6502

PRELIMINARY DRAINAGE REPORT
FOR

Devon Estates
Salem, Oregon

Prepared For:
HSF Development, LLC
3245 Boone Road SE
Salem, Oregon 97317

February 17, 2021

MULTI/TECH
ENGINEERING SERVICES, INC.

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Contents

INTRODUCTION

The Devon Estates development is a proposed multi-phased subdivision that was previously approved under the City of Salem Planning Case No. SUB19-05. The site is located south of Sahalee Court SE and west of Devon Avenue SE. The parcel of land to be developed is Tax Lot 300 of Marion County Assessor's Map 08 3W 22C. A vicinity map and supporting maps are in Appendix A of this report. An aerial image of the site can be seen below.

Project Site

Green Stormwater Infrastructure (GSI) to the Maximum Extent Feasible (MEF) is being used for the new developed areas per City of Salem Administrative Rules, Chapter 109, Division 004, Stormwater System, Appendix 4E and Ordinance No. 8-20 (Standards). Stormwater facilities will be constructed to meet the City of Salem standards.

EXISTING CONDITIONS

The entire 19.6-acre site is generally rectangular in the shape. Surface conditions consists of grass, brush and minimal trees. There are no identified wetlands, streams or sensitive areas located on the property.

A topographical high point is located on the southerly side of the site. Drainage from this high point flows westerly and easterly. The maximum relief is approximately 112-feet with a high point elevation of 651. The abutting properties are zoned single family residential with nearby public improvements that include minimal storm water conveyance systems. Appendix A contains multiple maps of the site.

Soils

The Natural Resources Conservation Service (NRCS) Soil Resource Report for Marion County was used to determine a Hydrological Soil Group classification for runoff calculations. The report identifies the site soils to be Jory, Nekia and Salkum soils. The predominate soils are in the hydrologic soil group C. The report is in Appendix B.

Infiltration

Infiltration testing was performed at the site to determine percolation rate of the soil. Test results recommend design infiltration rates below 0.5 inches per hour. Appendix B contains the geotechnical report with recommended infiltration rates.

Water Quality Methodology

Because of the poor percolation rates of the soils and natural steep slopes located on the site, green stormwater facilities are designed as volume control facilities with filtration. Because of natural steep grades, a 1-acre portion of Lone Oak Drive will treat runoff via Manufactured Treatment Technology devices; Contech Stormwater Solutions storm filters.

Water Quality Analysis

Water quality flow rates were calculated with HydroCAD 10.00. The SCS TR-20 Unit Hydrograph method will be used to generate the hydrographs. A Type 1A storm and a 24-hour rainfall depth of 1.38 inches per hour will be used to determine the water quality flow rate.

Manufactured Treatment Technology Design

The manufactured treatment devices will be CONTECH Stormwater Solutions storm filter catch basins using ZPG media. The system is designed in accordance with the manufacturer's recommendations per City of Salem design standards.

STORMWATER QUANTITY ANALYSIS

Stormwater quantity (Flow Control) is being handled by on-site detention and within Lone Oak Road. Runoff from the development is being routed to the facilities that ultimately controls runoff to pre-developed flow rates.

Per the standards, one-half of the post development peak runoff rate of the two-year storm must be equal to or less than one-half of the peak runoff rate of the pre-developed two-year, 24-hour storm. This also applies to the 10-year through 100-year, 24-hour storm events. Because of natural steep slopes and other factors, runoff from the Lone Oak facility will be controlled to the 10-year through 100-year pre-developed rates only. A design exception was previously approved and will be resubmitted for approval.

The pre-developed flow rates were calculated using HydroCAD 10.00. Table 1 below lists the 24-hour rainfall depths used in the analysis of each storm event. Please note that the 2-year event was halved and then analyzed.

Table 1

Storm Event	24-hour Rainfall Depth (in)
Half of 2-year	1.1
10-year	3.2
25-year	3.6
100-year	4.4

For the pre-developed conditions, a time of concentration of 22.2 minutes was calculated for Basin 1 and 21.6 minutes for Basin #2. The time of concentration data is in Appendix C. The calculations are incorporated in the HydroCAD output located in Appendix D. The entire area was classified as "City of Salem Pre-Development, HSG C" with a Curve Number (CN) of 72. A pre-developed basin map is in Appendix A.

The SCS TR-20 Unit Hydrograph method was used to generate the hydrographs. A Type 1A rainfall distribution was used with the above rainfall depths. Table 2A and 2B below identifies the allowable pre-developed release rates for each storm event.

Table 2A

Storm Event	Basin #1 Allowable Release Rate (cfs)	Off-site Allowable Release Rate (cfs)	Control MH#1 Allowable Release Rate (cfs)	Off-site North Allowable Release Rate (cfs)	Control MH#2 Allowable Release Rate (cfs)
Half of 2-year	0.03	0.01	0.04	N/A	N/A
10-year	1.49	0.52	2.01	0.12	2.13
25-year	2.13	0.75	2.88	0.17	3.05
100-year	3.56	1.25	4.81	0.29	5.10

Table 2B

Storm Event	Basin #2 Allowable Release Rate (cfs)
Half of 2-year	0.03
10-year	1.24
25-year	1.77
100-year	2.96

The post-developed site flow rates were calculated using HydroCAD 10.00. A time of concentration of 10 minutes was assumed for all basins. The calculations are incorporated in the HydroCAD output located in Appendix D. Each on-site basin was classified as 60 percent "Impervious, HSG C" with a CN of 98 and 40 percent “> 75% Grass cover, HSG C” with a CN of 74. This was based on code setback requirements and City street section standards. Off-site areas contributing to the development’s drainage system were classified as "City of Salem Pre-Development, HSG C" with a Curve Number (CN) of 72 or “Impervious, HSG C” with a CN of 98.

The post-developed Lone Oak flow rates were also calculated using HydroCAD 10.00. A time of concentration of 5 minutes was assumed for the basin. The basin was classified as 90 percent "Impervious, HSG C" with a CN of 98 and 10 percent “> 75% Grass cover, HSG C” with a CN of 74. Table 3 below lists the CN values for the developed basin areas. A developed basin map is in Appendix A.

Table 3

Basin	Impervious Area (Ac) CN = 98	Predeveloped Area (AC) CN = 72	Landscape Area (Ac) CN = 74	TOTAL Area (Ac)	Composite CN
Basin 1	6.00	N/A	4.00	10.00	88
Basin 1B	0.35	N/A	0.36	0.71	86
Basin 2	5.17	N/A	3.45	8.62	88
Basin 2B	0.07	N/A	0.17	0.24	81
Off-site	0.50	3.26	N/A	3.76	75
Off-Site North	0.76	N/A	0.10	0.86	95

A 0.71-acre portion of Basin 1, developed Basin 1B, will not drain into the detention pond that will serve the westerly side of the development. To compensate for this uncontrolled release, allowable release rates have been reduced in portion of the undetained flow. Likewise, the same approach was also taken in Basin 2. Table 4A & 4B provides an overview.

Table 4A

Storm Event	Control MH#1 Allowable Release Rate (cfs)	Undetained Release Rate (cfs)	Adjusted Allowable Release Rate (cfs)	Off-site North Allowable Release Rate (cfs)	Control MH#2 Allowable Release Rate (cfs)
Half of 2-year	0.04	0.02	0.02	N/A	N/A
10-year	2.01	0.32	1.69	0.12	1.81
25-year	2.88	0.39	2.49	0.17	2.66
100-year	4.81	0.53	4.28	0.29	4.57

Table 4B

Storm Event	Basin #2 Allowable Release Rate (cfs)	Undetained Release Rate (cfs)	Adjusted Allowable Release Rate (cfs)
Half of 2-year	0.03	0.003	0.03
10-year	1.24	0.08	1.16
25-year	1.77	0.10	1.67
100-year	2.96	0.15	2.81

DETENTION SYSTEM PHASE #1

In the detention analysis for this phase, the 10.0-acre site was considered a single basin draining into the detention pond. A basin map is in Appendix A. Site grading and conveyance pipe will direct stormwater runoff to the detention system. It should be noted that the Basin #1 pond has a capacity to detain 46,700 cubic feet of water. This exceeds the required detention volume of 33,740 cubic feet for the developed portion.

Based on the above design parameters, the half of the 2-year through the 100-year pre-developed release rates are controlled at 0.02, 1.52, 2.14 and 3.90 cfs. The release rates and detention requirements were generated from the HydroCAD software, which can be seen in Appendix D. Table 5 below summarizes the requirements for the storm events.

Table 5

Storm Event	Basin #1 Release Rate (cfs)	Required Detention Volume (ft^3)	Provided Detention Volume (ft^3)
Half of 2-year	0.02	11,150	46,700
10-year	1.52	25,225	46,700
25-year	2.14	27,900	46,700
100-year	3.90	33,740	46,700

(Basin #1 Pond Detention Summary)

Flow control is achieved with multiple orifices in a standard City of Salem control structure. The sizing of the orifice will use the standard orifice equation provided in the City of Salem Stormwater Management Manual.

Detention System Off-site North

In the detention analysis for this area, the 0.86-acre site was considered a single basin draining into the detention facility. A basin map is in Appendix A. Site grading and conveyance pipe will direct stormwater runoff to the detention pipe. The Lone Oak 60-inch detention pipe has a capacity to detain 2,350 cubic feet of water. This exceeds the required detention volume of 1,500 cubic feet for the Off-site North developed portion.

Based on the above design parameters, the 10-year through the 100-year pre-developed release rates are controlled at below the allowable rate. The release rate and detention requirements were generated from the HydroCAD software, which can be seen in Appendix D. Table 6 below summarizes the requirements for the storm events.

Table 6

Storm Event	Control MH#2 Release Rate (cfs)	Required Detention Volume (ft^3)	Provided Detention Volume (ft^3)
10-year	1.65	131	2,350
25-year	2.31	286	2,350
100-year	4.16	1,256	2,350

(Off-site North Detention Summary)

Flow control is achieved with a single orifice in a standard City of Salem control structure. The sizing of the orifice uses the standard orifice equation provided in the City of Salem Stormwater Management Manual. Table 8 below identifies orifice size, elevation and the water surface elevation.

Detention System Phase #2

In the detention analysis for this phase, the 8.9-acre site was considered a single basin draining into the detention pond. A basin map is in Appendix A. Site grading and conveyance pipe will direct stormwater runoff to the detention system. It should be noted that the Basin #2 pond has a capacity to detain 52,000 cubic feet of water. This exceeds the required detention volume of 25,900 cubic feet for the developed portion.

Based on the above design parameters, the half of the 2-year through the 100-year pre-developed release rates are controlled at 0.03, 1.06, 1.42 and 2.56 cfs. The release rates and detention requirements were generated from the HydroCAD software, which can be seen in Appendix D. Table 7 below summarizes the requirements for the storm events.

Table 7

Storm Event	Basin #2 Release Rate (cfs)	Required Detention Volume (ft^3)	Provided Detention Volume (ft^3)
Half of 2-year	0.03	8,325	52,000
10-year	1.06	18,935	52,000
25-year	1.42	21,750	52,000
100-year	2.56	25,900	52,000

(Basin #2 Pond Detention Summary)

Flow control is achieved with multiple orifices in a standard City of Salem control structure. The sizing of the orifice will use the standard orifice equation provided in the City of Salem Stormwater Management Manual.

It should be noted in the analysis for the area in Basin 2B, the area was reduced to 0.24 acres. Pervious concrete is proposed to be installed for all driveways fronting Devon Avenue SE to reduce runoff. Per City standards, the area can be ignored.

STORMWATER QUALITY ANALYSIS

Water quality flow rates were calculated using HydroCAD 10.00. The SCS TR-20 Unit Hydrograph method was used to generate the hydrographs. A Type 1A rainfall distribution was used with a 1.38 rainfall depth. Appendix E contains the analysis.

Detention Basins #1 and #3 serve as combination facilities and provide water quality treatment via filtration per City standards.

Because of natural slopes that exceed 10 percent along the lower portion of Lone Oak Drive, runoff will not drain into the water quality system. A manufactured stormwater treatment facility will be used to treat this runoff.

The proposed facility is a Contech StormFilter system using ZPG media. The filters will be in a catch basin with a high flow bypass to convey larger storm events. The media filters will be the 27-inch height type that have the capacity to treat 22.5 gpm per filter. The catch basins will be spaced at a maximum distance of 200-feet to treat a maximum area of approximately 12,000 square feet. 80 percent was considered impervious. A total of 3 catch basins in series will be installed along Lone Oak Drive. Appendix E contains the analysis and a generic plan of the Contech system. Table 8 below shows the required filters per catch basin.

Table 8

Storm Event	Basin WQ Flow Rate (cfs)	Basin WQ Flow Rate (gpm)	Required Filters	System Capacity (gpm)
WQ	0.05	22.4	1	22.5

CONCLUSION

Based on the presented information, the preliminary design can meet the City of Salem water quality and quantity standards. If there are any questions regarding this analysis or the design, please contact Matthew Hendrick at Multi/Tech Engineering by phone at (503) 363-9227 or via e-mail at mhendrick@mtengineering.net.

Appendix A

08 3W 22C

SEE MAP 083W21
4 COR
SEE MAP 083W22BC
SEE MAP 083W22
SEE MAP 083W22BD
SAHALEE COURT
SEE MAP 083W22AC
924 01 00 0
VACATED BY 2774-0009
C COR SEC
DEVON AVE
SAHALEE DR
LONE OAK RD
924 01 00 0
924 00 23 0
024 00 23 0
SEE MAP 083W21DA
SEE MAP 083W22CB
200 10.01 AC
100 9.71 AC
924 01 00 0
ELKINS WAY SE
SUNNY SIDE
300 19.89 AC
Site
924 00 23 0
SEE MAP 083W22DB
924 00
DEVON AVE SE
LL ROAD SE (CR 831)
TRILLIUM LANE
700 4.15 AC
600 4.70 AC
601 4.70 AC
500 4.73 AC
400 PARCEL 1 17.48 AC
FARMS
P.A.R.T. PLAT 1994-106
401 PARCEL 2 1.00 AC
20' ACCESS EASEMENT
REES HILL ROAD SE
(CR 831)
SEE MAP 083W22DC
REES HILL ROAD
SEE MAP 083W21DD
800 2.02 AC
1100 5.00 AC
1300 5.03 AC
1400 17.77 AC
1500 5.00 AC
1600 5.00 AC
1700 4.97 AC
1800 1.25 AC
2100 3.61 AC
2200 3.99 AC
2300 9.97 AC
024 00 23 0
900 5.36 AC
FRUIT
1401 2.23 AC
NO. 3
1900 3.77 AC
CHAMPION HILL ROAD SE
CREEK
POWELL
PUBLIC ROAD
1/4 COR
SEE MAP 083W28A
1000 2.65 AC
024 00 23 0
2000 2.35 AC
SUNNY SIDE FRUIT FARMS NO. 3
SEE MAP 083W27A
SEE MAP 083W27B
08 3W 22C

08 3W 22C
SALEM

MARION COUNTY, OREGON

SW1/4 SEC22 T8S R3W W.M.
SCALE 1" = 200'

LEGEND

LINE TYPES

- Taxlot Boundary
- Historical Boundary
- Road Right-of-Way
- Easement
- Railroad Right-of-Way
- Railroad Centerline
- Private Road ROW
- Taxcode Line
- Subdivision/Plat Bndry
- Map Boundary
- Waterline - Taxlot Bndry
- Waterline - Non Bndry

CORNER TYPES

- 1/16TH Section Cor.
- 1/4 Section Cor.
- DLC Corner
- Section Corner

NUMBERS

Tax Code Number
000 00 00 0

Acreage
0.25 AC

All acres listed are Net Acres, excluding any portions of the taxlot within public ROWs

NOTES

Tick Marks: A tick mark in the road indicates that the labeled dimension extends into the public ROW

200.00
175.00

CANCELLED NUMBERS

1200

DISCLAIMER: THIS MAP WAS PREPARED FOR ASSESSMENT PURPOSES ONLY

Marion County OREGON — Assessors Office Cartography Dept

FOR ADDITIONAL MAPS VISIT OUR WEBSITE AT
www.co.marion.or.us

PLOT DATE: 1/24/2018

SALEM
08 3W 22C

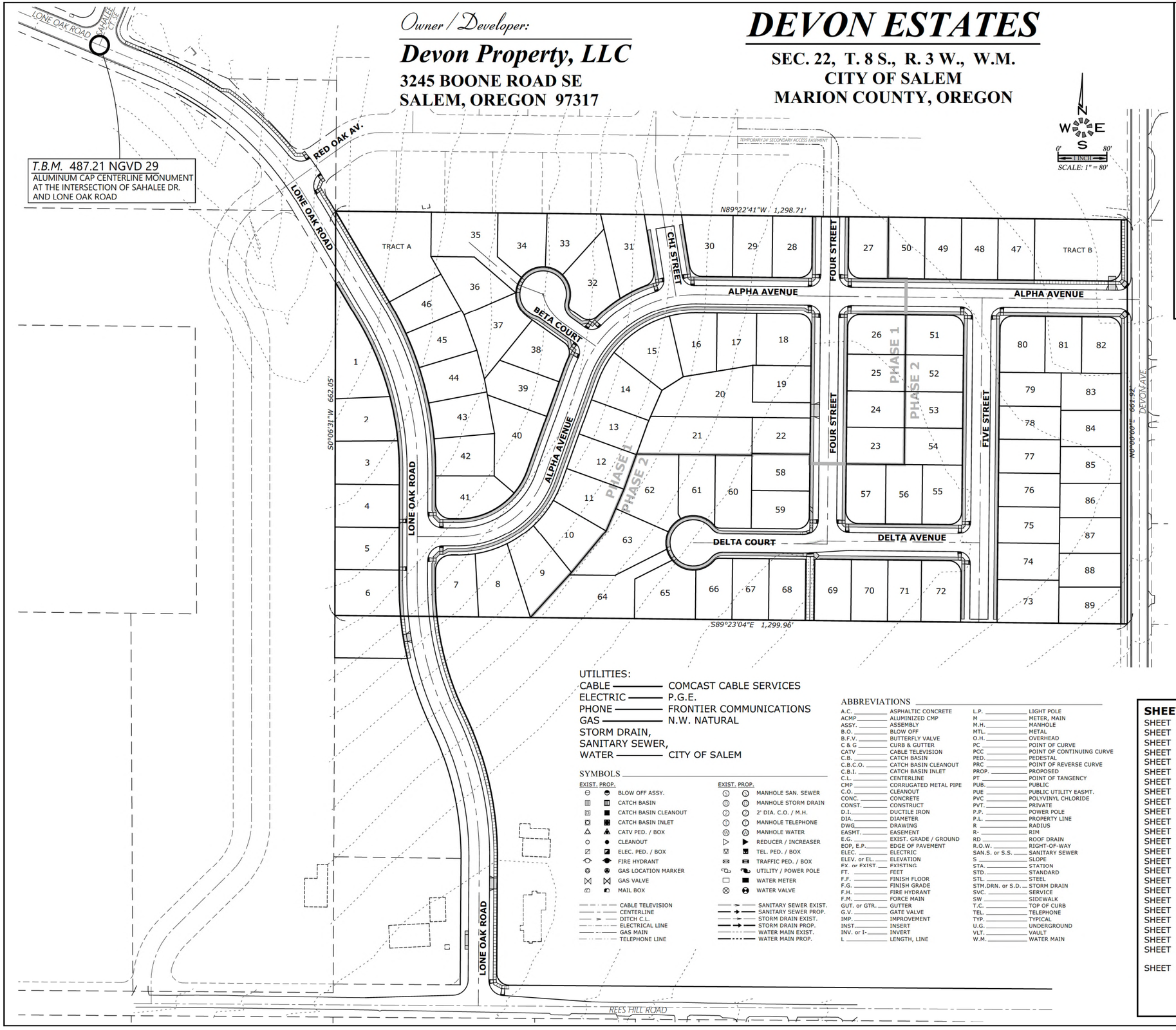
DEVON ESTATES

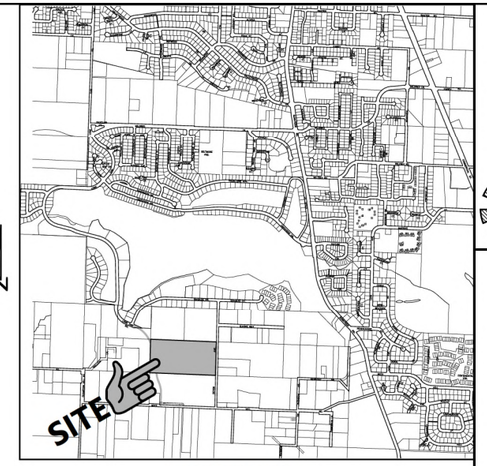
SEC. 22, T. 8 S., R. 3 W., W.M.
CITY OF SALEM
MARION COUNTY, OREGON

Owner / Developer:

Devon Property, LLC
3245 BOONE ROAD SE
SALEM, OREGON 97317

T.B.M. 487.21 NGVD 29
ALUMINUM CAP CENTERLINE MONUMENT AT THE INTERSECTION OF SAHALEE DR. AND LONE OAK ROAD





VICINITY MAP

TOTAL PARCEL SIZE:
DEVELOPABLE AREA — 19.745 Ac.
NUMBER OF UNITS — 89
DENSITY — 4.51 UNITS/Ac.
LARGEST LOT — 13,195 S.F.
SMALLEST LOT — 5,500 S.F.
AVERAGE — 6,753 S.F.

