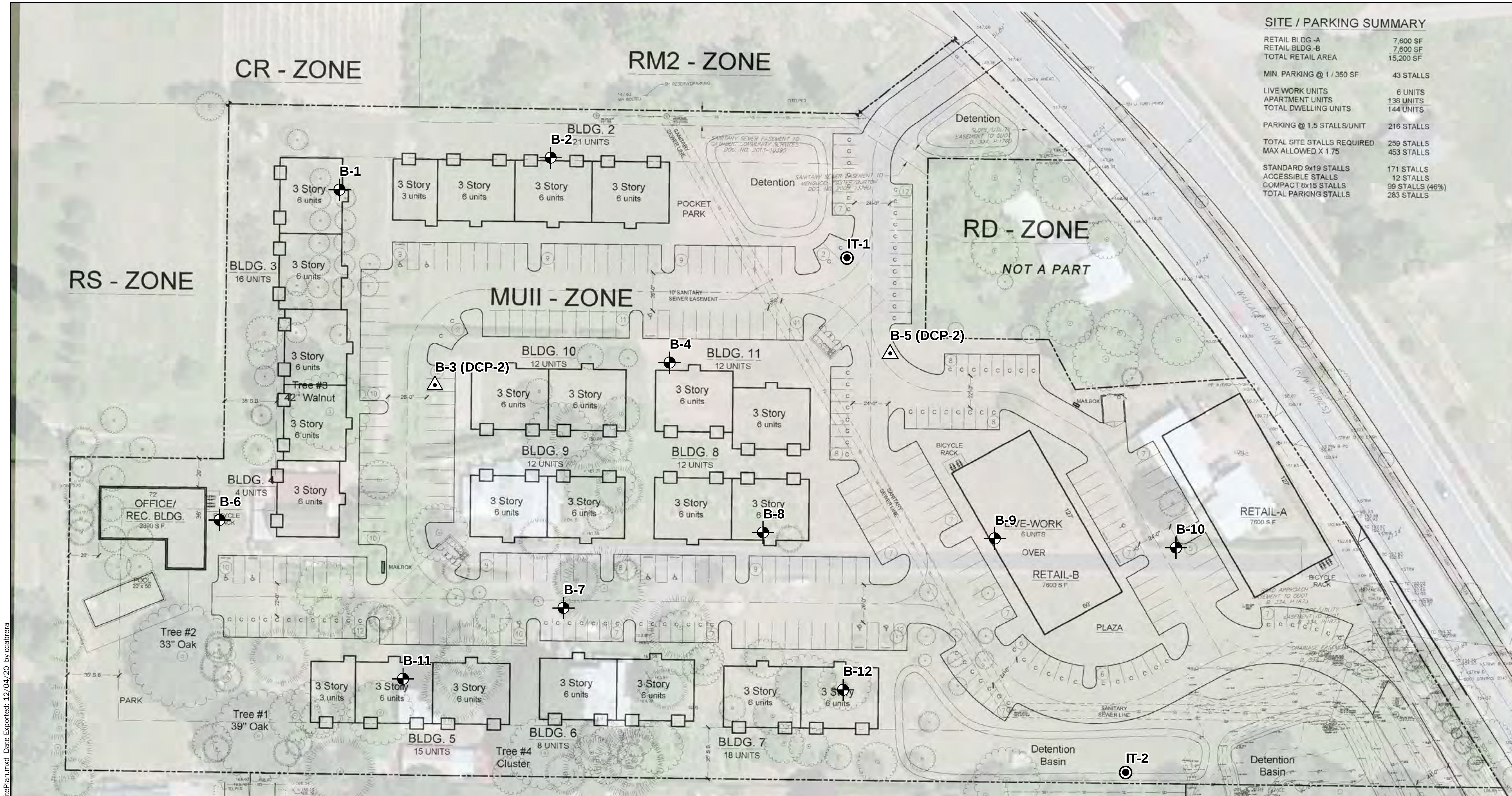
Vicinity Map

Riverbend Phase 2
Salem, Oregon

GEOENGINEERS

Figure 1

P:\23\23198001\GIS\MXD\2319800100_F01_VicinityMap.mxd Date Exported: 12/04/20 by ccabrera



Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: ESRI Clarity. Overlay from Lenity architecture, inc.

Projection: NAD 1983 UTM Zone 10N

Legend

- Boring Number and Approximate Location
- Boring & DCP Number and Approximate Location
- Infiltration Test Number and Approximate Location

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Site Plan

Riverbend Phase 2
Salem, Oregon

GEOENGINEERS

Figure 2

P:\23\23198001\GIS\MXD\2319800100_F02_SitePlan.mxd Date Exported: 12/04/20 by ccabrera

APPENDICES

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Soil and groundwater conditions at the site were explored on November 5 and 6, 2020, by completing twelve drilled borings, two infiltration tests, and two direct cone penetrometer (DCP) tests at the approximate locations shown in the Site Plan, Figures 2. The machine-drilled borings were advanced with a solid-stem auger using a trailer-mounted drill rig owned and operated by Dan Fischer Drilling.

The drilling was continuously monitored by an engineering geologist from our office who maintained detailed logs of subsurface exploration, visually classified the soil encountered, and obtained representative soil samples from the borings. Samples were collected using a 1-inch, inside-diameter, standard split spoon sampler and a 3-inch, inside-diameter, Dames and Moore (D&M) split spoon sampler. Samplers were driven into the soil using a rope and cathead 140-pound hammer, free-falling 30 inches on each blow. The number of blows required to drive the sampler each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the last two, 6-inch increments of penetration was reported on the boring logs as the ASTM International (ASTM) Standard Practices Test Method D 1556 standard penetration testing (SPT) N-value. The approximate N-values for D&M samples were converted to SPT N-values using the Lacroix-Horn Conversion [N(SPT) = (2*N1*W1*H1)/(175*D1*D1*L1), where N1 is the non-standard blowcount, W1 is the hammer weight in pounds (140), H1 is the hammer drop height in inches (30), D1 is the non-standard sampler outside diameter in inches (3.23), and L1 is the length of penetration in inches (12)].

Recovered soil samples were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Key to Exploration Logs, Figure A-1. Logs of the borings are presented in Figures A-2 through A-13. The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials or their characteristics change, although these changes might actually be gradual. Logs of DCP testing results are presented in Figures A-14 and A-15.