PHASE 1:
DEVELOPABLE AREA — 11.026 Ac.
NUMBER OF UNITS — 46
DENSITY — 4.17 UNITS/Ac.
LARGEST LOT — 13,195 S.F.
SMALLEST LOT — 5,700 S.F.
AVERAGE — 7,182 S.F.

PHASE 2:
DEVELOPABLE AREA — 8.719 Ac.
NUMBER OF UNITS — 43
DENSITY — 4.93 UNITS/Ac.
LARGEST LOT — 11,286 S.F.
SMALLEST LOT — 5,500 S.F.
AVERAGE — 6,295 S.F.

Drawing is NOT to scale

UTILITIES:
CABLE — COMCAST CABLE SERVICES
ELECTRIC — P.G.E.
PHONE — FRONTIER COMMUNICATIONS
GAS — N.W. NATURAL
STORM DRAIN, SANITARY SEWER, WATER — CITY OF SALEM

SYMBOLS

EXIST. PROP.		EXIST. PROP.	
⊖	BLOW OFF ASSY.	Ⓢ	MANHOLE SAN. SEWER
▥	CATCH BASIN	Ⓓ	MANHOLE STORM DRAIN
▣	CATCH BASIN CLEANOUT	②	2' DIA. C.O. / M.H.
▦	CATCH BASIN INLET	Ⓣ	MANHOLE TELEPHONE
△	CATV PED. / BOX	Ⓦ	MANHOLE WATER
○	CLEANOUT	▷	REDUCER / INCREASER
◪	ELEC. PED. / BOX	▣	TEL. PED. / BOX
⛑	FIRE HYDRANT	▭	TRAFFIC PED. / BOX
◉	GAS LOCATION MARKER	⚲	UTILITY / POWER POLE
⋈	GAS VALVE	□	WATER METER
▭	MAIL BOX	⊗	WATER VALVE
	CABLE TELEVISION		SANITARY SEWER EXIST.
	CENTERLINE		SANITARY SEWER PROP.
	DITCH C.L.		STORM DRAIN EXIST.
	ELECTRICAL LINE		STORM DRAIN PROP.
	GAS MAIN		WATER MAIN EXIST.
	TELEPHONE LINE		WATER MAIN PROP.

ABBREVIATIONS

A.C.	ASPHALTIC CONCRETE	L.P.	LIGHT POLE
ACMP	ALUMINIZED CMP	M	METER, MAIN
ASSY.	ASSEMBLY	M.H.	MANHOLE
B.O.	BLOW OFF	MTL.	METAL
B.F.V.	BUTTERFLY VALVE	O.H.	OVERHEAD
C & G	CURB & GUTTER	PC	POINT OF CURVE
CATV	CABLE TELEVISION	PCC	POINT OF CONTINUING CURVE
C.B.	CATCH BASIN	PED.	PEDESTAL
C.B.C.O.	CATCH BASIN CLEANOUT	PRC	POINT OF REVERSE CURVE
C.B.I.	CATCH BASIN INLET	PROP.	PROPOSED
C.L.	CENTERLINE	PT	POINT OF TANGENCY
CMP	CORRUGATED METAL PIPE	PUB.	PUBLIC
C.O.	CLEANOUT	PUE	PUBLIC UTILITY EASMT.
CONC.	CONCRETE	PVC	POLYVINYL CHLORIDE
CONST.	CONSTRUCT	PVT.	PRIVATE
D.I.	DUCTILE IRON	P.P.	POWER POLE
DIA.	DIAMETER	P.L.	PROPERTY LINE
DWG.	DRAWING	R	RADIUS
EASMT.	EASEMENT	R-	RIM
E.G.	EXIST. GRADE / GROUND	RD	ROOF DRAIN
EOP, E.P.	EDGE OF PAVEMENT	R.O.W.	RIGHT-OF-WAY
ELEC.	ELECTRIC	SAN.S. or S.S.	SANITARY SEWER
ELEV. or EL.	ELEVATION	S	SLOPE
FX. or EXIST.	EXISTING	STA.	STATION
FT.	FEET	STD.	STANDARD
F.F.	FINISH FLOOR	STL.	STEEL
F.G.	FINISH GRADE	STM.DRN. or S.D.	STORM DRAIN
F.H.	FIRE HYDRANT	SVC.	SERVICE
F.M.	FORCE MAIN	SW	SIDEWALK
GUT. or GTR.	GUTTER	T.C.	TOP OF CURB
G.V.	GATE VALVE	TEL.	TELEPHONE
IMP.	IMPROVEMENT	TYP.	TYPICAL
INST.	INSERT	U.G.	UNDERGROUND
INV. or I-	INVERT	VLT.	VAULT
L	LENGTH, LINE	W.M.	WATER MAIN

SHEET INDEX

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PRELIMINARY COVER SHEET

DEVON ESTATES

6500x20 P101CO
Design: M.D.G.
Drawn: P.H.S.
Checked: B.M.G.
Date: NOV. 2017
Scale: AS SHOWN
As-Built: ---

REGISTERED PROFESSIONAL ENGINEER
OREGON
JULY 1, 1978
HAROLD B. MCENTEE
EXPIRES: 06-30-2021

JOB # 6502

P101

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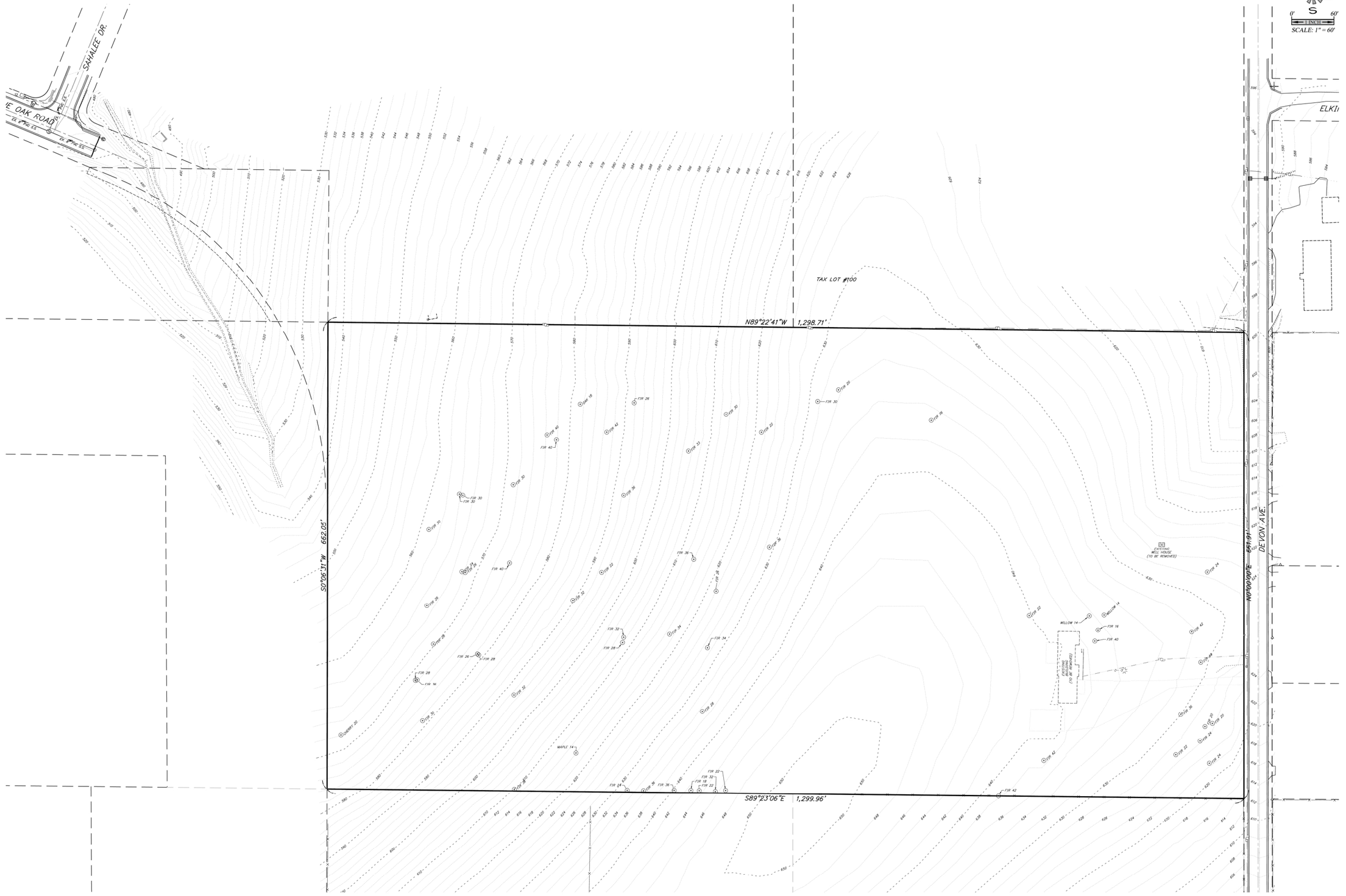
Drawing is NOT to scale



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EXISTING CONDITIONS PLAN

DEVON ESTATES

6502p P102XC

Design: M.D.G.
Drawn: P.H.S.
Checked: B.M.G.
Date: NOV. 2017
Scale: AS SHOWN

REGISTERED PROFESSIONAL ENGINEER
OREGON
JUNE 14, 2005
MARK D. GRENZ

EXPIRES: 06-30-2019

JOB # 6502

P102

J:\6500-6599\6502-DevonAvenueSubdivision\Dwg\6502p.dwg, P102XC, 5/3/2019 10:26:13 AM, PSanders

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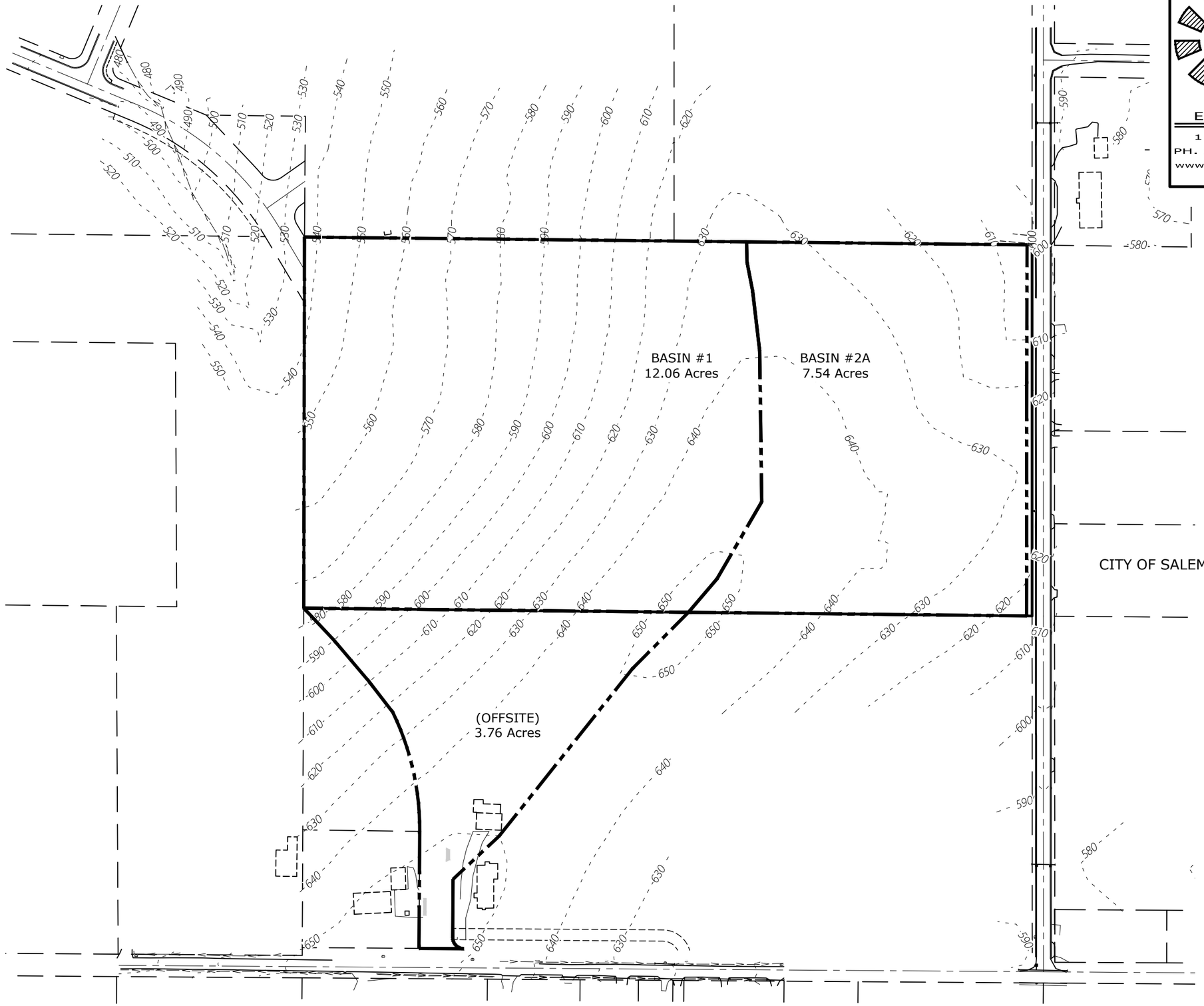
PRE-DEVELOPED BASIN MAP



N
W E
S

NOT TO SCALE

CITY OF SALEM PREDEVELOPED HSG C CN=72

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POST-DEVELOPED BASIN MAP

DETENTION BASIN #2 CAPACITY: 2,350 C.F.

DETENTION BASIN #1 CAPACITY: 47,000 C.F.

FUTURE DETENTION BASIN #3 CAPACITY: 52,000 C.F.

BASIN 2B*
0.07 Acre IMPERVIOUS SURFACE – HSG C CN = 98
0.17 Acre LANDSCAPE ——— HSG C CN = 74

BASIN: OFFSITE NORTH 0.86 Acre

BASIN 1B*
0.71 Acre

BASIN 1
10.00 Acres

BASIN 2
8.62 Acres

BASIN 2B*
0.27 Acre

(OFFSITE)
3.76 Acres

N
W E
S
NOT TO SCALE

BASIN 1B*
50% IMPERVIOUS SURFACE —— HSG C CN = 98
50% LANDSCAPE ——— HSG C CN = 74

DEVELOPED SITE (BASIN 1 & 2)
60% IMPERVIOUS SURFACE —— HSG C CN = 98
40% LANDSCAPE ——— HSG C CN = 74

OFFSITE:
CITY OF SALEM PREDEVELOPED — HSG C CN = 72
3.76 Acres
IMPERVIOUS SURFACE ——— HSG C CN = 98
0.50 Acre

OFFSITE NORTH:
0.76 Acre IMPERVIOUS SURFACE – HSG C CN = 98
0.10 Acre LANDSCAPE ——— HSG C CN = 74

* = CALCULATED FLOW RATES FROM THESE BASINS WERE REMOVED FROM THE PARENT BASINS ALLOWABLE RUNOFF RATE.

6502b20-o21.dwg/BASIN2:180

J:\6500-6599\6502-DevonAvenueSubdivision\Dwg20\OLD20\6502b20-o21.dwg, BASIN2, 2/17/2021 9:47:26 AM, PSaunders

Appendix B

United States Department of Agriculture

NRCS

Natural Resources Conservation Service

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

Custom Soil Resource Report for Marion County Area, Oregon

Devon Estates

October 11, 2018

Hydrologic Soil Group—Marion County Area, Oregon
(Devon Estates)

Map Scale: 1:2,190 if printed on A landscape (11" x 8.5") sheet.

N

Meters
0 30 60 120 180

Feet
0 100 200 400 600

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: Marion County Area, Oregon
Survey Area Data: Version 15, Sep 18, 2018

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

Date(s) aerial images were photographed: Jun 15, 2015—Jun 23, 2015

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
JoC	Jory silty clay loam, 7 to 12 percent slopes	C	1.8	8.8%
JoD	Jory silty clay loam, 12 to 20 percent slopes	C	10.3	50.5%
NeB	Nekia silty clay loam, 2 to 7 percent slopes	C	7.1	34.7%
NeC	Nekia silty clay loam, 7 to 12 percent slopes	C	0.4	1.8%
SlB	Salkum silty clay loam, basin, 0 to 6 percent slopes	B	0.8	4.1%
Totals for Area of Interest			**20.5**	**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

REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation

and

Geologic Hazard Assessment Services

Proposed Devon Avenue Residential Subdivision Site

Tax Lot No. 300 (Lots 13 and 14)

6719 Devon Avenue SE

Salem (Marion County), Oregon

for

Multi/Tech Engineering Services, Inc.

Project No. 1001.052.G
August 11, 2017

REDMOND GEOTECHNICAL SERVICES

August 11, 2017

Mr. Mark D. Grenz
Multi/Tech Engineering Services, Inc.
1155 13th Street SE
Salem, Oregon 97302

Dear Mr. Grenz:

Re: Geotechnical Investigation and Geologic Hazard Assessment Services, Proposed Devon Avenue Residential Subdivision Site, Tax Lot No. 300 (Lots 13 and 14), 6719 Devon Avenue SE, Salem (Marion County), Oregon

Submitted herewith is our report entitled "Geotechnical Investigation and Geologic Hazard Assessment Services, Proposed Devon Avenue Residential Subdivision Site, Tax Lot No. 300 (Lots 13 and 14), 6719 Devon Avenue SE, Salem (Marion County), Oregon". The scope of our services was outlined in our formal discussions with Mr. Mark D. Grenz of Multi/Tech Engineering Services, Inc on June 29, 2017. Verbal authorization of our services was provided by Mr. Mark D. Grenz on June 29, 2017.

During the course of our investigation, we have kept you and/or others advised of our schedule and preliminary findings. We appreciate the opportunity to assist you with this phase of the project. Should you have any questions regarding this report, please do not hesitate to call.

Sincerely,

Daniel M. Redmond, P.E., G.E.
President/Principal Engineer

12-3-18

PO Box 20547 • Portland, Oregon 97294 • Fax 503/286-7176 • Phone 503/285-0598

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GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARD ASSESSMENT
PROPOSED DEVON AVENUE RESIDENTIAL DEVELOPMENT SITE
TAX LOT NO. 300 (LOTS 13 AND 14)
6719 DEVON AVENUE SE
SALEM (MARION COUNTY) OREGON

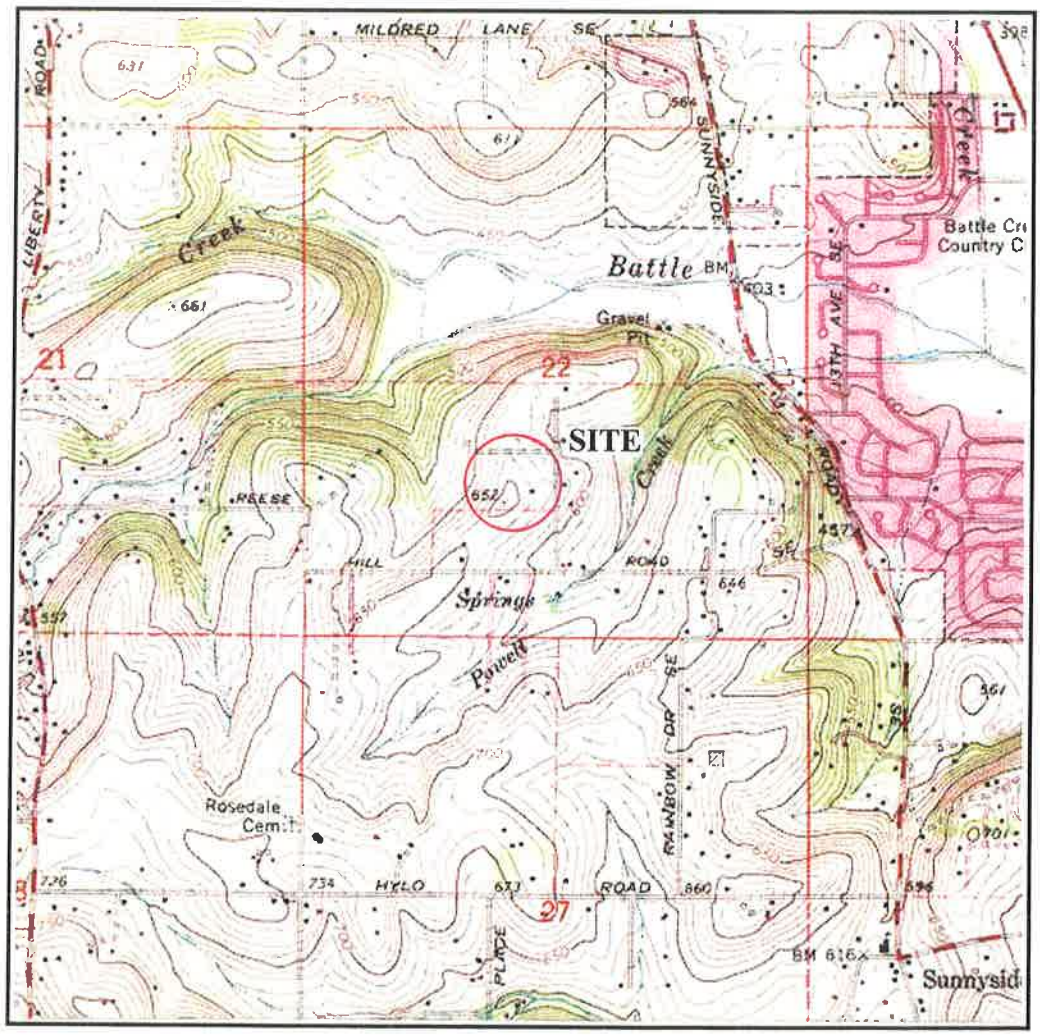
INTRODUCTION

Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Geologic Hazard Assessment at the site of the proposed new residential development located to the west of Devon Avenue SE and to the north of Reese Hill Road SE in Salem (Marion County), Oregon. The general location of the subject site is shown on the Site Vicinity Map, Figure No. 1. The purpose of our geotechnical investigation and geologic hazard study services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to evaluate any potential concerns with regard to potential slope failure at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new residential development project.

PROJECT DESCRIPTION

We understand that present plans are to develop the subject property into new single-family residential lots. Although the project is still in the preliminary planning stages, we understand that the proposed new residential development will consist of the construction of approximately ninety (90) new single-family residential lots ranging in size from about 6,000 to 10,000 square feet (see Site Exploration Plan, Figure No. 2). The new residential homes are anticipated to be of two- and/or three-story structures constructed with wood framing. Support of the new residential structures is anticipated to consist primarily of conventional shallow strip (continuous) footings although some individual (column) footings may also be required. Structural loading information, although unavailable at this time, is anticipated to be fairly typical and light for this type of wood-frame single-family residential structure and is expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 1.5 to 2.5 kips per lineal foot (klf) and 10 to 25 kips, respectively.

Although a site grading plan is not available at this time, we understand that both cuts and fills are presently planned for the residential project. In general, relatively minor cuts and/or fills (i.e., 5 to 8 feet) will be required across the proposed residential lots as well as the proposed new public street improvements. In this regard, due to the existing and/or finish grade sloping site conditions, some of the proposed new single-family residential structures and/or lots may also include the construction of a partial below grade floor(s) and/or retaining walls.



SIDNEY QUADRANGLE
OREGON
7.5 MINUTE SERIES (TOPOGRAPHIC)
SE/4 SALEM 15' QUADRANGLE

SCALE 1:24 000

1 0 1/2 1 MILE
1000 0 1000 2000 3000 4000 5000 6000 7000 FEET
1 .5 0 1 KILOMETER

CONTOUR INTERVAL 10 FEET
DOTTED LINES REPRESENT 5-FOOT CONTOURS
NATIONAL GEODETIC VERTICAL DATUM OF 1929

SITE VICINITY MAP

Project No. 1001.052.G	**DEVON AVENUE SUBDIVISION** **TL 300, 6719 DEVON AVENUE SE**	Figure No. 1

Other associated site improvements for the project will include construction of new public street improvements along Devon Avenue SE as well as new local residential streets. Additionally, the project will include the construction of new underground utility services as well as new concrete curbs and sidewalks. Further, we understand that the project will also include the collection of storm water from hard and//or impervious surfaces (i.e., roofs and pavements) for possible on-site treatment and/or disposal in a storm water system designed by the project civil engineer.

SCOPE OF WORK

The purpose of our geotechnical and/or geologic studies was to evaluate the overall subsurface soil and/or groundwater conditions underlying the subject site with regard to the proposed new residential development and construction at the site and any associated impacts or concerns with respect to potential slope failure at the site as well as provide appropriate geotechnical design and construction recommendations for the project. Additionally, due to the moderately steep sloping site gradients, a slope stability analysis was also performed.

Specifically, our geotechnical investigation and landslide hazard study performed as a collaboration with Northwest Geological Services, Inc. (NWGS, Inc.) included the following scope of work items:

1. Review of available and relevant geologic and/or geotechnical investigation reports for the subject site and/or area.

2. A detailed field reconnaissance and subsurface exploration program of the soil and ground water conditions underlying the site by means of eight (8) exploratory test pit excavations. The exploratory test pits were excavated to depths ranging from about five (5) to six (6) feet beneath existing site grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 2. Additionally, field infiltration testing was also performed within two (2) of the test pit excavations.

3. Laboratory testing to evaluate and identify pertinent physical and engineering properties of the subsurface soils encountered relative to the planned site development and construction at the site. The laboratory testing program included tests to help evaluate the natural (field) moisture content and dry density, maximum dry density and optimum moisture content, expansion index, gradational characteristics, Atterberg Limits and (remolded) direct shear strength tests as well as "R"-value tests.

4. A literature review and engineering evaluation and assessment of the regional seismicity to evaluate the potential ground motion hazard(s) at the subject site. The evaluation and assessment included a review of the regional earthquake history and sources such as potential seismic sources, maximum credible earthquakes, and reoccurrence intervals as well as a discussion of the possible ground response to the selected design earthquake(s), fault rupture, landsliding, liquefaction, and tsunami and seiche flooding.

5. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new residential structures. Recommendations include maximum design allowable contact bearing pressure(s), depth of footing embedment, estimates of foundation settlement, lateral soil resistance, and foundation subgrade preparation. Additionally, construction and/or permanent subsurface water drainage considerations have also been prepared. Further, our report includes recommendations regarding site preparation, placement and compaction of structural fill materials, suitability of the on-site soils for use as structural fill, criteria for import fill materials as well as preparation of foundation, pavement and/or floor slab subgrades.

6. Flexible pavement design and construction recommendations for the proposed new public street improvements.

SITE CONDITIONS

Site Geology

The subject site and/or area is underlain by highly weathered Basalt bedrock deposits and/or residual soils of the Columbia River Basalt formation. A more detailed description of the site geology across and/or beneath the site is presented in the Geologic Hazard Study in Appendix B.

Surface Conditions

The subject proposed new residential development property consists of one (1) rectangular shaped tax lot (Tax Lot 300) which includes Lots 13 and 14 and encompasses a total plan area of approximately 19.89 acres. The proposed residential development property is roughly located to the west of Devon Avenue SE and to the north of Reese Hill Road SE. The easterly portion of the subject proposed residential development site is presently improved and contain existing single-family residential homes while the remainder of the site is unimproved and consist of existing open land. Surface vegetation across the site generally consists of a light to moderate growth of grass, weeds and brush as well as numerous small to large size trees. Additionally, an existing seasonal drainage basin is located along the westerly portion of the site.

Topographically, the site is characterized as gently to moderately sloping terrain (10 to 25 percent) descending downward from the central portion of the site towards the west and east with overall topographic relief estimated at about seventy (70) feet and ranges from a low about Elevation 580 feet near the northwesterly corner of the subject site to a high of about Elevation 650 near the central portion of the site.

Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of eight (8) exploratory test pits excavated to depths ranging from about five (5) to six (6) feet beneath existing site grades on July 11, 2017 with a John Deere 200C track-mounted excavator.

The location of the exploratory test pits were located in the field by marking off distances from existing and/or known site features and are shown in relation to the proposed new residential structures and/or site improvements on the Site Exploration Plan, Figure No. 2. Detailed logs of the test pit explorations, presenting conditions encountered at each location explored, are presented in the Appendix, Figure No's. A-5 through A-8.

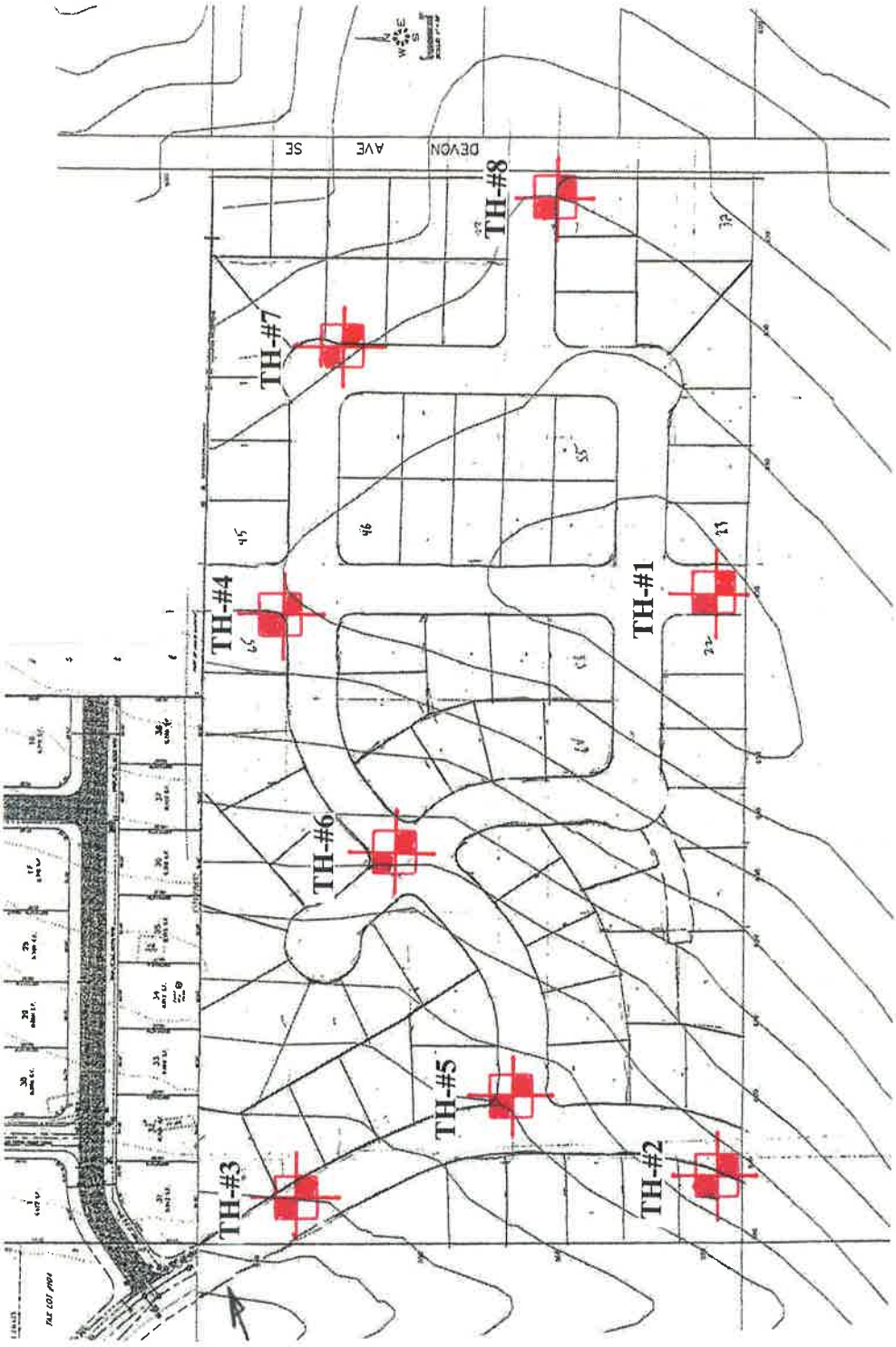
The exploratory test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test pit explorations and obtained representative samples of the subsurface soils encountered across the site. Additionally, the elevation of the exploratory test pit excavations were referenced from the proposed Site Development Plan prepared by Project Delivery Group. and should be considered as approximate. All subsurface soils encountered at the site and/or within the exploratory test pit excavations were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-4.

The test pit explorations revealed that the subject site is underlain by native soil deposits comprised of highly weathered bedrock and/or residual soils composed of a surficial layer of dark brown, moist, soft, organic, sandy, clayey silt topsoil materials to depths of about 6 to 12 inches. These surficial topsoil materials were inturn underlain by residual soils composed of medium to reddish--brown, moist to very moist, medium stiff to stiff, sandy, clayey silt to a depth of about two (2) to four (4) feet beneath the existing site and/or surface grades. These upper clayey silt (residual) subgrade soils are best characterized by relatively low to moderate strength and moderate compressibility. These upper clayey silt subgrade soils were inturn underlain by medium to orangish-brown, moist to very moist, medium dense becoming dense at depth, clayey, silty sand to highly weathered bedrock deposits to the maximum depth explored of about six (6) feet beneath the existing site and/or surface grades. These clayey, silty sand subgrade soils and/or highly weathered bedrock deposits are best characterized by relatively moderate to high strength and low compressibility.

Groundwater

Groundwater was generally not encountered within any of the exploratory test pit explorations (TH-#1 through TH-#8) at the time of excavation to depths of at least six (6) feet beneath existing surface grades. However, the westerly portion of the subject property is bounded by an existing seasonal drainage basin and/or surface feature.

In this regard, although groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions and/or associated with runoff within the westerly drainage basins as well as changes in site utilization, we are generally of the opinion that the static water levels and/or surface water ponding not observed during our recent field exploration work generally reflect a high seasonal groundwater level(s) at and/or beneath the site.



SITE EXPLORATION PLAN

Project No. 1001.052.G	**DEVON AVENUE SUBDIVISION** **TL 300, 6719 DEVON AVENUE SE**	Figure No. 2

INFILTRATION TESTING

We performed two (2) field infiltration tests at the site on July 11, 2017. The infiltration tests were performed in test holes TH-#2 and TH-#3 at depths of between two (2) to three (3) feet beneath the existing site and/or surface grades. The subgrade soils encountered in the infiltration test hole consisted of sandy, clayey silt.

The infiltration testing was performed in general conformance with current EPA and/or the City of Salem Department of Public Works Administrative Rules Chapter 109 Division 004 Appendix C Encased Falling Head test method which consisted of advancing a 6-inch diameter PVC pipe approximately 6 inches into the exposed soil horizon at each test location. Using a steady water flow, water was discharged into the pipe and allowed to penetrate and saturate the subgrade soils. The water level was adjusted over a two (2) hour period and allowed to achieve a saturated subgrade soil condition consistent with the bottom elevation of the surrounding test pit excavation. Following the required saturating period, water was again added into the PVC pipe and the time and/or rate at which the water level dropped was monitored and recorded. Each measurable drop in the water level was recorded until a consistent infiltration rate was observed and/or repeated.

Based on the results of the field infiltration testing at the site, we have found that the native sandy, clayey silt subgrade soil deposits posses an ultimate infiltration rate on the order of about 0.6 to 0.8 inches per hour (in/hr).

LABORATORY TESTING

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from various test pit excavations and returned to our laboratory for further examination and testing and/or to aid in the classification of the subsurface soils as well as to help evaluate and identify their engineering strength and compressibility characteristics. The laboratory testing consisted of visual and textural sample inspection, moisture content and dry density determinations, maximum dry density and optimum moisture content, expansion index, gradation analyses and Atterberg Limits as well as (remolded) direct shear strength and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-9 through A-13.

SEISMICITY AND EARTHQUAKE SOURCES

The seismicity of the southwest Washington and northwest Oregon area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan de Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers (km). The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Washington and Oregon coastlines. Sequences of interlayered peat and sands have been interpreted to be the result of large Subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A study by Geomatrix (1995) and/or USGS (2008) suggests that the maximum earthquake associated with the CSZ is moment magnitude (Mw) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes that have occurred within Subduction zones in other parts of the world. An Mw 9 earthquake would involve a rupture of the entire CSZ. As discussed by Geomatrix (1995) this has not occurred in other subduction zones that have exhibited much higher levels of historical seismicity than the CSZ. However, the 2008 USGS report has assigned a probability of 0.67 for a Mw 9 earthquake and a probability of 0.33 for a Mw 8.3 earthquake. For the purpose of this study an earthquake of Mw 9.0 was assumed to occur within the CSZ.