Laboratory Testing

Soil samples obtained from the explorations were visually classified in the field and in our laboratory using the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils, based on laboratory tests results. Moisture content tests were performed in general accordance with ASTM D 2216-05, moisture density tests of the ring samples were estimated in general accordance with ASTM Test Method D 7263, Percent Passing the No. 200 Sieve tests were performed in general accordance with ASTM D 1140, and Atterberg Limits tests were performed in general accordance with ASTM Test Method D 4318-05. Results of the laboratory testing are presented in the appropriate exploration logs at the respective sample depths and Atterberg Limits test results are presented in Figures A-16 through A-18.

SOIL CLASSIFICATION CHART

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

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

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Figure A-1

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft) 16.5	Logged By AB / Checked By	Driller Dan Fischer Excavating	Drilling Method Solid-stem Auger

Surface Elevation (ft) 180 / Vertical Datum NAVD88	Hammer Data Manual Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment Trailer Drill Rig
Latitude 44.970551 / Longitude -123.06486	System Decimal Degrees / Datum WGS84 (feet)	Groundwater not observed at time of exploration

Notes: D&M N-values reduced using Lacroix Horn Equation to approximate SPT N-values.

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
180	0						ML	Gray-red mottled silt (very stiff, moist) (Alluvium)			
		12	18		1 / AL				32		AL (LL=44, PI=13)
175	5	18	8*		2			Becomes brown and stiff			DD = 81 pcf
		18	5		3			Becomes medium stiff			
170	10	18	4*		4						
165	15	18	7		5			Increase in moisture content			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-1

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft)	21.5	Logged By AB Checked By	Driller	Dan Fischer Excavating	Drilling Method	Solid-stem Auger

Surface Elevation (ft) Vertical Datum	170 NAVD88	Hammer Data	Manual Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude Longitude	44.970636 -123.06422	System Datum	Decimal Degrees WGS84 (feet)	See "Remarks" section for groundwater observed	

Notes: D&M N-values reduced using Lacroix Horn Equation to approximate SPT N-values.

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0						ML	Brown-gray silt (very stiff, moist) (Alluvium)			
		15	22		1 / MC				27.4		
165	5	18	20*		2			Becomes brown			
		18	6		3			Becomes medium stiff			
160	10	18	5*		4						
							SM	Brown silt (soft, moist)			
155	15	18	4		5 / %F					89.5	
							SP	Dark gray-blackish sand (very dense, wet)			Groundwater inferred by soil sample moisture content
150	20	5	50/5.5"		6						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-2

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Figure A-3
Sheet 1 of 1

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft)	6.5	Logged By AB / Checked By	Driller Dan Fischer Excavating	Drilling Method	Solid-stem Auger

Surface Elevation (ft) / Vertical Datum	170 / NAVD88	Hammer Data	Manual Cathead / 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude / Longitude	44.970215 / -123.064588	System / Datum	Decimal Degrees / WGS84 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0		18	11		1		ML	Brown silt (stiff, moist) (Alluvium)			
			12	20		2			Becomes very stiff			
165	5		18	14		3 MC			Becomes stiff	27.1		

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-3

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft) 16.5	Logged By AB Checked By	Driller Dan Fischer Excavating	Drilling Method Solid-stem Auger

Surface Elevation (ft) 170 Vertical Datum NAVD88	Hammer Data: Manual Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment: Trailer Drill Rig
Latitude 44.970237 Longitude -123.063977	System: Decimal Degrees Datum: WGS84 (feet)	Groundwater not observed at time of exploration

Notes: D&M N-values reduced using Lacroix Horn Equation to approximate SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
170	0							ML	Gray-brown silt (very stiff, moist) (Alluvium)			
			15	16		1						
165	5		18	8*		2			Becomes brown and stiff			DD = 80 pcf
			18	9		3			Becomes medium stiff			
160	10		18	7*		4						
155	15		18	5		5						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-4

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft)	6.5	Logged By AB Checked By	Driller	Dan Fischer Excavating	Drilling Method	Solid-stem Auger

Surface Elevation (ft) Vertical Datum	170 NAVD88	Hammer Data	Manual Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude Longitude	44.970272 -123.063356	System Datum	Decimal Degrees WGS84 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Brown sandy silt (stiff, moist) (Alluvium)			
		☒	12	12		1						
		☒	18	13		2 MC				27.9		
165	5								Decreasing sand content			
		☒	14	15		3						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-5

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft)	21.5	Logged By AB Checked By	Driller Dan Fischer Excavating	Drilling Method	Solid-stem Auger

Surface Elevation (ft) Vertical Datum	170 NAVD88	Hammer Data	Manual Cathead 140 (lbs) / 30 (in) Drop
Drilling Equipment	Trailer Drill Rig		
Latitude Longitude	44.969998 -123.065159	System Datum	Decimal Degrees WGS84 (feet)
See "Remarks" section for groundwater observed			

Notes: D&M N-values reduced using Lacroix Horn Equation to approximate SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
170	0							ML	Brown-grayish silty sand (stiff, moist) (Alluvium)			
			15	15		1						
165	5		18	15		2 MC, AL			Becomes brown	33.6		AL (LL=43, PI=15)
			18	8*		3			No sand content			
160	10		18	12		4 MC				37.5		
155	15		18	3*		5			Becomes soft			
												Groundwater inferred by soil sample moisture content
150	20		16	4		6			Becomes medium stiff with trace fine sand			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-6

GEOENGINEERS

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft) 16.5	Logged By AB / Checked By	Driller Dan Fischer Excavating	Drilling Method Solid-stem Auger

Surface Elevation (ft) / Vertical Datum	170 / NAVD88	Hammer Data	Manual Cathead / 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude / Longitude	44.969778 / -123.064245	System / Datum	Decimal Degrees / WGS84 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Gray silt with trace sand (very stiff, moist) (Alluvium)			
			14	16		1						
165	5		18	13		2			Becomes brown with no sand and stiff			
			18	9		3						
160	10		18	5		4			Becomes medium stiff			Increase in moisture content
155	15		18	6		5			With occasional fine sand			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-7

GEOENGINEERS

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft)	16.5	Logged By AB Checked By	Driller Dan Fischer Excavating	Drilling Method Solid-stem Auger

Surface Elevation (ft) Vertical Datum	170 NAVD88	Hammer Data	Manual Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude Longitude	44.969961 -123.063714	System Datum	Decimal Degrees WGS84 (feet)	Groundwater not observed at time of exploration	

Notes: D&M N-values reduced using Lacroix Horn Equation to approximate SPT N-values.

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
170	0							ML	Brown silt (very stiff, moist) (Alluvium)			
			5	21		1						
165	5		18	10*		2			Becomes stiff			
			18	7		3 / MC			Becomes gray and medium stiff	36.6		
160	10		18	3*		4			Becomes soft			
155	15		18	10		5			Becomes stiff			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-8

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Figure A-9
Sheet 1 of 1

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft) 21.5	Logged By AB Checked By	Driller Dan Fischer Excavating	Drilling Method Solid-stem Auger

Surface Elevation (ft) / Vertical Datum	170 / NAVD88	Hammer Data	Manual Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude / Longitude	44.969938 / -123.063045	System Datum	Decimal Degrees WGS84 (feet)	See "Remarks" section for groundwater observed	

Notes: D&M N-values reduced using Lacroix Horn Equation to approximate SPT N-values.

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
170	0				ML	Gray-brown silt with trace sand (stiff, moist) (Alluvium)			
		18	14	1 / MC			27.9		
165	5	18	10*	2		Becomes brown with no sand, very stiff			DD = 82 pcf
		18	6	3 / MC			43.4		
						Becomes medium stiff			
160	10	18	4*	4					
					SM	Brown silt (soft, moist)			
155	15	18	3	5					
					SP	Gray poorly graded sand (dense, wet)			Groundwater inferred by soil sample moisture content
150	20	18	31	6					

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-9

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Figure A-10
Sheet 1 of 1

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft)	21.5	Logged By AB Checked By	Driller	Dan Fischer Excavating	Drilling Method	Solid-stem Auger

Surface Elevation (ft) Vertical Datum	170 NAVD88	Hammer Data	Manual Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude Longitude	44.969914 -123.062802	System Datum	Decimal Degrees WGS84 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval / Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Collected Sample	FIELD DATA: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0						ML	Gray silt with trace sand (very stiff, moist) (Alluvium)			
		14	16		1 / AL				24		AL (LL=41, PI=15)
165	5	18	16		2 / MC			Becomes brownish with no sand	32.9		
		18	5		3			Becomes medium stiff			
160	10	18	4		4						Increase in moisture content
							SM	Brown-gray silt (medium stiff, moist)			
155	15	18	7		5 / %F					90.7	
							GP	Gray poorly graded gravel with sand (dense, wet)			Groundwater inferred by soil sample moisture content
150	20	10	41		6						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-10

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft) 16.5	Logged By AB Checked By	Driller Dan Fischer Excavating	Drilling Method Solid-stem Auger

Surface Elevation (ft) Vertical Datum	180 NAVD88	Hammer Data	Manual Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude Longitude	44.969914 -123.062802	System Datum	Decimal Degrees WGS84 (feet)	Groundwater not observed at time of exploration	

Notes: D&M N-values reduced using Lacroix Horn Equation to approximate SPT N-values.

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
180	0							ML	Brown silt with sand (very stiff, moist) (Alluvium)			
			16	16		1						
175	5		16	16		2 / MC				26.3		
			18	21		3			With no sand content			
170	10		18	14		4			Becomes grayish-brown and stiff			
165	15		18	5*		5			Becomes medium stiff			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-11

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

Drilled	Start 11/5/2020	End 11/5/2020	Total Depth (ft) 16.5	Logged By AB / Checked By	Driller Dan Fischer Excavating	Drilling Method Solid-stem Auger

Surface Elevation (ft) / Vertical Datum	170 / NAVD88	Hammer Data	Manual Cathead / 140 (lbs) / 30 (in) Drop	Drilling Equipment	Trailer Drill Rig
Latitude / Longitude	44.969634 / -123.063522	System / Datum	Decimal Degrees / WGS84 (feet)	Groundwater not observed at time of exploration	

Notes: D&M N-values reduced using Lacroix Horn Equation to approximate SPT N-values.

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
170	0							ML	Gray-brown silt with trace sand (stiff, moist) (Alluvium)			
			15	13		1 / MC				24		
165	5		15	19		2			Becomes very stiff with no sand			
			18	8*		3			Becomes stiff			
160	10		18	20		4 / MC			Becomes brown and very stiff	41.6		
155	15		18	5		5			Becomes medium stiff			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on GPS (Rec). Vertical approximated based on GPS (Rec).

Log of Boring B-12

GeoEngineers

Project: Scott Martin Construction - Riverbend Phase 2
Project Location: Salem, Oregon
Project Number: 23198-001-00

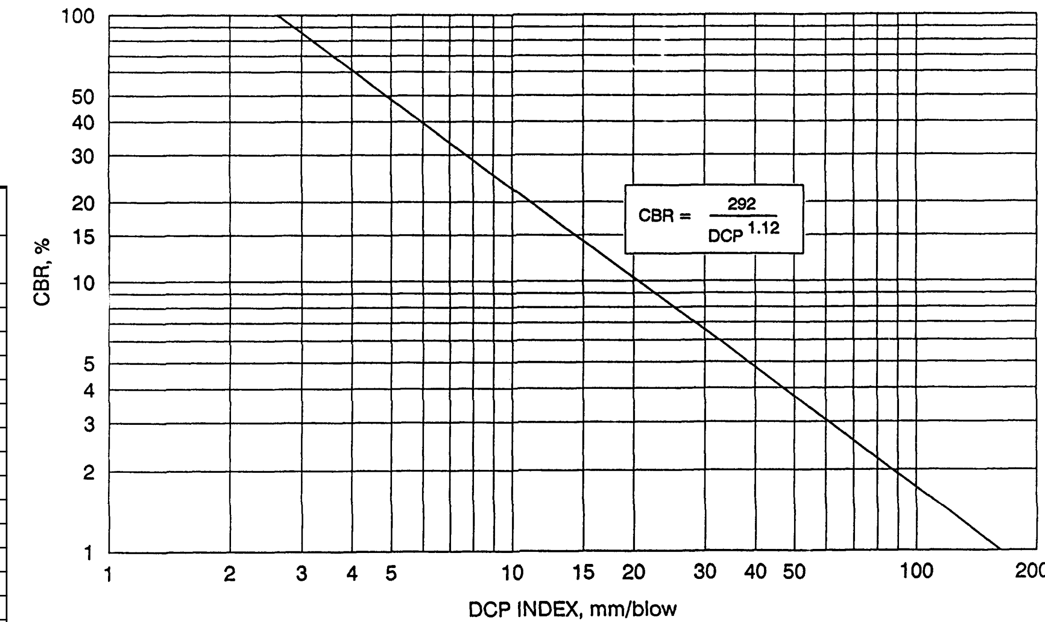
Location: Riverbend Ph. 2 - River Road, Salem
Depth to bottom: 61 152.5
Tester's Name: John Lawes
Tester's Company: GeoEngineers, Inc.
Notes: Driven from base of 24-inch sampler