The intraplate zone encompasses the portion of the subducting Juan de Fuca Plate located at a depth of approximately 30 to 50 km below western Washington and western Oregon. Very low levels of seismicity have been observed within the intraplate zone in western Oregon and western Washington. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of Subduction between Oregon, Washington, and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia magnitude 7.1 and the 1965 Puget Sound magnitude 6.5 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The third source of seismicity that can result in ground shaking within the Vancouver and southwest Washington area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in this area is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0), Oregon earthquakes were crustal earthquakes.

Liquefaction

Seismic induced soil liquefaction is a phenomenon in which lose, granular soils and some silty soils, located below the water table, develop high pore water pressures and lose strength due to ground vibrations induced by earthquakes. Soil liquefaction can result in lateral flow of material into river channels, ground settlements and increased lateral and uplift pressures on underground structures.

Buildings supported on soils that have liquefied often settle and tilt and may displace laterally. Soils located above the ground water table cannot liquefy, but granular soils located above the water table may settle during the earthquake shaking.

Our review of the subsurface soil test pit logs from our exploratory field explorations (TH-#1 through TH-#8) and laboratory test results indicate that the site is generally underlain by medium stiff to stiff, sandy, clayey silt soils and/or medium dense to dense highly weathered bedrock deposits to depths of at least 6.0 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#8) at the site during our field exploration work to depths of at least 6.0 feet.

As such, due to the medium stiff to stiff and/or cohesive nature of the sandy, clayey silt subgrade soils and/or medium dense to dense highly weathered bedrock deposits beneath the site, it is our opinion that the native sandy, clayey silt subgrade soil and/or highly weathered bedrock deposits located beneath the subject site have a very low potential for liquefaction during the design earthquake motions previously described.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, development of the subject site into the planned residential homes sites does not appear to present a potential geologic and/or landslide hazard provided that the site grading and development activities conform with the recommendations presented within this report. A more detailed assessment of the potential landslide hazard of the subject site is presented in the Geologic Hazard Study in Appendix B.

Surface Rupture

Although the site is generally located within a region of the country known for seismic activity, no known faults exist on and/or immediately adjacent to the subject site. As such, the risk of surface rupture due to faulting is considered negligible.

Tsunami and Seiche

A tsunami, or seismic sea wave, is produced when a major fault under the ocean floor moves vertically and shifts the water column above it. A seiche is a periodic oscillation of a body of water resulting in changing water levels, sometimes caused by an earthquake. Tsunami and seiche are not considered a potential hazard at this site because the site is not near to the coast and/or there are no adjacent significant bodies of water.

Flooding and Erosion

Stream flooding is a potential hazard that should be considered in lowland areas of Marion County and Salem. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new residential structures and site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Marion County requirements for the 100-year flood levels of any nearby creeks, streams and/or drainage basins.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our field explorations, laboratory testing, and engineering analyses, it is our opinion that the site is presently stable and suitable for the proposed new single-family residential development and its associated site improvements provided that the recommendations contained within this report are properly incorporated into the design and construction of the project.

The primary features of concern at the site are 1) the presence of highly moisture sensitive clayey and silty (residual) subgrade soils across the site, 2) the presence of gently to moderately sloping site conditions across the proposed new residential lots and/or home sites, and 3) the relatively low infiltration rates anticipated within the near surface clayey and silty subgrade soils.

With regard to the moisture sensitive clayey and silty residual subgrade soils, we are generally of the opinion that all site grading and earthwork activities be scheduled for the drier summer months which is typically June through September.

In regards to the gently to moderately sloping site conditions across the proposed new residential home sites and/or lots, we are of the opinion that site grading and/or structural fill placement should be minimized where possible and should generally limit cuts and/or fills to about eight (8) feet or less without the approval of the Geotechnical Engineer. Additionally, where existing site slopes and/or surface grades exceed about 20 percent (1V:5H) and in order to construct the proposed new residential lots and/or new local residential streets, benching and keying of all fills into the natural site slopes may be required.

With regard to the relatively low infiltration rates anticipated within the residual clayey and silty subgrade soils beneath the site, we generally do not recommend any storm water infiltration within structural and/or embankment fills. However, some limited storm water infiltration may be feasible within the lower westerly portion of the subject property as well as the proposed residential lots and/or areas of the site where the existing and/or finish slope gradients are no steeper than about 20 percent (1V:5H). In this regard, we recommend that all proposed storm water detention and/or infiltration systems for the project be reviewed and approved by Redmond Geotechnical Services, LLC.

The following sections of this report provide specific recommendations regarding subgrade preparation and grading as well as foundation and floor slab design and construction for the new residential development project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new residential building sites and/or lots as well as their associated structural and/or site improvement area(s) be stripped and cleared of all existing improvements, any existing unsuitable fill materials, surface debris, existing vegetation, topsoil materials, and/or any other deleterious materials present at the time of construction. In general, we envision that the site stripping to remove existing vegetation and topsoil materials will generally be about 6 to 12 inches. However, localized areas requiring deeper removals, such as existing undocumented and/or unsuitable fill materials as well as old foundation remnants, will likely be encountered and should be evaluated at the time of construction by the Geotechnical Engineer. The stripped and cleared materials should be properly disposed of as they are generally considered unsuitable for use/reuse as fill materials.

Following the completion of the site stripping and clearing work and prior to the placement of any required structural fill materials and/or structural improvements, the exposed subgrade soils within the planned structural improvement area(s) should be inspected and approved by the Geotechnical Engineer and possibly proof-rolled with a half and/or fully loaded dump truck. Areas found to be soft or otherwise unsuitable should be over-excavated and removed or scarified and recompacted as structural fill. During wet and/or inclement weather conditions, proof rolling and/or scarification and recompaction as noted above may not be appropriate.

The on-site native sandy, clayey silt (residual) subgrade soil materials are generally considered suitable for use/reuse as structural fill materials provided that they are free of organic materials, debris, and rock fragments in excess of about 6 inches in dimension. However, if site grading is performed during wet or inclement weather conditions, the use of some of the on-site native soil materials which contain significant silt and clay sized particles will be difficult at best. In this regard, during wet or inclement weather conditions, we recommend that an import structural fill material be utilized which should consist of a free-draining (clean) granular fill (sand & gravel) containing no more than about 5 percent fines. Representative samples of the materials which are to be used as structural fill materials should be submitted to the Geotechnical Engineer and/or laboratory for approval and determination of the maximum dry density and optimum moisture content for compaction.

In general, all site earthwork and grading activities should be scheduled for the drier summer months (June through September) if possible. However, if wet weather site preparation and grading is required, it is generally recommended that the stripping of topsoil materials be accomplished with a tracked excavator utilizing a large smooth-toothed bucket working from areas yet to be excavated. Additionally, the loading of strippings into trucks and/or protection of moisture sensitive subgrade soils will also be required during wet weather grading and construction.

In this regard, we recommend that areas in which construction equipment will be traveling be protected by covering the exposed subgrade soils with a woven geotextile fabric such as Mirafi FW404 followed by at least 12 inches or more of crushed aggregate base rock. Further, the geotextile fabric should have a minimum Mullen burst strength of at least 250 pounds per square inch for puncture resistance and an apparent opening size (AOS) between the U.S. Standard No. 70 and No. 100 sieves.

All structural fill materials placed within the new building and/or pavement areas should be moistened or dried as necessary to near (within 3 percent) optimum moisture conditions and compacted by mechanical means to a minimum of 92 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Structural fill materials should be placed in lifts (layers) such that when compacted do not exceed about 8 to 9 inches. Additionally, all fill materials placed within five (5) lineal feet of the perimeter (limits) of the proposed residential structures and/or pavements should be considered structural fill. Additionally, due to the sloping site conditions, we recommend that all structural fill materials planned in areas where existing surface and/or slope gradients exceed about 20 percent (1V:5H) be properly benched and/or keyed into the native (natural) slope subgrade soils. In general, a bench width of approximately ten (10) feet and a keyway depth of approximately two (2) foot is recommended. However, the actual bench width and keyway depth should be determined at the time of construction by the Geotechnical Engineer. Further, all fill slopes should be constructed with a finish slope surface gradient no steeper than about 2H:1V.

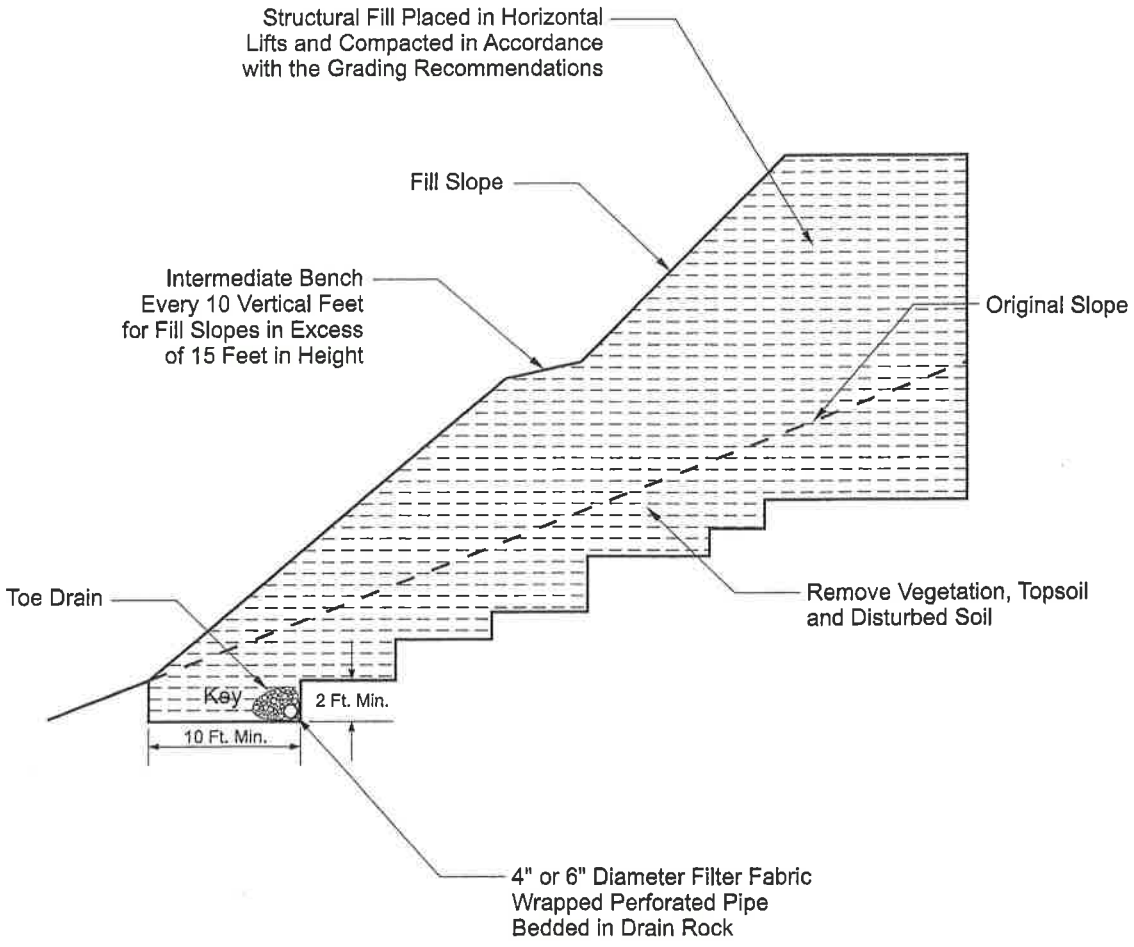
As such, settlement sensitive site and/or surface improvements (i.e., concrete curbs and sidewalks) should not be constructed until after primary consolidation and/or settlement has been completed. All aspects of the site grading, including a review of the proposed site grading plan(s), should be approved and/or monitored by a representative of Redmond Geotechnical Services, LLC.

Foundation Support

Based on the results of our investigation, it is our opinion that the site of the proposed new residential development is suitable for support of the two- and/or three-story wood-frame structures provided that the following foundation design recommendations are followed. The following sections of this report present specific foundation design and construction recommendations for the planned new residential structures.

Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved native (untreated) subgrade soil materials and/or silty structural fill soils based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). This recommended allowable contact bearing pressure is intended for dead loads and sustained live loads and may be increased by one-third for the total of all loads including short-term wind or seismic loads.



TYPICAL FILL SLOPE DETAIL

Project No. 1001.052.G	**DEVON AVENUE SUBDIVISION TL 300, 6719 DEVON AVENUE SE**	Figure No. 3

In general, continuous strip footings should have a minimum width of at least 16 inches and be embedded at least 18 inches below the lowest adjacent finish grade (includes frost protection). Individual column footings (where required) should be embedded at least 18 inches below grade and have a minimum width of at least 24 inches. Additionally, if foundation excavation and construction work is planned to be performed during wet and/or inclement weather conditions, we recommend that a 2 to 4 inch layer of compacted crushed rock be used to help protect the exposed foundation bearing surfaces until the placement of concrete.

Total and differential settlements of foundations constructed as recommended above and supported by approved native subgrade soils or by properly compacted structural fill materials are expected to be well within the tolerable limits for this type of lightly loaded wood-frame structure and should generally be less than about 1-inch and 1/2-inch, respectively.

Allowable lateral frictional resistance between the base of the footing element and the supporting subgrade bearing soil can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30 and 0.50 for native silty subgrade soils and/or import gravel fill materials, respectively. In addition, lateral loads may be resisted by passive earth pressures on footings poured "neat" against in-situ (native) subgrade soils or properly backfilled with structural fill materials based on an equivalent fluid density of 250 pounds per cubic foot (pcf). This recommended value includes a factor of safety of approximately 1.5 which is appropriate due to the amount of movement required to develop full passive resistance.

Floor Slab Support

In order to provide uniform subgrade reaction beneath concrete slab-on-grade floors, we recommend that the floor slab area be underlain by a minimum of 6 inches of free-draining (less than 5 percent passing the No. 200 sieve), well-graded, crushed rock. The crushed rock should help provide a capillary break to prevent migration of moisture through the slab. However, additional moisture protection can be provided by using a 10-mil polyolefin geo-membrane sheet such as StegoWrap.

The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Where floor slab subgrade materials are undisturbed, firm and stable and where the underslab aggregate base rock section has been prepared and compacted as recommended above, we recommend that a modulus of subgrade reaction of 150 pci be used for design.

Retaining/Below Grade Walls

Retaining and/or below grade walls should be designed to resist lateral earth pressures imposed by native soils or granular backfill materials as well as any adjacent surcharge loads. For walls which are unrestrained at the top and free to rotate about their base, we recommend that active earth pressures be computed on the basis of the following equivalent fluid densities:

Non-Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	35	30
3H:1V	60	50
2H:1V	90	80

For walls which are fully restrained at the top and prevented from rotation about their base, we recommend that at-rest earth pressures be computed on the basis of the following equivalent fluid densities:

Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	45	35
3H:1V	65	60
2H:1V	95	90

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher.

Backfill materials behind walls should be compacted to 90 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Special care should be taken to avoid over-compaction near the walls which could result in higher lateral earth pressures than those indicated herein. In areas within three (3) to five (5) feet behind walls, we recommend the use of hand-operated compaction equipment.

Pavements

Flexible pavement design for the proposed street improvements along the west side of Devon Avenue SE as well as the proposed new street improvements for the residential development project was determined in accordance with the City of Salem Department of Public Works Administrative Rules Chapter 109-006 (Street Design Standards) Section 6 dated January 1, 2014.

Specifically, on July 11, 2016, samples of the subgrade soils from the existing and/or proposed public streets were collected by means of various test hole excavations. The subgrade soils encountered in the test holes located across the proposed residential subdivision site as well as along the westerly side of the existing pavement grade of Devon Avenue SE and/or across the proposed new public street improvement areas generally consisted of native and/or residual soils comprised of medium to reddish-brown, medium stiff to stiff, sandy, clayey SILT (ML).

The subgrade soil samples collected at the site were tested in the laboratory in accordance with the ASTM Vol. 4.08 Part D-2844-69 (AASHTO T-190-93) test method for the determination of the subgrade soil "R"-value and expansion pressure. The results of the "R"-value testing was then converted to an equivalent Resilient Modulus (M_{RSG}) in accordance with current AASHTO methodology. The results of the laboratory "R"-value tests revealed that the subgrade soils have an apparent "R"-value of between 28 and 30 with an average "R"-value of 29 (see Figure No. A-12). Using the current AASHTO methodology for converting "R"-value to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of about 5,865 psi which is classified a "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

In addition to the above, Dynamic Cone Penetration (DCP) tests were performed along the proposed new interior public street alignment at approximate 100- to 200-feet intervals. The results of the DCP tests found that the underlying native sandy, clayey silt subgrade soils have a DCP value of between 3 to 4 blows per 2-inches which correlates to a California Bearing Ratio (CBR) of between 12 and 15. Using current AASHTO methodology for converting CBR to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of between 10,637 and 12,392 psi with an average M_{RSG} of 11,530 psi which is classified as "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

Devon Avenue SE

The following documents and/or design input parameters were used to help determine the flexible pavement section design for improvements to Devon Avenue SE:

- **Street Classification:** Collector Street
- **Design Life:** 20 years
- **Serviceability:** 4.2 initial, 2.5 terminal
- **Traffic Loading Data:** 1,000,000 18-kip EAL's
- **Reliability Level:** 90%
- **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- **Asphalt Structural Coefficient:** 0.41
- **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 4.1 was determined.

In this regard, we recommend the following flexible pavement section for the new improvements to Devon Avenue SE:

Material Type	Pavement Section (inches)
Asphaltic Concrete	5.0
Aggregate Base Rock	14.0

Local Residential Streets

The following documents and/or design input parameters were used to help determine the flexible pavement section design for new local residential streets:

- **Street Classification:** Local Residential Street
- **Design Life:** 25 years
- **Serviceability:** 4.2 initial, 2.5 terminal
- **Traffic Loading Data:** 100,000 18-kip EAL's
- **Reliability Level:** 90%
- **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- **Asphalt Structural Coefficient:** 0.41
- **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 2.6 was determined.

In this regard, we recommend the following flexible pavement section for the construction of new Local Residential Streets:

Material Type	Pavement Section (inches)
Asphaltic Concrete	4.0
Aggregate Base Rock	10.0

Wet Weather Grading and Soft Spot Mitigation

Construction of the proposed new public street improvements is generally recommended during dry weather. However, during wet weather grading and construction, excavation to subgrade can proceed during periods of light to moderate rainfall provided that the subgrade remains covered with aggregate. A total aggregate thickness of 8-inches may be necessary to protect the subgrade soils from heavy construction traffic. Construction traffic should not be allowed directly on the exposed subgrade but only atop a sufficient compacted base rock thickness to help mitigate subgrade pumping.

If the subgrade becomes wet and pumps, no construction traffic shall be allowed on the road alignment. Positive site drainage away from the street shall be maintained if site paving will not occur before the on-set of the wet season.

Depending on the timing for the project, any soft subgrade found during proof-rolling or by visual observations can either be removed and replaced with properly dried and compacted fill soils or removed and replaced with compacted crushed aggregate. However, and where approved by the Geotechnical Engineer, the soft area may be covered with a bi-axial geogrid and covered with compacted crushed aggregate.