Date: 11/5/2020
Dimension: 2"
Tester's Contact No: 971-409-7390

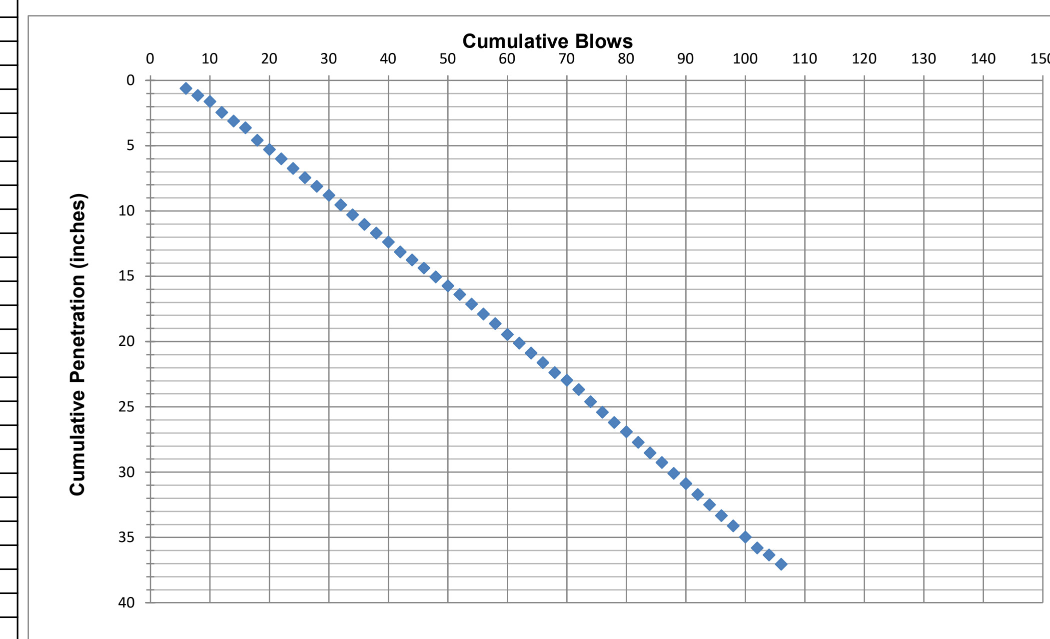
Test Hole Number: B-3 (DCP-1)
Test Method: Dynamic Cone Penetration
GeoEngineers Job: 23198-001-00
Project Name Riverbend Ph.2

Depth, feet		Soil Texture
0-6.5	Brown to gray-brown silt	Medium stiff

Test increment	Number of blows	Cumulative blows	Depth below base of S-1	Penetration per increment	Cumulative penetration	Cummulative Penetration	Penetration per blow set	Penetration per blow	Hammer blow factor	DCP Index	DCP Index	CBR	M_R
#	#	#	(in)	(mm)	(mm)	(in)	(in)	(in)	1 for 8-kg 2 for 4.6-kg hammer	in/blow	mm/blow	%	psi
1	2	6	0.6	15.0	15.0	0.6	0.6	0.30	2	0.59	15.00	14	5967
2	2	8	1.1	14.0	29.0	1.1	0.6	0.28	2	0.55	14.00	15	6130
3	2	10	1.6	12.0	41.0	1.6	0.5	0.24	2	0.47	12.00	18	6510
4	2	12	2.4	21.0	62.0	2.4	0.8	0.41	2	0.83	21.00	10	5234
5	2	14	3.1	17.0	79.0	3.1	0.7	0.33	2	0.67	17.00	12	5683
6	2	16	3.6	13.0	92.0	3.6	0.5	0.26	2	0.51	13.00	17	6310
7	2	18	4.6	24.0	116.0	4.6	0.9	0.47	2	0.94	24.00	8	4968
8	2	20	5.3	18.0	134.0	5.3	0.7	0.35	2	0.71	18.00	11	5558
9	2	22	6.0	18.0	152.0	6.0	0.7	0.35	2	0.71	18.00	11	5558
10	2	24	6.7	19.0	171.0	6.7	0.7	0.37	2	0.75	19.00	11	5442
11	2	26	7.4	18.0	189.0	7.4	0.7	0.35	2	0.71	18.00	11	5558
12	2	28	8.1	17.0	206.0	8.1	0.7	0.33	2	0.67	17.00	12	5683
13	2	30	8.8	17.0	223.0	8.8	0.7	0.33	2	0.67	17.00	12	5683
14	2	32	9.5	19.0	242.0	9.5	0.7	0.37	2	0.75	19.00	11	5442
15	2	34	10.3	19.0	261.0	10.3	0.7	0.37	2	0.75	19.00	11	5442
16	2	36	11.0	19.0	280.0	11.0	0.7	0.37	2	0.75	19.00	11	5442
17	2	38	11.7	17.0	297.0	11.7	0.7	0.33	2	0.67	17.00	12	5683
18	2	40	12.4	17.0	314.0	12.4	0.7	0.33	2	0.67	17.00	12	5683
19	2	42	13.1	19.0	333.0	13.1	0.7	0.37	2	0.75	19.00	11	5442
20	2	44	13.7	16.0	349.0	13.7	0.6	0.31	2	0.63	16.00	13	5819
21	2	46	14.4	16.0	365.0	14.4	0.6	0.31	2	0.63	16.00	13	5819
22	2	48	15.0	17.0	382.0	15.0	0.7	0.33	2	0.67	17.00	12	5683
23	2	50	15.7	17.0	399.0	15.7	0.7	0.33	2	0.67	17.00	12	5683
24	2	52	16.4	17.0	416.0	16.4	0.7	0.33	2	0.67	17.00	12	5683
25	2	54	17.1	19.0	435.0	17.1	0.7	0.37	2	0.75	19.00	11	5442
26	2	56	17.9	19.0	454.0	17.9	0.7	0.37	2	0.75	19.00	11	5442
27	2	58	18.6	19.0	473.0	18.6	0.7	0.37	2	0.75	19.00	11	5442
28	2	60	19.4	21.0	494.0	19.4	0.8	0.41	2	0.83	21.00	10	5234
29	2	62	20.1	17.0	511.0	20.1	0.7	0.33	2	0.67	17.00	12	5683
30	2	64	20.9	19.0	530.0	20.9	0.7	0.37	2	0.75	19.00	11	5442
31	2	66	21.6	19.0	549.0	21.6	0.7	0.37	2	0.75	19.00	11	5442
32	2	68	22.4	19.0	568.0	22.4	0.7	0.37	2	0.75	19.00	11	5442
33	2	70	23.0	15.0	583.0	23.0	0.6	0.30	2	0.59	15.00	14	5967
34	2	72	23.7	18.0	601.0	23.7	0.7	0.35	2	0.71	18.00	11	5558
35	2	74	24.6	24.0	625.0	24.6	0.9	0.47	2	0.94	24.00	8	4968
36	2	76	25.4	20.0	645.0	25.4	0.8	0.39	2	0.79	20.00	10	5334
37	2	78	26.2	20.0	665.0	26.2	0.8	0.39	2	0.79	20.00	10	5334
38	2	80	26.9	18.0	683.0	26.9	0.7	0.35	2	0.71	18.00	11	5558
39	2	82	27.7	21.0	704.0	27.7	0.8	0.41	2	0.83	21.00	10	5234
40	2	84	28.5	20.0	724.0	28.5	0.8	0.39	2	0.79	20.00	10	5334
41	2	86	29.3	19.0	743.0	29.3	0.7	0.37	2	0.75	19.00	11	5442
42	2	88	30.1	21.0	764.0	30.1	0.8	0.41	2	0.83	21.00	10	5234
43	2	90	30.9	20.0	784.0	30.9	0.8	0.39	2	0.79	20.00	10	5334
44	2	92	31.7	21.0	805.0	31.7	0.8	0.41	2	0.83	21.00	10	5234
45	2	94	32.5	20.0	825.0	32.5	0.8	0.39	2	0.79	20.00	10	5334
46	2	96	33.3	21.0	846.0	33.3	0.8	0.41	2	0.83	21.00	10	5234
47	2	98	34.1	20.0	866.0	34.1	0.8	0.39	2	0.79	20.00	10	5334
48	2	100	35.0	22.0	888.0	35.0	0.9	0.43	2	0.87	22.00	9	5140
49	2	102	35.8	21.0	909.0	35.8	0.8	0.41	2	0.83	21.00	10	5234
50	2	104	36.3	14.0	923.0	36.3	0.6	0.28	2	0.55	14.00	15	6130
51	2	106	37.0	18.0	941.0	37.0	0.7	0.35	2	0.71	18.00	11	5558



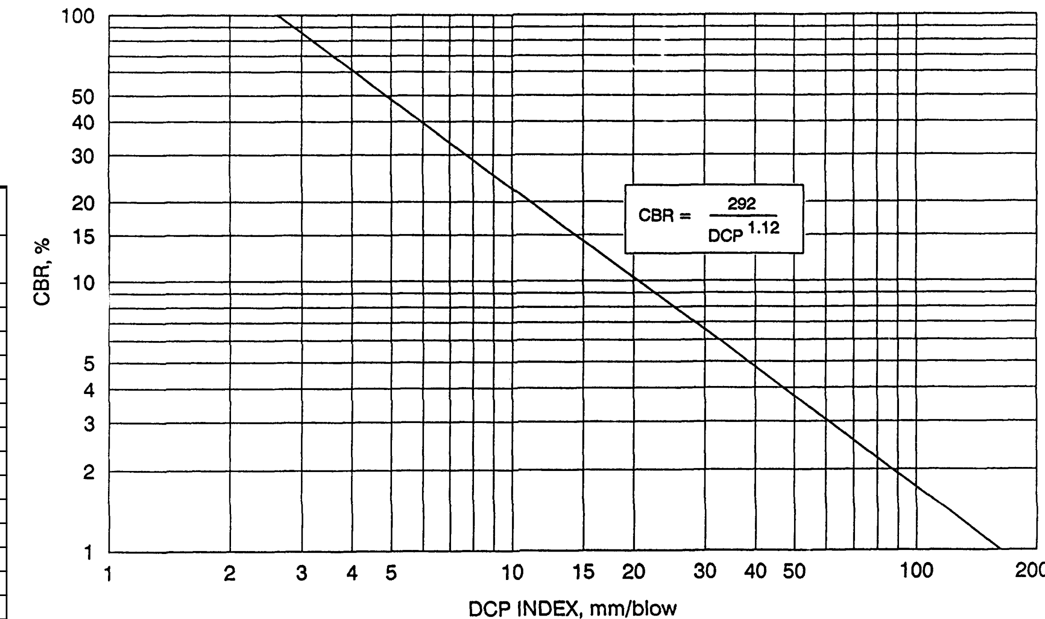
(after Webster et al., 1992)
Webster, S. L., Grau, R. H., and Williams, T. P. (1992). Description and application of dual mass dynamic cone penetrometer. Department of the Army Waterways Equipment Station, No. GL-92-3.



Location:	Riverbend Ph. 2 - River Road, Salem	Date:	11/5/2020	Test Hole Number:	B-5 (DCP-2)
Depth to bottom:	61 152.5	Dimension:	2"	Test Method:	Dynamic Cone Penetration
Tester's Name:	John Lawes			GeoEngineers Job:	23198-001-00
Tester's Company:	GeoEngineers, Inc.	Tester's Contact No:	971-409-7390	Project Name	Riverbend Ph.2
Notes:	Driven from base of 24-inch sampler				

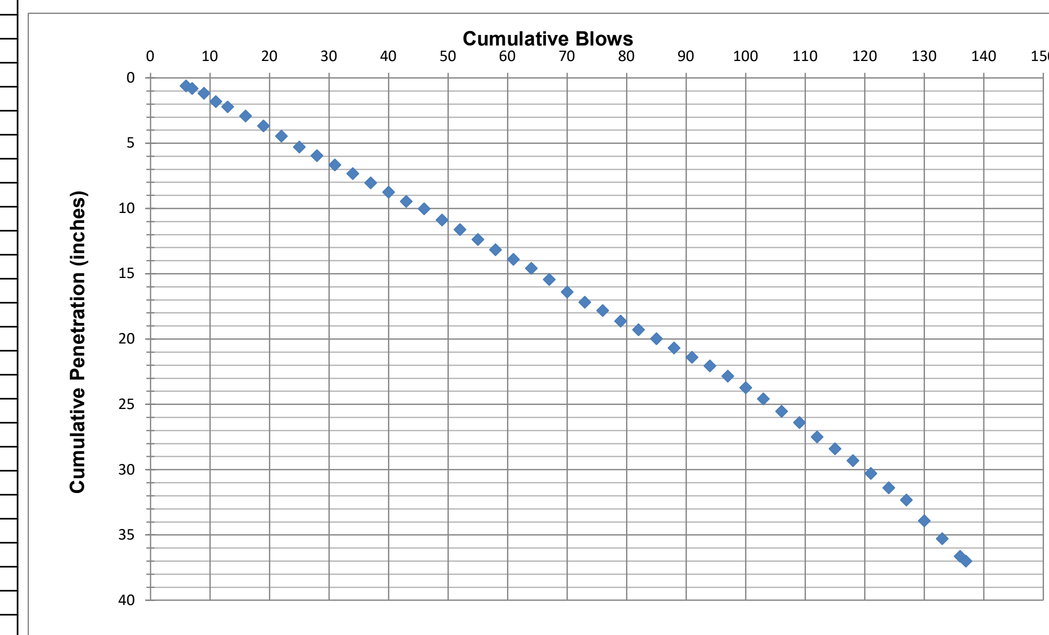
Depth, feet		Soil Texture
0-6.5	Brown to gray-brown silt	Medium stiff