Soil Shrink-Swell and Frost Heave

The results of the laboratory "R"-value tests indicate that the native subgrade soils possess a low to moderate expansion potential. As such, the exposed subgrade soils should not be allowed to completely dry and should be moistened to near optimum moisture content (plus or minus 3 percent) at the time of the placement of the crushed aggregate base rock materials. Additionally, exposure of the subgrade soils to freezing weather may result in frost heave and softening of the subgrade. As such, all subgrade soils exposed to freezing weather should be evaluated and approved by the Geotechnical Engineer prior to the placement of the crushed aggregate base rock materials.

Excavation/Slopes

Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations. Temporary excavations greater than about four (4) feet but less than eight (8) feet should be excavated with inclinations of at least 1 to 1 (horizontal to vertical) or properly braced/shored. Where excavations are planned to exceed about eight (8) feet, this office should be consulted. All shoring systems and/or temporary excavation bracing for the project should be the responsibility of the excavation contractor. Permanent slopes should be constructed no steeper than about 2H to 1V unless approved by the Geotechnical Engineer.

Depending on the time of year in which trench excavations occur, trench dewatering may be required in order to maintain dry working conditions if the invert elevations of the proposed utilities are located at and/or below the groundwater level. If groundwater is encountered during utility excavation work, we recommend placing trench stabilization materials along the base of the excavation. Trench stabilization materials should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock with a maximum particle size of 4 inches and less than 5 percent fines passing the No. 200 sieve. The material should be free of organic matter and other deleterious material and placed in a single lift and compacted until well keyed.

Surface Drainage/Groundwater

We recommend that positive measures be taken to properly finish grade the site so that drainage waters from the residential structures and landscaping areas as well as adjacent properties or buildings are directed away from the new residential structures foundations and/or floor slabs.

All roof drainage should be directed into conduits that carry runoff water away from the residential structures to a suitable outfall. Roof downspouts should not be connected to foundation drains. A minimum ground slope of about 2 percent is generally recommended in unpaved areas around the proposed new residential structures.

Groundwater was not encountered at the site in any of the exploratory test pits (TH-#1 through TH-#8) at the time of excavation to depths of at least 7 feet beneath existing site grades. Additionally, surface water ponding was not observed at the site during our field exploration work. However, the northeasterly portion of the site contains an existing seasonal drainage basin feature. Further, groundwater elevations in the area and/or across the subject property may fluctuate seasonally and may temporarily pond/perch near the ground surface during periods of prolonged rainfall.

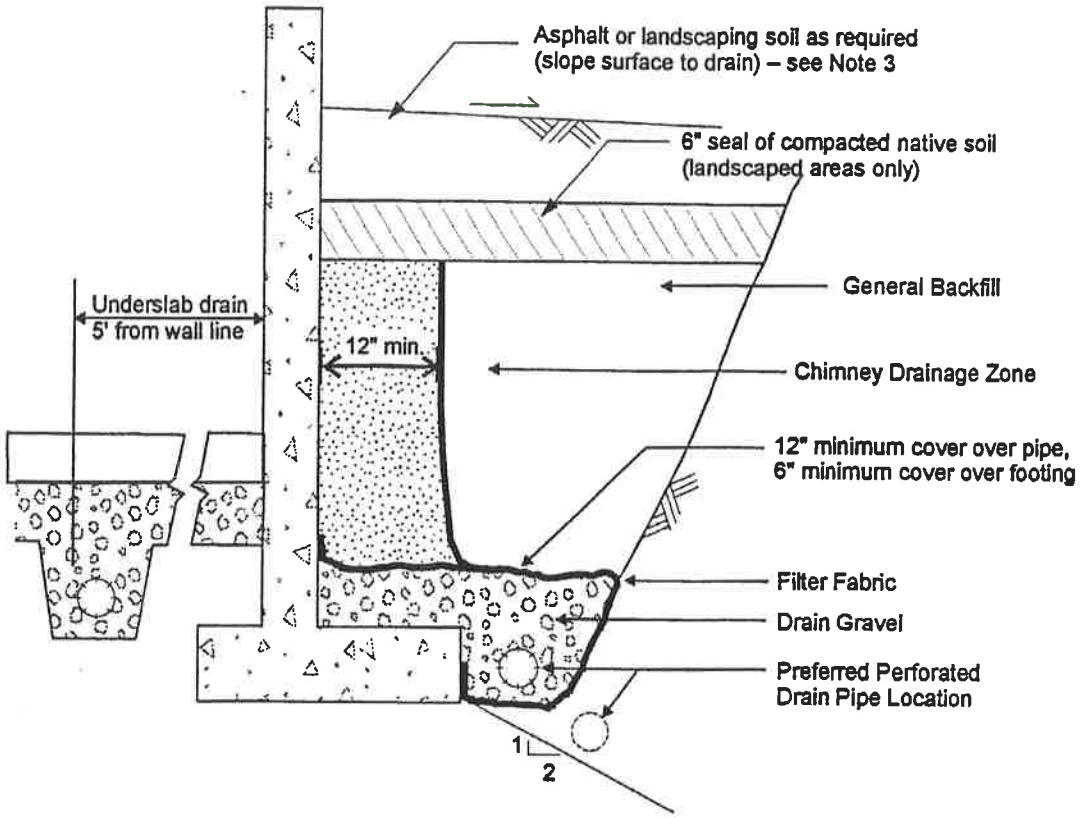
As such, based on our current understand of the possible site grading required to bring the subject site and/or residential lots to finish design grade(s), we are of the opinion that an underslab drainage system is not required for the proposed single-family residential structures. However, a perimeter foundation drain is recommended for any perimeter footings and/or below grade retaining walls. A typical recommended perimeter footing/retaining wall drain detail is shown on Figure No. 4. Further, due to our understanding that various surface infiltration ditches and/or swales may be utilized for the project as well as the relatively low infiltration rates of the near surface sandy, clayey silt subgrade soils anticipated within and/or near to the foundation bearing level of the proposed residential structures, we are generally of the opinion that storm water detention and/or disposal systems should not be utilized within the residential lots and/or around the proposed residential structures unless approved by the Geotechnical Engineer.

Design Infiltration Rates

Based on the results of our field infiltration testing, we recommend using the following infiltration rate to design any on-site near surface storm water infiltration and/or disposal systems for the project:

Subgrade Soil Type	**Recommended Infiltration Rate**
sandy, clayey SILT (ML)	0.3 to 0.4 inches per hour (in/hr)

Note: A safety factor of two (2) was used to calculate the above recommended design infiltration rate. Additionally, given the gradational variability of the on-site sandy, clayey sit subgrade soils beneath the site as well as the anticipation of some site grading for the project, it is generally recommended that field testing be performed during and/or following construction of any on-site storm water infiltration system(s) in order to confirm that the above recommended design infiltration rates are appropriate.



SCHEMATIC - NOT TO SCALE

NOTES:

1. Filter Fabric to be non-woven geotextile (Amoco 4545, Mirafi 140N, or equivalent)
2. Lay perforated drain pipe on minimum 0.5% gradient, widening excavation as required. Maintain pipe above 2:1 slope, as shown.
3. All-granular backfill is recommended for support of slabs, pavements, etc. (see text for structural fill).
4. Drain gravel to be clean, washed ¾" to 1½" gravel.
5. General backfill to be on-site gravels, or ¾""-0 or 1½"-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

PERIMETER FOOTING/RETAINING WALL DETAIL

Project No. 1001.052.G	DEVON AVENUE SUBDIVISION TL 300, 6719 DEVON AVENUE SE	Figure No. 4

Seismic Design Considerations

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the latest edition (2014) of the State of Oregon Structural Specialty Code (OSSC) and/or Amendments to the 2015 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Oregon Structural Specialty Code and/or from the National Earthquake Hazard Reduction Program (NEHRP) "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" published by the Building Seismic Safety Council.

We recommend Site Class "C" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from the 2015 IBC to determine the maximum considered earthquake spectral response acceleration for the project. However, we have assumed the following response spectrum for the project:

Table 1. Recommended Seismic Design Parameters

Site Class	S_S	S_1	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}
C	0.917	0.435	1.033	1.365	0.947	0.594	0.631	0.396

Notes: 1. S_S and S_1 were established based on the USGS 2015 mapped maximum considered earthquake spectral acceleration maps for 2% probability of exceedence in 50 years.

2. Fa and Fv were established based on IBC 2015 tables using the selected S_S and S_1 values.

CONSTRUCTION MONITORING AND TESTING

We recommend that **Redmond Geotechnical Services, LLC** be retained to provide construction monitoring and testing services during all earthwork operations for the proposed new residential development. The purpose of our monitoring services would be to confirm that the site conditions reported herein are as anticipated, provide field recommendations as required based on the actual conditions encountered, document the activities of the grading contractor and assess his/her compliance with the project specifications and recommendations. It is important that our representative meet with the contractor prior to any site grading to help establish a plan that will minimize costly over-excavation and site preparation work. Of primary importance will be observations made during site preparation and stripping, structural fill placement, footing excavations and construction as well as retaining wall backfill.

CLOSURE AND LIMITATIONS

This report is intended for the exclusive use of the addressee and/or their representative(s) to use to design and construct the proposed new single-family residential structures and their associated site improvements described herein as well as to prepare any related construction documents. The conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume that the explorations are representative of the subsurface conditions between the explorations and/or at other locations across the study area. The data, analyses, and recommendations herein may not be appropriate for other structures and/or purposes. We recommend that parties contemplating other structures and/or purposes contact our office. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report. Additionally, the above recommendations are contingent on Redmond Geotechnical Services, LLC being retained to provide all site inspections and constriction monitoring services for this project. Redmond Geotechnical Services, LLC will not assume any responsibility and/or liability for any engineering judgment, inspection and/or testing services performed by others.

It is the owners/developers responsibility for insuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

If during any future site grading and construction, subsurface conditions different from those encountered in the explorations are observed or appear to be present beneath excavations, we should be advised immediately so that we may review these conditions and evaluate whether modifications of the design criteria are required. We also should be advised if significant modifications of the proposed site development are anticipated so that we may review our conclusions and recommendations.

LEVEL OF CARE

The services performed by the Geotechnical Engineer for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in the area under similar budget and time restraints. No warranty or other conditions, either expressed or implied, is made.

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Appendix "A"

Test Pit Logs and Laboratory Test Data

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions at the site were explored by excavating eight (8) exploratory test pits (TH-#1 through TH-#8) on July 11, 2017. The approximate location of the test pit explorations are shown in relation to the proposed new residential lots and the associated site improvements on the Site Exploration Plan, Figure No. 2.

The test pits were excavated using track-mounted excavating equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test pits were excavated to depths ranging from about 5.0 to 6.0 feet beneath existing site grades. Detailed logs of the test pits are presented on the Log of Test Pits, Figure No's. A-5 through A-8. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-4.

The exploration program was coordinated by a field engineer who monitored the excavating and exploration activity, obtained representative samples of the subsurface soils encountered, classified the soils by visual and textural examination, and maintained continuous logs of the subsurface conditions. Disturbed and/or undisturbed samples of the subsurface soils were obtained at appropriate depths and/or intervals and placed in plastic bags and/or with a thin walled ring sample.

Groundwater was not encountered in any of the exploratory test pits (TH-#1 through TH-#8) at the time of excavating to depths of at least 6.0 feet beneath existing surface grades.

LABORATORY TESTING

Pertinent physical and engineering characteristics of the soils encountered during our subsurface investigation were evaluated by a laboratory testing program to be used as a basis for selection of soil design parameters and for correlation purposes. Selected tests were conducted on representative soil samples. The program consisted of tests to evaluate the existing (in-situ) moisture-density, maximum dry density and optimum moisture content, expansion index, gradational characteristics, and Atterberg Limits as well as direct shear strength and "R"-value tests.

Dry Density and Moisture Content Determinations

Density and moisture content determinations were performed on both disturbed and relatively undisturbed samples from the test pit explorations in general conformance with ASTM Vol. 4.08 Part D-216. The results of these tests were used to calculate existing overburden pressures and to correlate strength and compressibility characteristics of the soils. Test results are shown on the test pit logs at the appropriate sample depths.

Maximum Dry Density

Two (2) Maximum Dry Density and Optimum Moisture Content tests were performed on representative samples of the on-site sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This tests were conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-9.

Expansion Index

Two (2) Expansion Index tests were performed on representative samples of the near surface clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-4829-95. The tests were conducted to help evaluate the expansive properties of the near surface soils and their potential impact to residential foundations. The test results are presented on Figure No. A-9.

Atterberg Limits

Two (2) Liquid Limit (LL) and Plastic Limit (PL) tests were performed on representative samples of the sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-4318-85. These tests were conducted to facilitate classification of the soils and for correlation purposes. The test results appear on Figure No. A-10.

Gradation Analysis

Two (2) Gradation analyses were performed on representative samples of the sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-422. The test results were used to classify the soil in accordance with the Unified Soil Classification System (USCS). The test results are shown graphically on Figure No. A-11.

Direct Shear Strength Test

One (1) Direct Shear Strength test was performed on an undisturbed and/or remolded sample at a continuous rate of shearing deflection (0.02 inches per minute) in accordance with ASTM Vol. 4.08 Part D-3080-79. The test results were used to determine engineering strength properties and are shown graphically on Figure No. A-12.

"R"-Value Tests

Two (2) "R"-value tests were performed on remolded sandy, clayey silt subgrade soil samples in accordance with ASTM Vol. 4.08 Part D-2844. The test results were used to help evaluate the subgrade soils supporting and performance capabilities when subjected to traffic loading. The test results are shown on Figure No. A-13.

The following figures are attached and complete the Appendix:

Figure No. A-4	Key To Exploratory Test Pit Logs
Figure No's. A-5 through A-8	Log of Test Pits/Dynamic Cone
Figure No. A-9	Maximum Density & Expansion Index Test Results
Figure No. A-10	Atterberg Limits Test Results
Figure No. A-11	Gradation Test Results
Figure No. A-12	Direct Shear Strength Test Results
Figure No. A-13	Results of "R"-Value Tests
Figure No's. A-14 and A-15	Field Infiltration Test Results

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

DEFINITION OF TERMS

	U.S. STANDARD SERIES SIEVE			CLEAR SQUARE SIEVE OPENINGS				
	200	40	10	4	3/4"	3"	12"	
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS	
	FINE	MEDIUM	COARSE	FINE	COARSE			

GRAIN SIZES

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT†
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

CLAYS AND PLASTIC SILTS	STRENGTH‡	BLOWS/FOOT†
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

†Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

REDMOND GEOTECHNICAL SERVICES
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KEY TO EXPLORATORY TEST PIT LOGS
Unified Soil Classification System (ASTM D-2487)

DEVON AVENUE SUBDIVISION
TL 300, 6719 DEVON AVENUE SE

PROJECT NO.	DATE	
1001.052.G	8/11/17	Figure A-4

BACKHOE COMPANY: Gene S McMurrin BUCKET SIZE: 24 inches DATE: 7/11/17

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
TEST PIT NO. TH-#1 ELEVATION						
0					ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
	X			21.1		
					ML	Medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
	X			14.8		
5					SM/RK	Medium to orangish-brown, moist, medium dense to dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						
TEST PIT NO. TH-#2 ELEVATION						
0					ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
	X			33.5		
					ML	Medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
5					SM/RK	Medium to orangish-brown, moist to very moist, medium dense to dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.052.G | DEVON AVENUE SUBDIVISION | FIGURE NO. A-5

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 7/11/17

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#3 ELEVATION
0					ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
	X			26.6	ML	Medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
	X			15.1		
5					SM/RK	Medium to orangish-brown, moist to very moist, medium dense to dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						
						TEST PIT NO. TH-#4 ELEVATION
0					ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
	X			20.5	ML	Medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
5					SM/RK	Medium to orangish-brown, moist, medium dense to dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.052.G | DEVON AVENUE SUBDIVISION | FIGURE NO. A-6

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 7/11/17

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#5 ELEVATION
0					ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
5					SM/RK	Medium to orangish-brown, moist to very moist, medium dense to dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						
						TEST PIT NO. TH-#6 ELEVATION
0					ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
5					SM/RK	Medium to orangish-brown, moist, medium dense to dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.052.G | DEVON AVENUE SUBDIVISION | FIGURE NO. A-7

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 7/11/17

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#7 ELEVATION
0					ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT
5					SM/RK	Medium to orangish-brown, moist, medium dense to dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						
						TEST PIT NO. TH-#8 ELEVATION
0	X			19.9	ML	Dark brown, moist, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, moust to very moist, medium stiff to stiff, sandy, clayey SILT
5					SM/RK	Medium to orangish-brown, moist, medium dense to dense, clayey, silty SAND to highly weathered bedrock
						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.052.G | DEVON AVENUE SUBDIVISION | FIGURE NO. A-8

MAXIMUM DENSITY TEST RESULTS

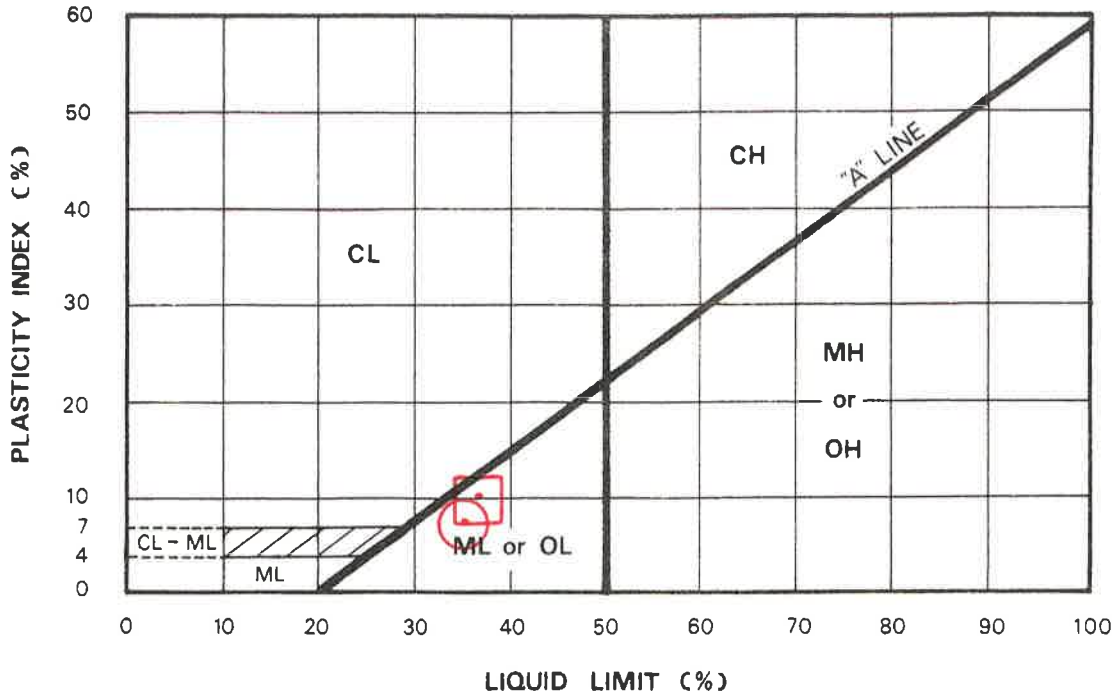
SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#1 @ 1.5'	Medium to reddish-brown, sandy, clayey SILT (ML)	102.0	28.0
TH-#2 @ 1.5'	Medium to reddish-brown, sandy, clayey SILT (ML)	100.0	30.0

EXPANSION INDEX TEST RESULTS

SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (%)	EXPANSION INDEX	EXPANSIVE CLASS.

MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1001.052.G | DEVON AVENUE SUBDIVISION | FIGURE NO.: A-9



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
⊙	TH-#1	1.5	21.1	35.0	7.2	90.9		ML
▣	TH-#2	1.5	33.5	37.2	10.1	86.3		ML

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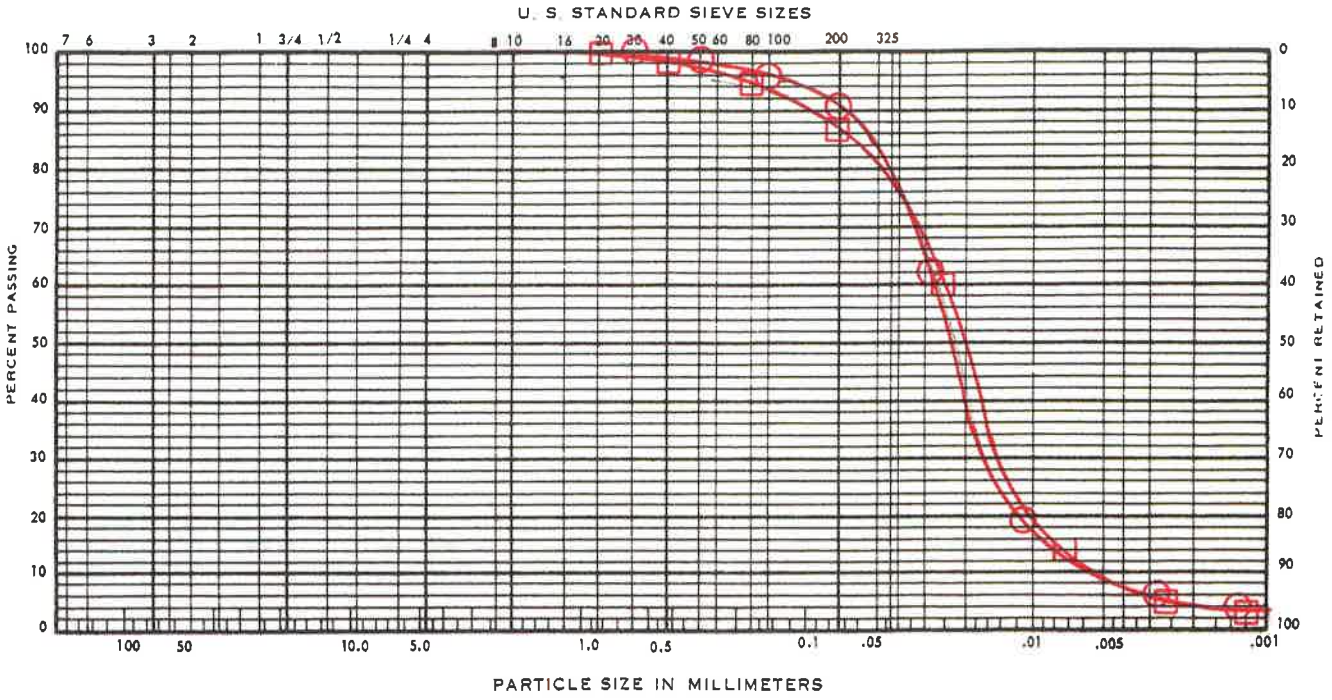
PLASTICITY CHART AND DATA

DEVON AVENUE SUBDIVISION
TL 300, 6719 DEVON AVENUE SE

PROJECT NO.	DATE	
1001.052.G	8/11/17	Figure A-10

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 422-72)



COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	ELEV. (feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
⊖	TH-#1	1.5		ML	Medium to reddish-brown, sandy, clayey SILT
⊟	TH-#2	1.5		ML	Medium to reddish-brown, sandy, clayey SILT

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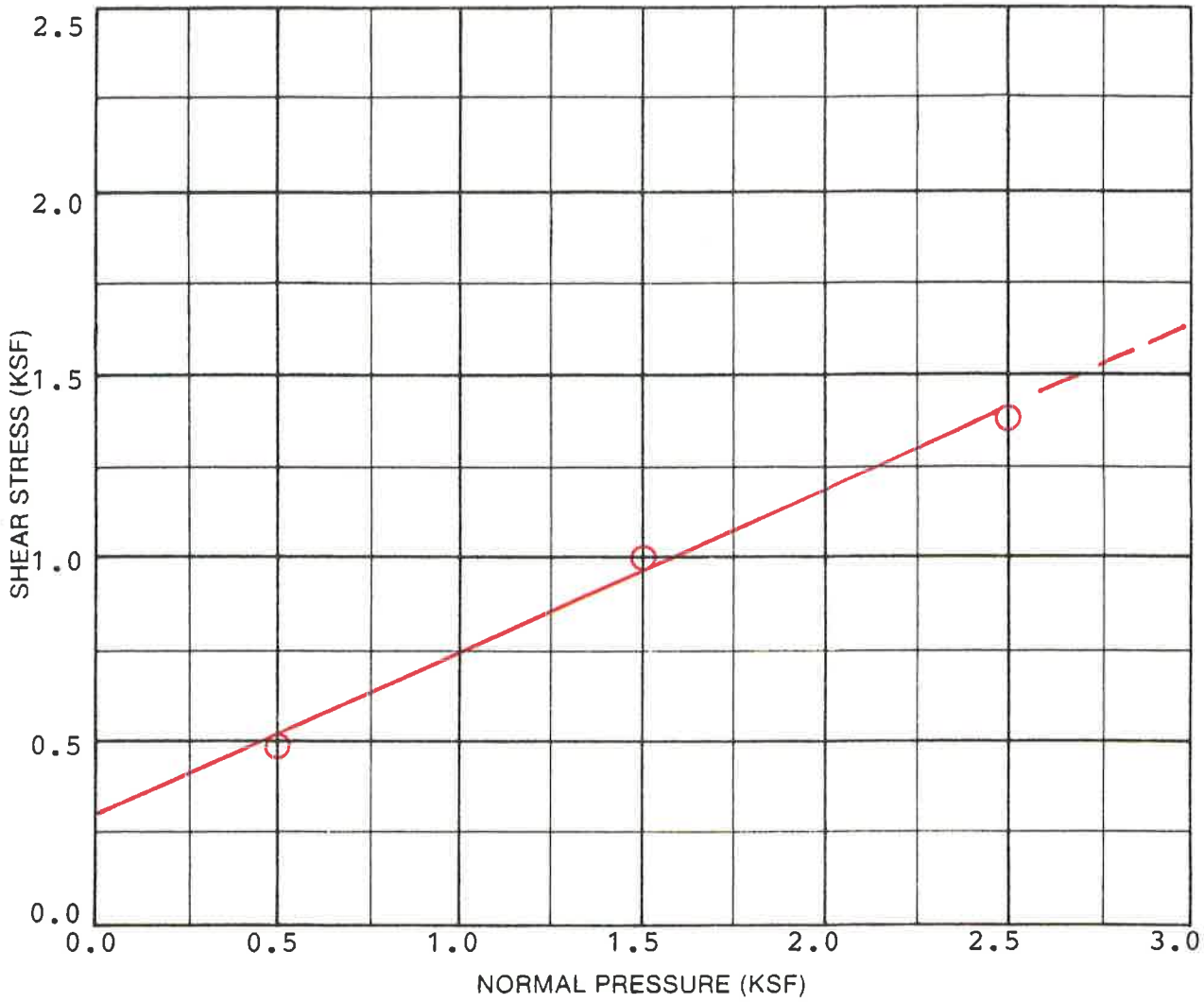
GRADATION TEST DATA

DEVON AVENUE SUBDIVISION
TL 300, 6719 DEVON AVENUE SE

PROJECT NO.	DATE	FIGURE A-11



SAMPLE DATA	
DESCRIPTION: Medium to reddish-brown, sandy, clayey SILT (ML) (Remolded)	
BORING NO.: TH-#2	
DEPTH (ft.): 1.5'	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 300 psf	
APPARENT ANGLE OF INTERNAL FRICTION (Φ): 22°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.5	1.0	1.4	
INITIAL H_2O CONTENT (%)	30.0	30.0	30.0	
FINAL H_2O CONTENT (%)	30.6	26.1	20.9	
INITIAL DRY DENSITY (PCF)	90.0	90.0	90.0	
FINAL DRY DENSITY (PCF)	90.4	94.6	99.8	
STRAIN RATE: 0.02 inches per minute				

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DIRECT SHEAR TEST DATA

DEVON AVENUE SUBDIVISION
TL 300, 6719 DEVON AVENUE SE

PROJECT NO	DATE	Figure A-12

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#1

SAMPLE DEPTH: 1.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	219	329	431
Expansion Dial (o.ooo1")	0	1	3
Expansion Pressure (psf)	0	3	9
Moisture Content (%)	30.6	27.4	22.1
Dry Density (pcf)	93.4	98.2	102.6
Resistance Value, "R"	17	29	37
"R"-Value at 300 psi Exudation Pressure = 28			

SAMPLE LOCATION: TH-#2

SAMPLE DEPTH: 1.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	208	326	439
Expansion Dial (0.0001")	0	2	5
Expansion Pressure (psf)	0	6	15
Moisture Content (%)	32.3	28.6	23.9
Dry Density (pcf)	92.1	96.1	100.7
Resistance Value "R"	15	27	35
"R"-Value at 300 psi Exudation Pressure = 26			

Division 004 Appendix C - Infiltration Testing

Location: TL 300, 6719 Devon Avenue SE	**Date:** July 11, 2017	**Test Hole:** TH-#2
Depth to Bottom of Hole: 3.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	Soil Characteristics
0-0.5	Dark brown Topsoil
0.5-3.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
10:00	0	24.00	----		Filled w/12" water
10:20	20	24.50	0.50	1.50	
10:40	20	24.92	0.42	1.26	
11:00	20	25.27	0.35	1.05	
11:20	20	25.57	0.30	0.90	
11:40	20	25.83	0.26	0.78	
12:00	20	26.06	0.23	0.69	
12:20	20	26.27	0.21	0.63	
12:40	20	26.47	0.20	0.60	

Infiltration Test Data Table

Division 004 Appendix C - Infiltration Testing

Location: TL 300, 6719 Devon Avenue SE	**Date:** July 11, 2017	**Test Hole:** TH-#3
Depth to Bottom of Hole: 2.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	Soil Characteristics
0-0.5	Dark brown Topsoil
0.5-2.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
10:20	0	12.00	----		Filled w/12" water
10:40	20	12.70	0.70	2.10	
11:00	20	13.22	0.52	1.56	
11:20	20	13.66	0.44	1.32	
11:40	20	14.04	0.38	1.14	
12:00	20	14.37	0.33	0.99	
12:20	20	14.67	0.30	0.90	
12:40	20	14.95	0.28	0.84	
1:00	20	15.22	0.27	0.81	

Infiltration Test Data Table

Appendix "B"

Geologic Hazard Assessment

NORTHWEST GEOLOGICAL SERVICES, INC.
Consulting Geologists and Hydrogeologists
2505 N.E. 42nd Avenue, Portland, Oregon 97213-1201
503-249-1093 ngs@teleport.com

Redmond Geotechnical Services, LLC 27 July 2017
P.O. Box 20547
Portland, Oregon 97294

Attn: Dan Redmond

Geological Assessment
8S/3W-22C Tax Lot 300
Salem, Marion County, Oregon

Dear Dan:

The purpose of this letter is to present Northwest Geological Services, Inc. (NGS') Geologic Hazard Assessment for the above referenced property. This study includes the engineering geology tasks required by Salem and Marion County to develop in areas that appear to have potential geologic hazards. We understand that our services are in support of your client's efforts to partition and develop the property for residential use. The current proposal is to partition the site into approximately 80 residential lots with access streets, and infrastructure as needed. The work for this study was done in accordance with your email authorization of 28 June 2017.

1. SCOPE OF STUDY

The scope of our study was limited to the engineering geologic consultation necessary to assess potential slope hazards, as required by Salem and Marion County. Specifically, our work included:

- Obtain and review LIDAR and historic aerial photographs of the site;
- Obtain and review well logs for the site area;
- Review available geologic and geologic hazard investigations of the site and site area;
- Conduct a geologic reconnaissance of the site and adjacent area;
- Evaluate the potential landslide hazards using the information developed; and,
- Prepare this letter describing our work, findings and recommendations.

Our work did not include some items the County may request for Geological Assessments of slope hazard areas. Specifically, the excluded items are: site grading plans showing cuts and fills; and geologic cross sections showing subsurface conditions. We understand the grading plan will be developed as part of the plans for the building permit application for the site. In our opinion, the geology of the site is simple (Sections 3 and 4) and a cross section is not required to comprehend the subsurface conditions. Nor is a grading plan necessary to asses the stability of the natural slopes. However, those items should be developed in the Geotechnical report for the site.

2. SITE SETTING

The site is south of Battle Creek, north of Rees Hill Rd. and west of Devon Ave SE (Figures 1 and 2) in the northeast corner of the southwest quarter of Section 22, T8S/R3W. The 19.89 acre property is currently accessed from Devon Ave SE. City of Salem Zoning Map 8322S (S ½ 22-8S3W) shows the site is zoned RA (Residential Agricultural). The site is in the South Salem Hills Ground Water Limited Area.

2.1 Location and Physiography

The property is south of Battle Creek and straddles the summit of Reese Hill, a NE-SW ridge that extends from Hylo Rd. SE northeast to Battle Creek (Figures 1 and 2). An intermittent tributary of Battle Creek extend N-S just west of the site. Elevations at the site range from about 652 ft in the south central area (i.e., Reese Hill) down to about 546 ft at the NW corner (Figures 1 and 4). Overall slopes are gentle, but local steep areas occur (Figures 1, 2, 4 and Section 3.2). The overall slope west from the crest of Reese Hill averages 11% to 14%. The slope east towards Devon Ave SE averages 4% to 7%. Both west and east slopes have locally irregular topography with small scale areas of slope up to 30% or rarely 50% (Figure 4). These declivities are 4 to 6 ft high by 20 to 50 ft wide mounds. They lack corresponding uphill depressions as one would expect of slope failures.[1]

The west northeast parts of the site – the areas with irregular topography – are currently covered by mixed conifer and deciduous trees with understory brush (Figure 2). The mature trees have significant root mounds because of the shallow site soils (Figures 4 and 7). Additionally, many of the irregularities appear to be remnants of the former prune orchards. The central and east parts of the site are cleared field with scattered mature Douglas Firs in the SW corner. The existing residence and outbuildings are located in the SW corner (Figure 2).

There are no drainage ways on the site. Drainage is by sheet runoff and via small, shallow declivities developed during past logging and farming of the property. Drainage is towards an unnamed tributary of Battle Creek[2] located about just west of the site and Powell Creek east of the site (Figures 1).

The geology of the area around the site (Section 2.3) is very well known. It was mapped by the State (Bella, 1981), for a Portland State MS thesis (Hoffman, 1981), by the USGS (Beeson and Tolan, 2001) and by us for Chinook Estates and Marion County (NGS, 1994, 1997). Figure 5 shows our mapping of the area around the site.[3] All studies found the site underlain by Miocene volcanic rocks of the Columbia River Basalt (Figures 3 and 4) and pre-basalt sedimentary strata at considerable depth (Section 2.3).

2.2 Site Area Geology

The site lies on the north flank of the west Salem Hills. These hills are an anticlinal uplift that extends from the Willamette River north about 10 miles to Salem and from the

1 The detailed topography (Figures 4 and 7) is interpreted from LIDAR flown for DOGAMI in 2009. Reconnaissance and digital images indicate that site clearing has smoothed or removed some irregularities.

2 Informally called Champion Swale by the City and shown as that on the City LIDAR (figures 4 and 7).

3 The geologic interpretation shown on Figure 5 is based on surficial geologic mapping, aerial photo interpretation, and our evaluation of over 200 water well logs (NGS, 1997). Identification of basalt flows is based on our previous experience and several chemical analyses done by Hoffman (1981).

river to east of I-5. Bella (1981) mapped the site as underlain by Columbia River Basalt (CRB), in substantial agreement with earlier mapping by the U.S. Geological Survey (Fox-worthy, 1970). Both studies found the CRB to be at least 350 ft thick in the site area. Our mapping (Figure 5) and review of well logs in the area indicates that the CRB is at least 350 to 400 ft thick beneath the site. The review also suggests the basalt dips gently north and northeast towards Salem.

Mapping by Hoffman (1981) identified the individual flows within the CRB. The east part of the site is underlain by the youngest hi-magnesium flow of the Grande Ronde Basalt (now known as the Sentinel Bluffs). The Winter Water flow of the Grande Ronde Basalt underlies the steeper slopes marginal to the unnamed drainage west of the site (Figures 4 and 5).

In the site area, the upper few feet of the basalt bedrock underlying ridges and hillsides is generally weathered or decomposed to a hard, red-brown gritty, silty clay or clayey silt. However, the original volcanic texture of the basalt is preserved by the weathering. Thus, the basalt is generally recognizable, even when decomposed.[4] The distinction is important, because the marine sedimentary strata are often involved in slope failures. The weathered basalt is generally not involved in slope failures, except where its physical properties have been ignored during development.

2.3 State and City Estimates of Landslide Hazard Susceptibility

The State conducted assessments of potential landslide hazards for Salem and Marion County. These included notably slide-prone parts of the area (OFR O-77-4 by Schlicker, 1977), the west Salem Hills (IMS-6 by Harvey and Peterson, 1998) and IMS-17 (Hofmeister and others, 2000). These assessments were based on the available geologic studies, including the aforementioned NGS studies, information about soils strength and GIS modeling using the USGS topographic DEM.

IMS-6 does not extend east to the site nor is the site in an active or inactive landslide area (e.g., as defined by OFR O-77-4). IMS-17 estimates the site ranges from very low to moderate relative risk of earth-induced landsliding (Figure 6). The latest State estimates are incorporated into SLIDO[5] that shows no nearby active, historic or prehistoric landslides. However, the SLIDO landslide susceptibility map shows no to moderate landslide hazard in agreement with IMS-17 estimates.

In our experience, IMS-6 estimates for water induced landslide risks in areas similar to the site are generally Category 4. Thus, were this site within the area covered by IMS-6, it would likely have a low to moderate risk of water induced landslides.

The City of Salem provides Landslide Hazard Maps based on slope (generally from LIDAR) and available risk assessments from various government sources (Figure 7). Salem's map assigns 2 to 3 landslide hazard points (low to moderate) to the site.

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4 However, the relict volcanic texture in soils derived from weathering of volcanic units can be hard to see on a cloudy or rainy day. Thus, some investigators have incorrectly mapped decomposed Columbia River Basalt as weathered tuffaceous sedimentary strata, Willamette Silt, or even landslide deposits.

5 SLIDO is Oregon State DOGAMI's Statewide Landslide Information Layer for Oregon. SLIDO compiles available DOGAMI & USGS geologic and hazard mapping: http://www.oregongeology.org/sub/slido/

3. SITE SPECIFIC STUDIES

3.1 Previous Site Development

We reviewed available historic topographic maps and aerial photographs[6] for indications of slope failures at and near the site. The aerial photographs were also reviewed to identify potential areas of cut or fill made during previous use or development of the site.