Test increment	Number of blows	Cumulative blows	Depth below base of S-1	Penetration per increment	Cumulative penetration	Cummulative Penetration	Penetration per blow set	Penetration per blow	Hammer blow factor	DCP Index	DCP Index	CBR	M_R
#	#	#	(in)	(mm)	(mm)	(in)	(in)	(in)	1 for 8-kg 2 for 4.6-kg hammer	in/blow	mm/blow	%	psi
1	1	6	0.6	15.0	15.0	0.6	0.6	0.59	2	1.18	30.00	6	4554
2	1	7	0.8	5.0	20.0	0.8	0.2	0.20	2	0.39	10.00	22	6990
3	2	9	1.2	10.0	30.0	1.2	0.4	0.20	2	0.39	10.00	22	6990
4	2	11	1.8	16.0	46.0	1.8	0.6	0.31	2	0.63	16.00	13	5819
5	2	13	2.2	10.0	56.0	2.2	0.4	0.20	2	0.39	10.00	22	6990
6	3	16	2.9	18.0	74.0	2.9	0.7	0.24	2	0.47	12.00	18	6510
7	3	19	3.7	19.0	93.0	3.7	0.7	0.25	2	0.50	12.67	17	6374
8	3	22	4.4	20.0	113.0	4.4	0.8	0.26	2	0.52	13.33	16	6248
9	3	25	5.3	21.0	134.0	5.3	0.8	0.28	2	0.55	14.00	15	6130
10	3	28	5.9	17.0	151.0	5.9	0.7	0.22	2	0.45	11.33	19	6657
11	3	31	6.7	18.0	169.0	6.7	0.7	0.24	2	0.47	12.00	18	6510
12	3	34	7.3	17.0	186.0	7.3	0.7	0.22	2	0.45	11.33	19	6657
13	3	37	8.0	18.0	204.0	8.0	0.7	0.24	2	0.47	12.00	18	6510
14	3	40	8.7	18.0	222.0	8.7	0.7	0.24	2	0.47	12.00	18	6510
15	3	43	9.4	18.0	240.0	9.4	0.7	0.24	2	0.47	12.00	18	6510
16	3	46	10.0	14.0	254.0	10.0	0.6	0.18	2	0.37	9.33	24	7180
17	3	49	10.9	22.0	276.0	10.9	0.9	0.29	2	0.58	14.67	14	6020
18	3	52	11.6	19.0	295.0	11.6	0.7	0.25	2	0.50	12.67	17	6374
19	3	55	12.4	19.0	314.0	12.4	0.7	0.25	2	0.50	12.67	17	6374
20	3	58	13.1	20.0	334.0	13.1	0.8	0.26	2	0.52	13.33	16	6248
21	3	61	13.9	19.0	353.0	13.9	0.7	0.25	2	0.50	12.67	17	6374
22	3	64	14.6	17.0	370.0	14.6	0.7	0.22	2	0.45	11.33	19	6657
23	3	67	15.4	22.0	392.0	15.4	0.9	0.29	2	0.58	14.67	14	6020
24	3	70	16.4	24.0	416.0	16.4	0.9	0.31	2	0.63	16.00	13	5819
25	3	73	17.2	20.0	436.0	17.2	0.8	0.26	2	0.52	13.33	16	6248
26	3	76	17.8	16.0	452.0	17.8	0.6	0.21	2	0.42	10.67	21	6816
27	3	79	18.6	21.0	473.0	18.6	0.8	0.28	2	0.55	14.00	15	6130
28	3	82	19.3	17.0	490.0	19.3	0.7	0.22	2	0.45	11.33	19	6657
29	3	85	20.0	17.0	507.0	20.0	0.7	0.22	2	0.45	11.33	19	6657
30	3	88	20.7	18.0	525.0	20.7	0.7	0.24	2	0.47	12.00	18	6510
31	3	91	21.4	18.0	543.0	21.4	0.7	0.24	2	0.47	12.00	18	6510
32	3	94	22.0	17.0	560.0	22.0	0.7	0.22	2	0.45	11.33	19	6657
33	3	97	22.8	20.0	580.0	22.8	0.8	0.26	2	0.52	13.33	16	6248
34	3	100	23.7	22.0	602.0	23.7	0.9	0.29	2	0.58	14.67	14	6020
35	3	103	24.6	22.0	624.0	24.6	0.9	0.29	2	0.58	14.67	14	6020
36	3	106	25.5	24.0	648.0	25.5	0.9	0.31	2	0.63	16.00	13	5819
37	3	109	26.4	22.0	670.0	26.4	0.9	0.29	2	0.58	14.67	14	6020
38	3	112	27.5	28.0	698.0	27.5	1.1	0.37	2	0.73	18.67	11	5480
39	3	115	28.4	23.0	721.0	28.4	0.9	0.30	2	0.60	15.33	14	5917
40	3	118	29.3	23.0	744.0	29.3	0.9	0.30	2	0.60	15.33	14	5917
41	3	121	30.3	25.0	769.0	30.3	1.0	0.33	2	0.66	16.67	13	5727
42	3	124	31.4	28.0	797.0	31.4	1.1	0.37	2	0.73	18.67	11	5480
43	3	127	32.3	24.0	821.0	32.3	0.9	0.31	2	0.63	16.00	13	5819
44	3	130	33.9	40.0	861.0	33.9	1.6	0.52	2	1.05	26.67	7	4768
45	3	133	35.3	35.0	896.0	35.3	1.4	0.46	2	0.92	23.33	9	5023
46	3	136	36.6	34.0	930.0	36.6	1.3	0.45	2	0.89	22.67	9	5080
47	1	137	37.0	10.0	940.0	37.0	0.4	0.39	2	0.79	20.00	10	5334

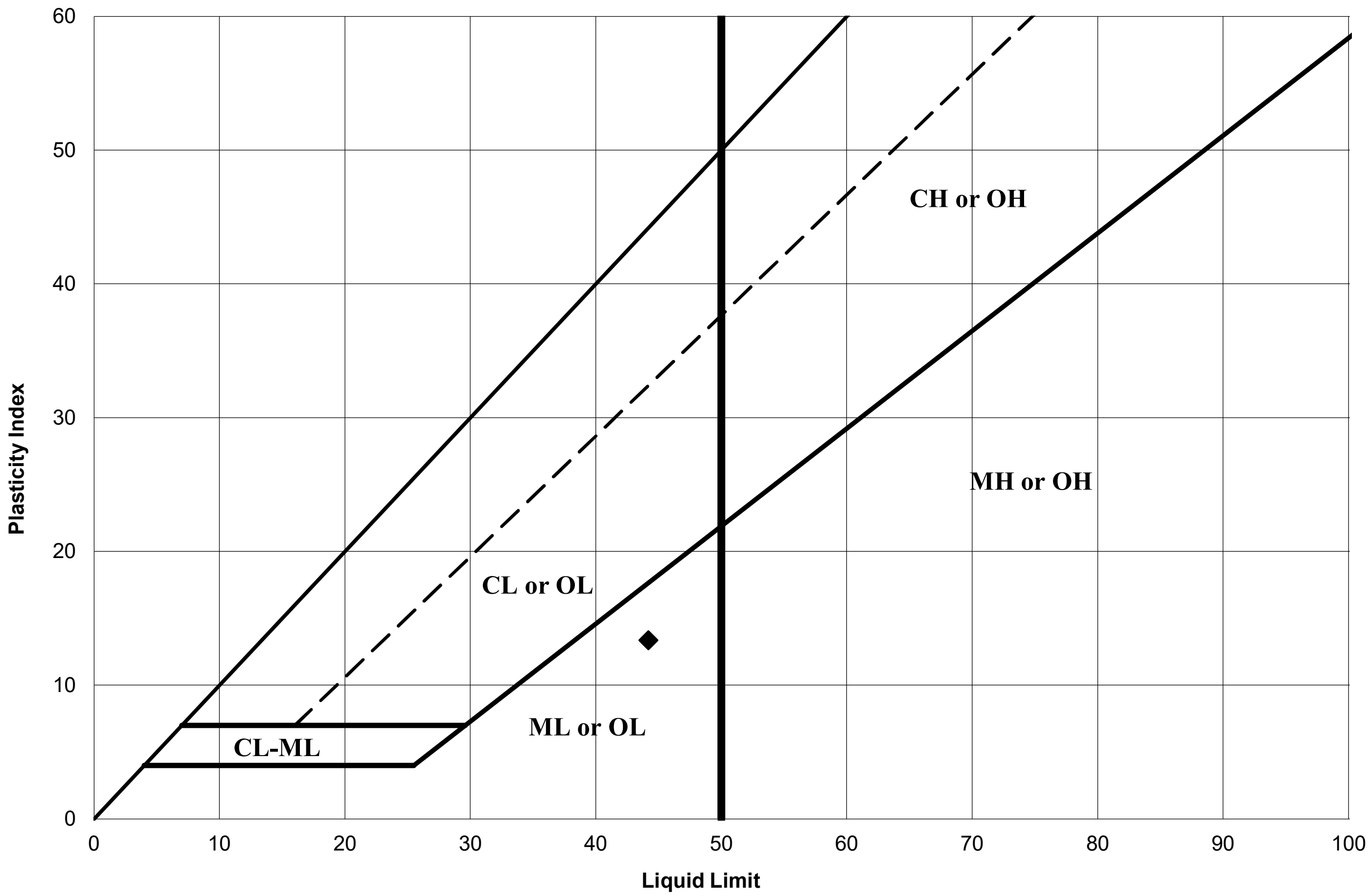


(after Webster et al., 1992)

Webster, S. L., Grau, R. H., and Williams, T. P. (1992). Description and application of dual mass dynamic cone penetrometer. Department of the Army Waterways Equipment Station, No. GL-92-3.



Project:	Riverbend Ph.2		
Project No.	23198-001-00	**Date:**	12/05/20
Boring/TP No.	B-1	**Tested By:**	AB/JL
Sample No./Depth:	1 at 2.5	**Checked By:**	JL
USCS Classification:	ML	**PA/PM:**	JCV



Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Description
32	44	31	13	**ML**	**Brown SILT**

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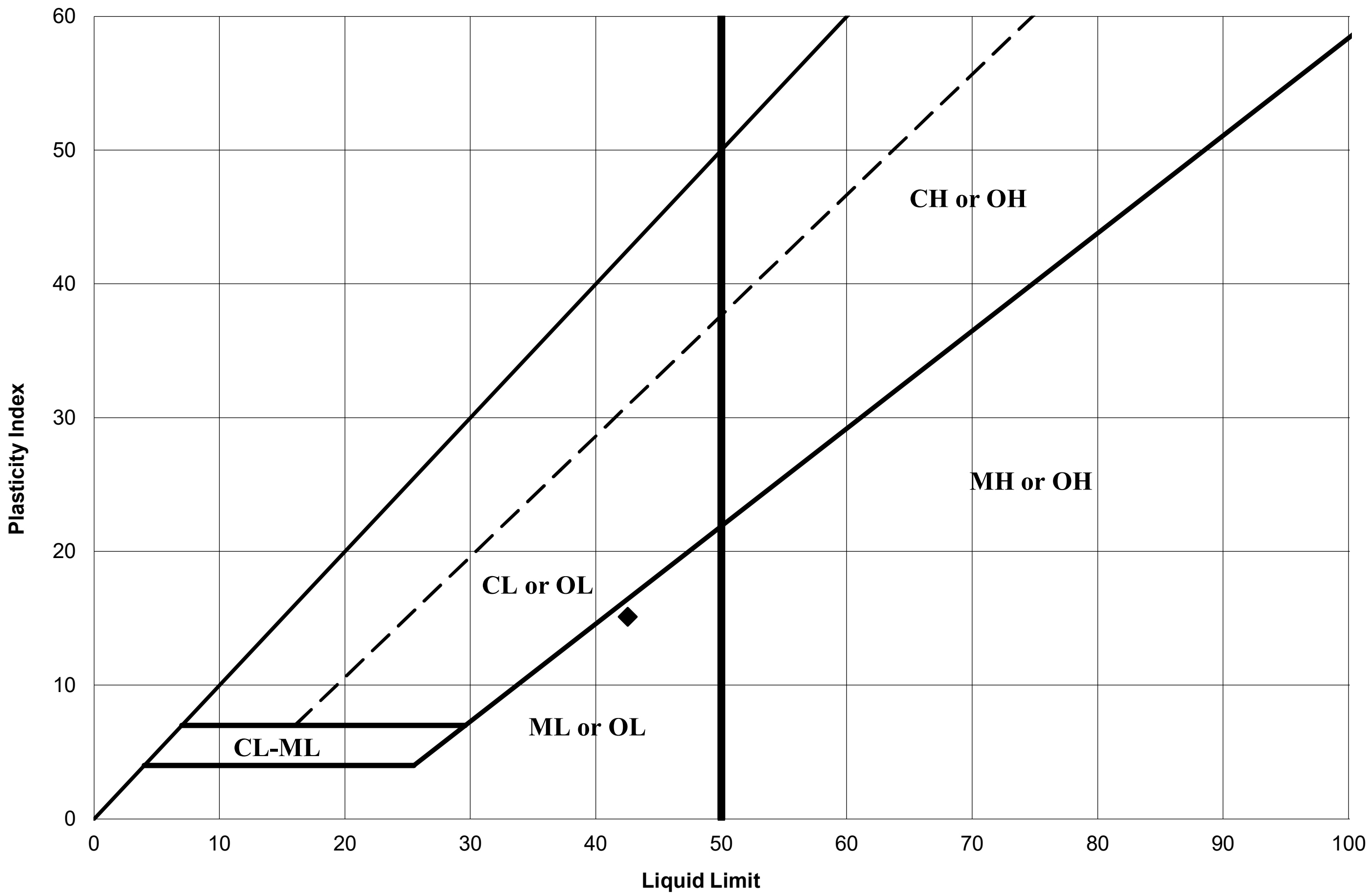
4000 Kruse Way Place, Lake Oswego, OR 97035