The maps and photos show that the site has a long history of use as orchard and pasture. The 1936 (Figure 8) and 1944 aerial photos, and the 1950s topographic map and aerial photos show the site as mostly prune orchard typical of the area (Meyering, 2008) with a residence in the southwest part. By 1955, the summit area was cleared of trees and used as pasture. The 1967 photos show the orchard was mostly cut with a few conifers starting in the NW and along Devon Ave SE. A few fruit tree remained along the west end of the site. Most of the site appears fallow and unused in 1967 and in 1976. However, the 1971 and 1985 photos show the east 1/3 of the site around the residence mowed, presumably for hay and/or fire control. The remainder appears to be brush and conifers. The 1990 and 1994 images show only a 2 to 3 acre area around the residence was maintained as yard. The remainder was brush and maturing conifers with a few trails cleared through the site. Digital imagery shows that from 2010 through the summer of 2016 the site was progressively cleared, trees thinned and topography between the trees smoothed.

In summary, the historic maps and photos show the site has been farm and/or orchard with a residence since the 1930s. Properties north, east and south have also been small farms and/or low-density residential. Property to the west has been intermittently logged and cleared as wood lot.

No signs of slope instability or failure were observed on the aerial photographs we examined. The resolution of the aerial photos is adequate to see vehicles on roads and relatively minor earthworks. Consequently, we believe that any significant slope failure should have been identifiable on the photos we reviewed.

3.2 Surface and Subsurface Observations

We conducted a walking reconnaissance of the site, and observed road cuts and accessible excavations in the site neighborhood. As noted, we previously mapped the site area and also conducted an assessment of TL 200 immediately north (NGS, 2008), so we reviewed maps and notes our previous work for this study.

At the highest site elevations, the surficial soils are mostly derived from weathering of the basalt bedrock with an admixture of loess blown up from the Willamette floodplain. These soils generally have a thin topsoil of fine to medium sandy clayey SILT with abundant organic material and occasional pebbles, cobbles and boulders of weathered rock. On slopes below the ridgetop, soils are decomposed basalt: red brown, stiff to hard silty sandy CLAY to sandy clayey SILT with sparse to abundant rock fragments.

Four test pits were excavated on 11 July 2017 to assess site soils (Figure 7). TP-1, -2 and -4 found severely weathered Sentinel Bluffs basalt at 3 ft, 3.5 ft and 2.5 ft, respectively. TP-3 found small boulders of weathered Winter Water basalt at 1 ft. The boulders were in a

6 Stereo pairs of aerial photos taken in 1936, 1944, 1955, 1967, 1971, 1976, 1985, 1990 and 2000 were reviewed. We also reviewed USGS, Google Earth and Earth Explorer imagery from 1994-2016.

matrix of severely weathered to decomposed basalt. Practical refusal with the small excavator was reached at depths of 3 to 5 ft in all test pits.

Soils above the weathered basalt were 1 to 2 ft of medium, red brown fine sandy SILT (loess) in TP-1 and -4. Weathered basalt in TP-3 was overlain by medium, red brown clayey SILT that graded to stiff at 2.5 ft and to hard decomposed basalt from 2.5 to 3.5-4 ft. In TP-4 the basalt was overlain by 1 ft of organic SILT topsoil.

The complete natural weathering profile is exposed north of the site in an excavation along Sahalee Dr SE. (Figure 4). About 1/4 mile north of the site, excavations for Lone Oak Rd SE expose unweathered Sentinel Bluffs and Winterwater basalt at depths of 8 to 15 ft.

In summary, soils exposed in the test pits at the site are consistent with soils in the surrounding area: thin, competent surficial soils derived mostly from in-situ weathering of the basalt with an admixture of loess along ridges and uplands. Valley soils are also competent, thin and a mix of colluvium and decomposed basalt.

3.3 Ground Water Observations

No seeps or springs were observed at the site. Springs have been reported at the contact between Sentinel Bluffs and Winterwater Basalt on neighboring properties. The nearest recorded spring is south of the site just south of Rees Hill Rd SE.

Driller's logs of nearby wells indicate that the regional water table is below the elevation of the Battle Creek flood plain. Most wells have modest yields and depths to water of 170 to 250 ft. However, local perched zones occur between the basalt flows. Such perched zones supply the aforementioned springs.

We suspect that the relatively clayey, moderately-low-permeability soils saturate quickly during heavy precipitation. That is because the severely weathered top of the basalt bedrock is shallow and relatively impervious compared to the overlying soils.

4. Interpretation of Site Conditions

The reconnaissance, test pits, area roadcuts and excavations, and historic aerial photographs, confirm that the site is underlain by bedrock consisting of Columbia River Basalt, as mapped previously (NGS, 1994, 1997, 2008). The top of the Basalt is severely weathered to decomposed, but it is still relatively competent material. The relatively fresh bedrock on the slopes is typically covered by 3 to 5 ft of soils (decomposed rock). However, basalt crops out locally and may be found near ground surface anywhere in the immediate site area.[7]

Typically, the soils derived from the basalt creep on moderate and steep slopes. Curved and pistol-butted trees are present on the steepest slopes at the site and these trees are consistent with soil creep. However, the 60 to 80 year-old firs are erect at the tops, indicating that creep is slow enough for the trees to keep up with it.

Available information indicates that the Basalt extends for 350 to 400 ft depth below the site. Site mapping indicates that the flows dip gently north beneath the site. Together

[7] Except that bedrock is at greater depths below the alluvium of the Battle Creek floodplain, north of the site, and its tributaries east and west of the site.

with the competent nature of the site materials, it seems most unlikely to us that there is any significant risk of slope failure involving the bedrock. This interpretation is **supported** by a complete lack of evidence that the site has suffered from slope instability in the past.

However, a few cuts for roads in the area have failed locally where they were too steep or became locally saturated during intense storms. Additionally, our previous experience in the area indicates that careful design and construction is required to use local soils as fill.

5. Conclusions and Recommendations

We found no evidence that slopes at the site have ever failed, nor indication that they will fail under the expected range of future conditions. The site and neighbors have survived severe rainfall events in 1964, 1974, 1994, 1996, 2003, 2006 and most recently December 2016. Numerous slides occurred at other sites in the Salem region during these severe storms.

Even though the site soils have not failed, they do creep, and similar soils have failed locally in overly-steep excavations (NGS, 2008). Consequently, we recommend that foundations be placed on competent material. Foundations and retaining walls should be designed by a qualified professional to withstand forces from soil creep and lateral loads from earthquakes. Given the thin soils and shallow depth to weathered bedrock, this requirement should not be onerous.

Cuts higher than 4 ft and steeper than 1V:2H, and fills more than 2 ft thick, should be designed by a qualified professional and the design reviewed by a geotechnical engineer. Walls, including retaining walls or foundation walls higher than 4 ft, should also be designed by a qualified professional and the design reviewed by a geotechnical Engineer.

Additionally, we recommend against infiltration of large volumes of water into the small volumes of ground, particularly during intense rainfall events such as those noted above. Some slope failures in the Eola and Salem Hills have been caused by injection of large volumes of storm water. Consequently, it is our opinion that surface runoff from roofs and pavements should be dispersed over a broad area to simulate natural conditions. If it is found necessary to dispose of large amounts runoff to the soil, the location and method should be thoroughly evaluated by a qualified professional. Your geotechnical Engineer should also review any such plans.

In our opinion, if you follow the above recommendations, partitioning and development of TL 300 for single family residences as you propose (Figure 9) should not increase the potential for slope hazards on the site or adjacent properties.

6. LIMITATIONS AND LIABILITY

We call your attention to the paragraphs on Warranty and Liability in the General Conditions (dated 1/2016) approved previously by you. Interpretations and recommendations presented herein are based on limited data and observations. Actual subsurface conditions may vary from those inferred from the limited information available to us. If site excavations for development find conditions to differ significantly from those inferred

herein, you should contact us and provide an opportunity for us to review our recommendations for the site.

We thank you for the opportunity to assist you with your project. Please contact us if you have questions about the report.

Yours very truly,
Northwest Geological Services, Inc.

Clive F. (Rick) Kienle, Jr., PhD, CEG
Principal Geologist and Vice President

NGS Reference 235.96-1

7. REFERENCES

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8S/3W-22C Tax Lot 300		
Salem, Marion County, Oregon		
Landslide Hazard Study		
	Site Location Map	
NGS, Inc.	July 2017	Figure 1

8S/3W-22C Tax Lot 300
Salem, Marion County, Oregon
Landslide Hazard Study
Oblique View 23 July 2016 Aerial Image

NGS, Inc.	July 2017	Figure 2

N E SSE

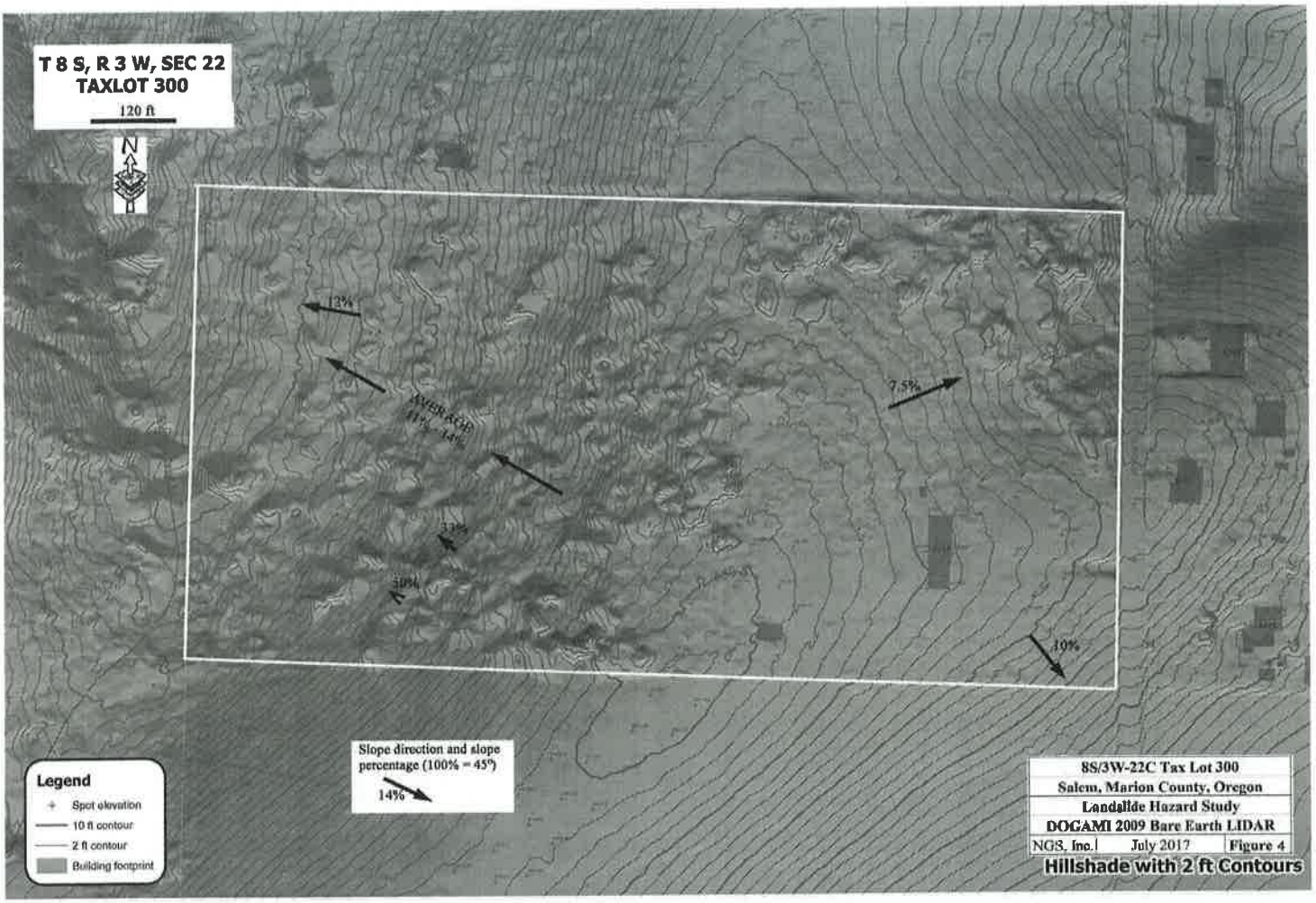
Panorama of central part of site looking N (on left) and around to SSE (on right). Note brush mound at far right behind vehicle.

Mature Douglas Firs at the East end of the site show slight curvature from moderate creep in the thin site soil.

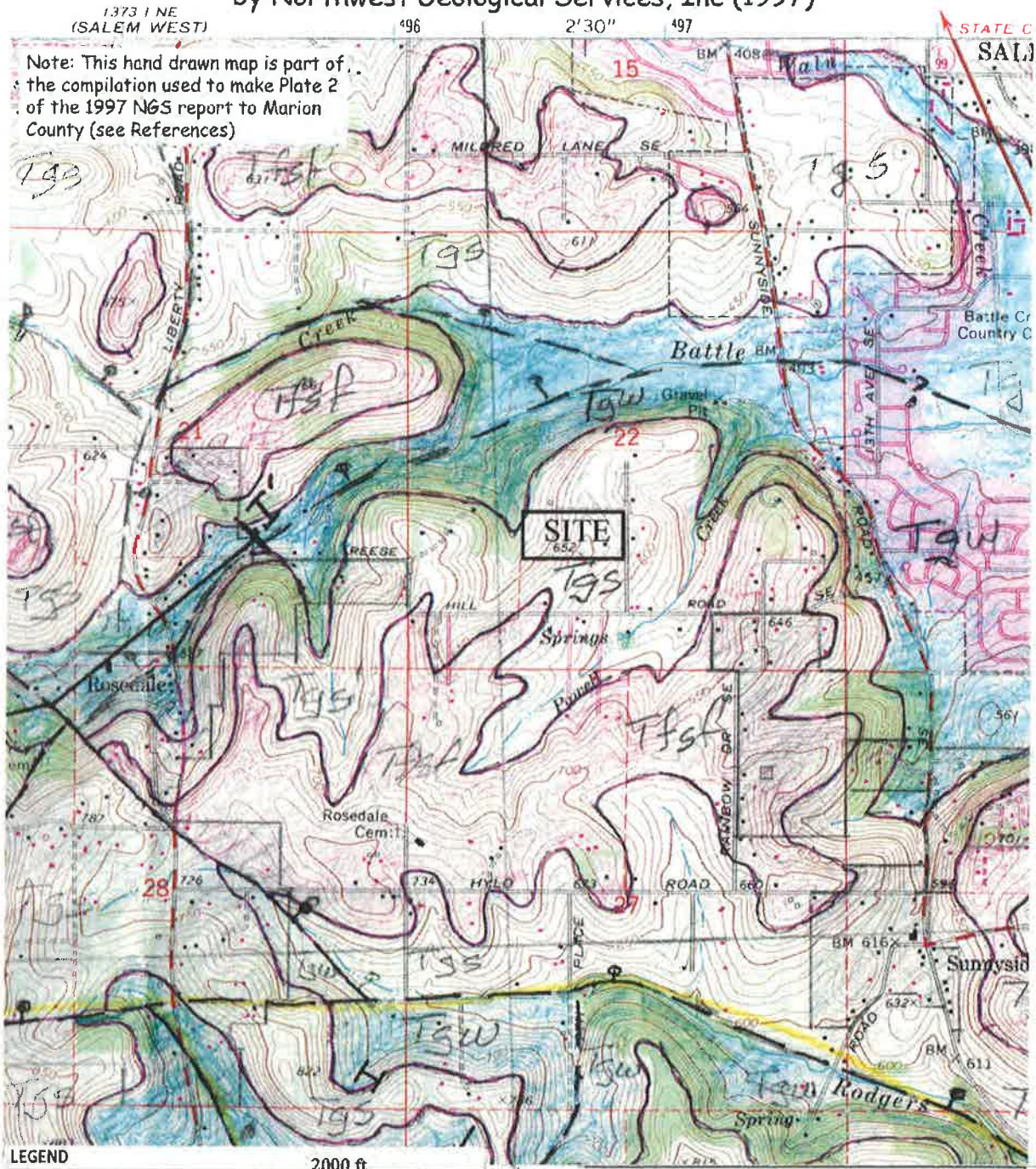
Smooth topography in NE part of site is typical of the entire site with exception of brush-covered piles of soil and woody debris left from clearing old orchard.