Atterberg Limits ASTM D 4318-05

Figure A-16

Project:	Riverbend Ph.2		
Project No.	23198-001-00	**Date:**	12/05/20
Boring/TP No.	B-6	**Tested By:**	AB/JL
Sample No./Depth:	2 at 5ft	**Checked By:**	JL
USCS Classification:	ML	**PA/PM:**	JCV



Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Description
31	43	27	15	**ML**	**Brown SILT**

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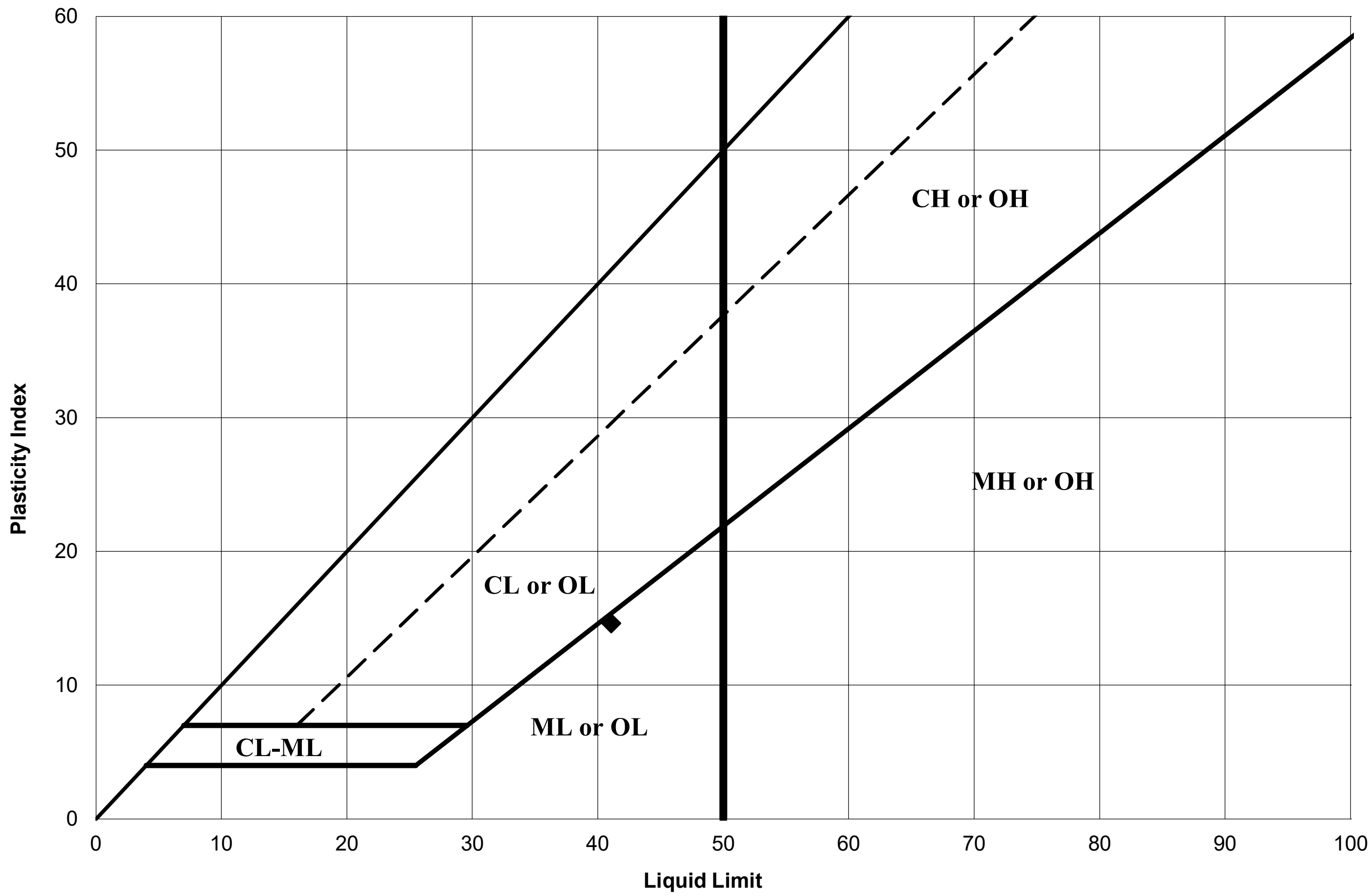
4000 Kruse Way Place, Lake Oswego, OR 97035

Atterberg Limits ASTM D 4318-05

Figure A-17

Project:	Riverbend Ph.2		
Project No.	23198-001-00	**Date:**	12/05/20
Boring/TP No.	B-10	**Tested By:**	AB/JL
Sample No./Depth:	1 at 2.5ft	**Checked By:**	JL
USCS Classification:	ML	**PA/PM:**	JCV



Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Description
24	41	26	15	**ML**	**Brown SILT**

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Atterberg Limits ASTM D 4318-05

Figure A-18

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for Scott Martin Construction, Inc., and their agents for the Project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with Scott Martin Construction, Inc. dated October 12, 2020, and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed Riverbend Phase 2 Development Project in Salem, Oregon. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns Are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted, or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

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