Decomposed Sentinel Bluff basalt spoils from test pit TP-1

8S/3W-22C Tax Lot 300		
Salem, Marion County, Oregon		
Landslide Hazard Study		
Site Photographs		
NGS, Inc.	July 2017	Figure 3


T 8 S, R 3 W, SEC 22
TAXLOT 300
120 ft
N
13%
AVERAGE
11% - 14%
33%
30%
7.5%
10%
Slope direction and slope percentage (100% = 45°)
14%
Legend
Spot elevation
10 ft contour
2 ft contour
Building footprint
8S/3W-22C Tax Lot 300
Salem, Marion County, Oregon
Landslide Hazard Study
DOGAMI 2009 Bare Earth LIDAR
NG3, Inc. | July 2017 | Figure 4
Hillshade with 2 ft Contours

Portion of Geologic Map of Sidney Quadrangle by Northwest Geological Services, Inc (1997)



LEGEND

2000 ft

N

Geologic Bedrock Units

Tfs - Basalt of Silver Falls (Frenchman Springs Mamber, Wanapum Basalt)

Tfg - Basalt of Gingko (Frenchman Springs Mamber, Wanapum Basalt)

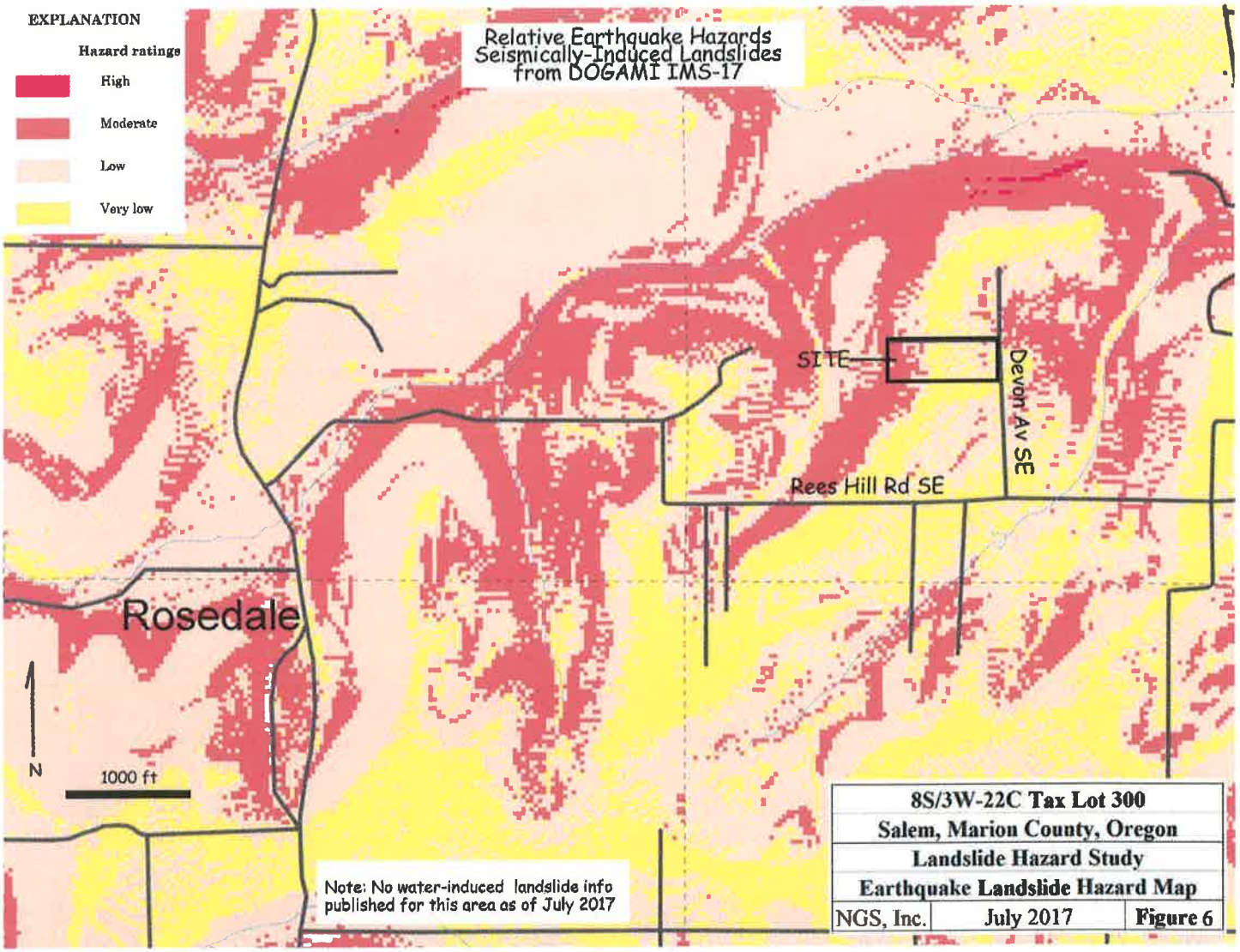
Tgs - Sentinel Bluff Basalt (Grand Ronde Basalt)

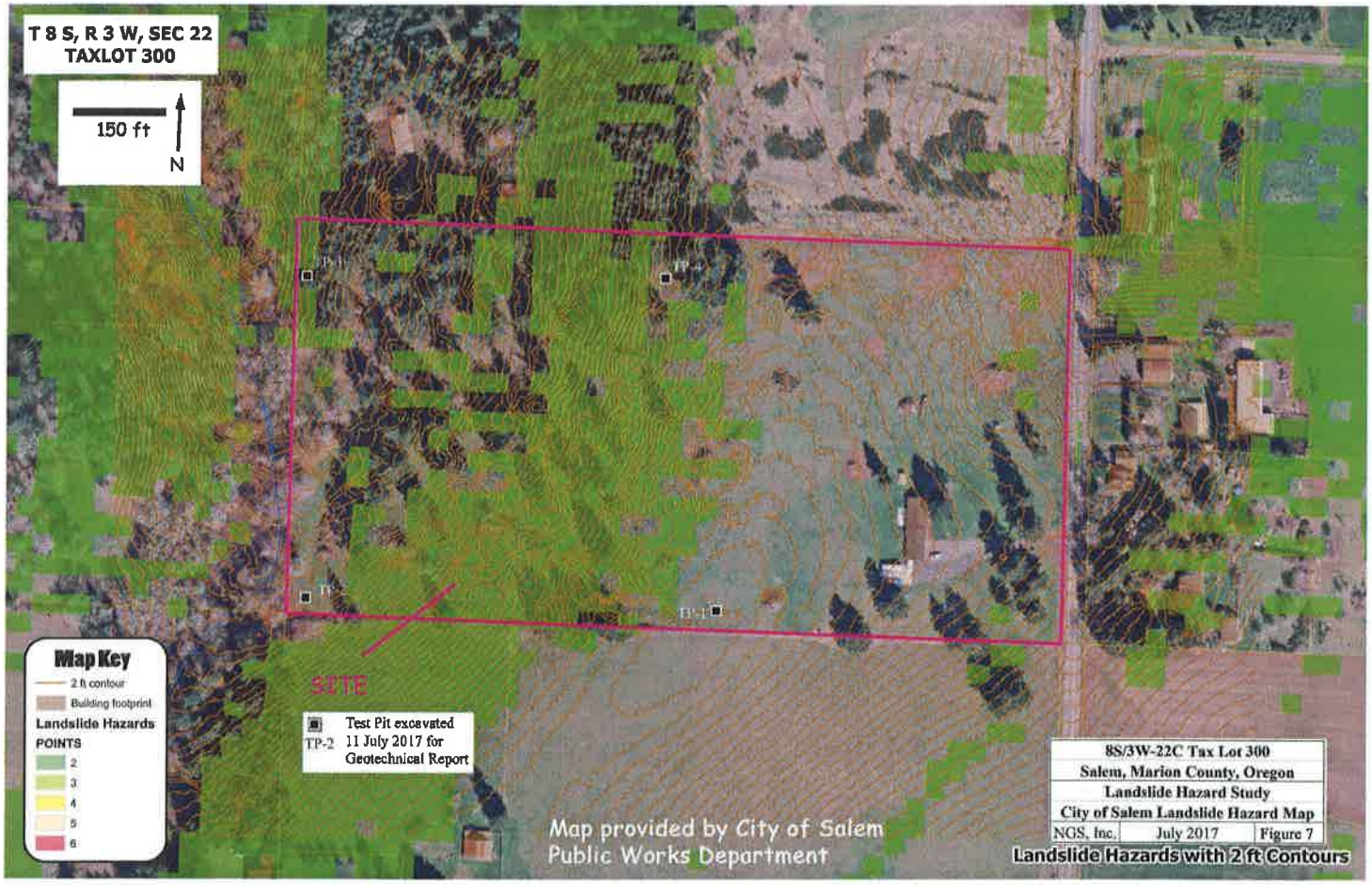
Tgw - Winterwater Basalt (Grande Ronde Basalt)

Geologic Structures

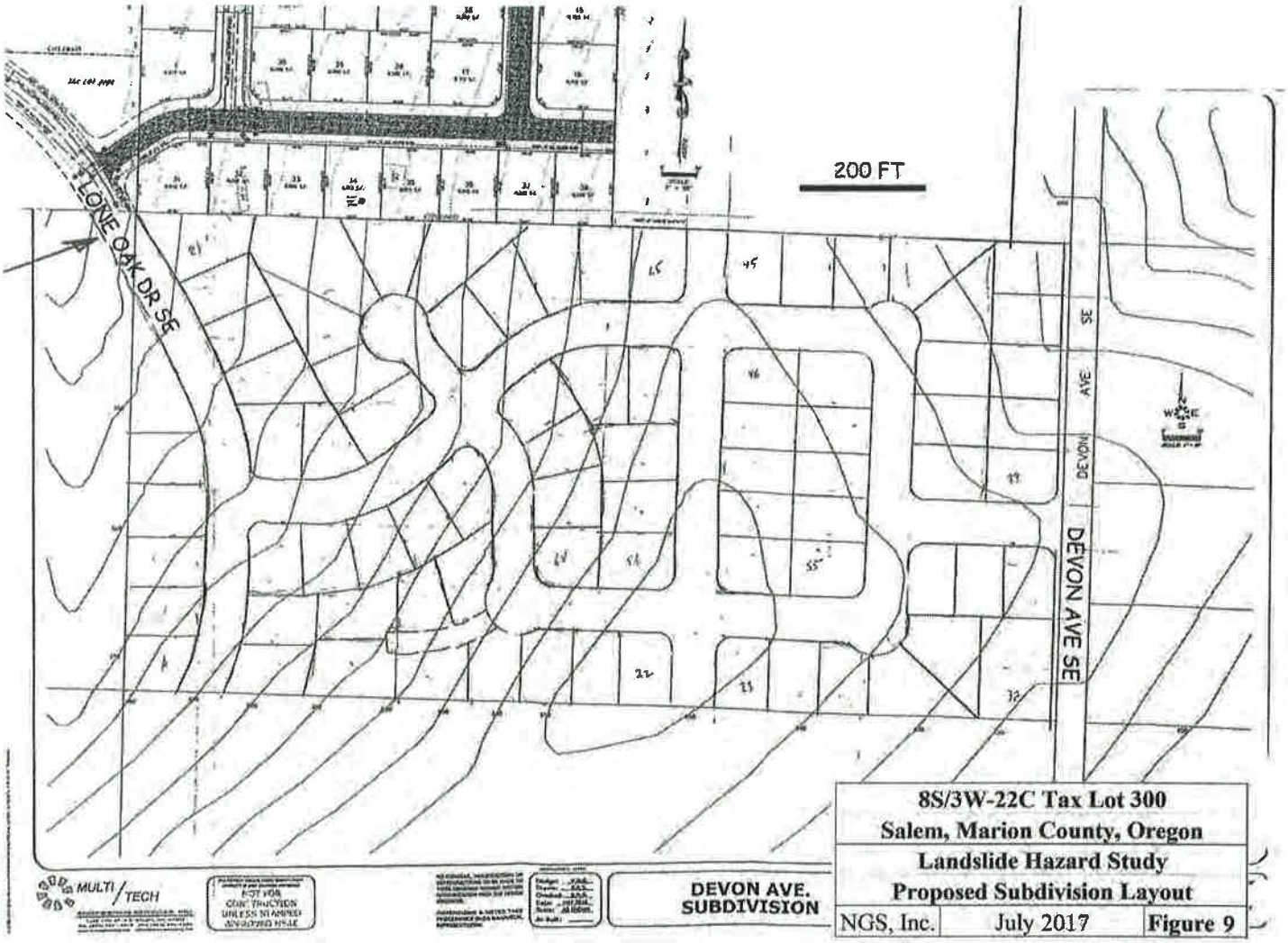
Normal Fault, ball on down side, dashed where approximate

8S/3W-22C Tax Lot 300		
Salem, Marion County, Oregon		
Landslide Hazard Study		
Area Geologic Map		
NGS, Inc.	July 2017	**Figure 5**


EXPLANATION
Hazard ratings
High
Moderate
Low
Very low
Relative Earthquake Hazards
Seismically-Induced Landslides
from DOGAMI IMS-17
SITE
Devon Av SE
Rees Hill Rd SE
Rosedale
N
1000 ft
Note: No water-induced landslide info published for this area as of July 2017
8S/3W-22C Tax Lot 300
Salem, Marion County, Oregon
Landslide Hazard Study
Earthquake Landslide Hazard Map
NGS, Inc.
July 2017
Figure 6


T 8 S, R 3 W, SEC 22
TAXLOT 300
150 ft
N
TP-1
TP-4
TP-3
B-1
SITE
Map Key
2 ft contour
Building footprint
Landslide Hazards
POINTS
2
3
4
5
6
TP-2 Test Pit excavated 11 July 2017 for Geotechnical Report
Map provided by City of Salem Public Works Department
8S/3W-22C Tax Lot 300
Salem, Marion County, Oregon
Landslide Hazard Study
City of Salem Landslide Hazard Map
NGS, Inc. July 2017 Figure 7
Landslide Hazards with 2 ft Contours

8S/3W-22C Tax Lot 300		
Salem, Marion County, Oregon		
Landslide Hazard Study		
1936 Aerial Photograph		
NGS, Inc.	July 2017	Figure 8



8S/3W-22C Tax Lot 300
Salem, Marion County, Oregon
Landslide Hazard Study
Proposed Subdivision Layout

NGS, Inc.	July 2017	**Figure 9**

Appendix C

Time of Concentration

Drawing is NOT to Scale

SCALE: 1" = 60'

Point F
Elev. 604

L = 510'

Point E
Elev. 642

Point D
Elev. 651

L = 230'

Point C
Elev. 539

L = 580'

Point B
Elev. 622

L = 250'

Point A
Elev. 651

SAHALEE DR.

LONE OAK ROAD

DEVON AVE.

TAX LOT #100

N89°22'41"W 1,298.71'

S0°06'31"W 662.05'

N0°00'00"E 661.91'

S89°23'06"E 1,299.96'

MULTI/TECH

ENGINEERING SERVICES, INC.
1155 13th ST. SE. SALEM, OR. 97302
(503) 363-9227 FAX: (503) 364-1260
www.mtengineering.net office@mtengineering.net

EXISTING CONDITIONS PLAN

DEVON ESTATES

Design: M.D.G.
Drawn: P.H.S.
Checked: B.M.G.
Date: NOV. 2017
Scale: AS SHOWN

REGISTERED PROFESSIONAL ENGINEER
OREGON
JUNE 14, 2013
MARK D. GRENZ

EXPIRES: 06-30-2019

JOB # 6502

P102

J:\6500-6599\6502-DevonAvenueSubdivision\Dwg\6502p.dwg, P102XC, 12/10/2018 3:44:28 PM, PStorders

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project	By	Date
Devon Estates (Basin #1)	M. Hendrick	10/2018
Location	**Checked**	**Date**
Salem, Oregon		

Check one: ☑ Present ☐ Developed

Check one: ☑ T_c ☑ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to Tc only)

	Segment ID	A-B	
1.	Surface description (Table 4D-4)	Meadow/Pasture/Farm	
2.	Manning's roughness coefficient, n (Table 4D-4)	0.15	
3.	Flow length, L (total L † 300 ft) ft	250	
4.	Two-year 24-hour rainfall, P_2 in	2.2	
5.	Land slope, s ft/ft	0.116	
6.	$T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t hr	0.20 +	= 0.20

Shallow concentrated flow

	Segment ID	B-C	
7.	Surface description (paved or unpaved)	Forest & Meadow	
8.	Flow length, L ft	580	
9.	Watercourse slope, s ft/ft	0.143	
10.	Average velocity, V (figure 3-1) ft/s	0.95	
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t hr	0.17 +	= 0.17

Channel flow

	Segment ID		
12.	Cross sectional flow area, a ft^2		
13.	Wetted perimeter, p_w ft		
14.	Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ft		
15	Channel slope, s ft/ft		
16.	Manning's roughness coefficient, n		
17.	$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ft/s		
18.	~~Flow~~ length, L ft		
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t hr	+	=
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) Hr		0.37

0.37 Hrs = 22.2 Minutes

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project	By	Date
Devon Estates (Basin #2)	M. Hendrick	1/2021
Location	**Checked**	**Date**
Salem, Oregon		

Check one: ☑ Present ☐ Developed

Check one: ☑ T_c ☑ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to Tc only)

	Segment ID	D-E		
1.	Surface description (Table 4D-4)	Meadow/Pasture/Farm		
2.	Manning's roughness coefficient, n (Table 4D-4)	0.15		
3.	Flow length, L (total L † 300 ft) ft	230		
4.	Two-year 24-hour rainfall, P_2 in	2.2		
5.	Land slope, s ft/ft	0.039		
6.	$T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t hr	0.29	+	= 0.29

Shallow concentrated flow

	Segment ID	E-F		
7.	Surface description (paved or unpaved)	Pasture		
8.	Flow length, L ft	510		
9.	Watercourse slope, s ft/ft	0.075		
10.	Average velocity, V (figure 3-1) ft/s	2.0		
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t hr	0.07	+	= 0.07

Channel flow

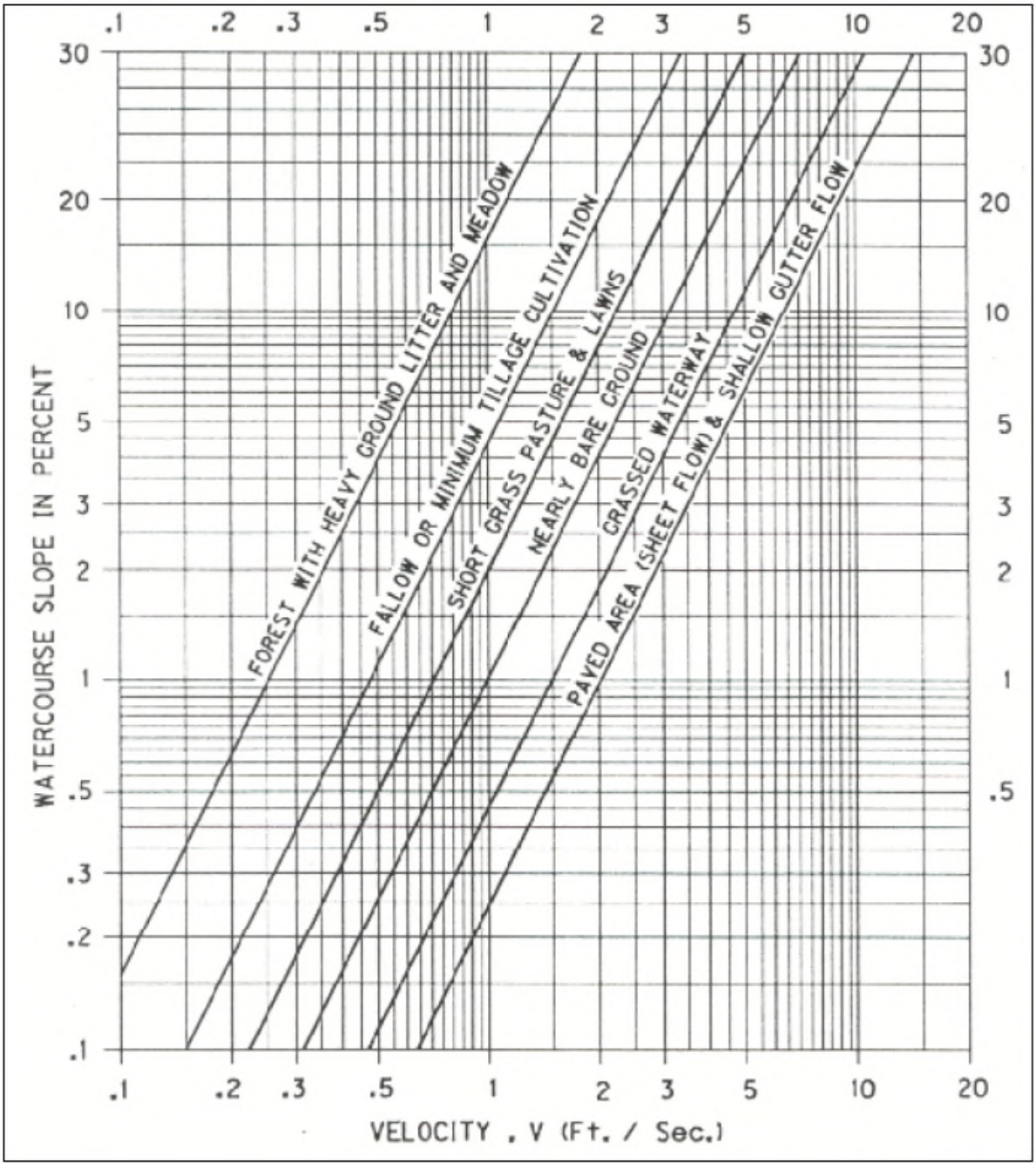
	Segment ID			
12.	Cross sectional flow area, a ft^2			
13.	Wetted perimeter, p_w ft			
14.	Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ft			
15	Channel slope, s ft/ft			
16.	Manning's roughness coefficient, n			
17.	$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ft/s			
18.	~~Flow~~ length, L ft			
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t hr		+	=

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) Hr **0.36**

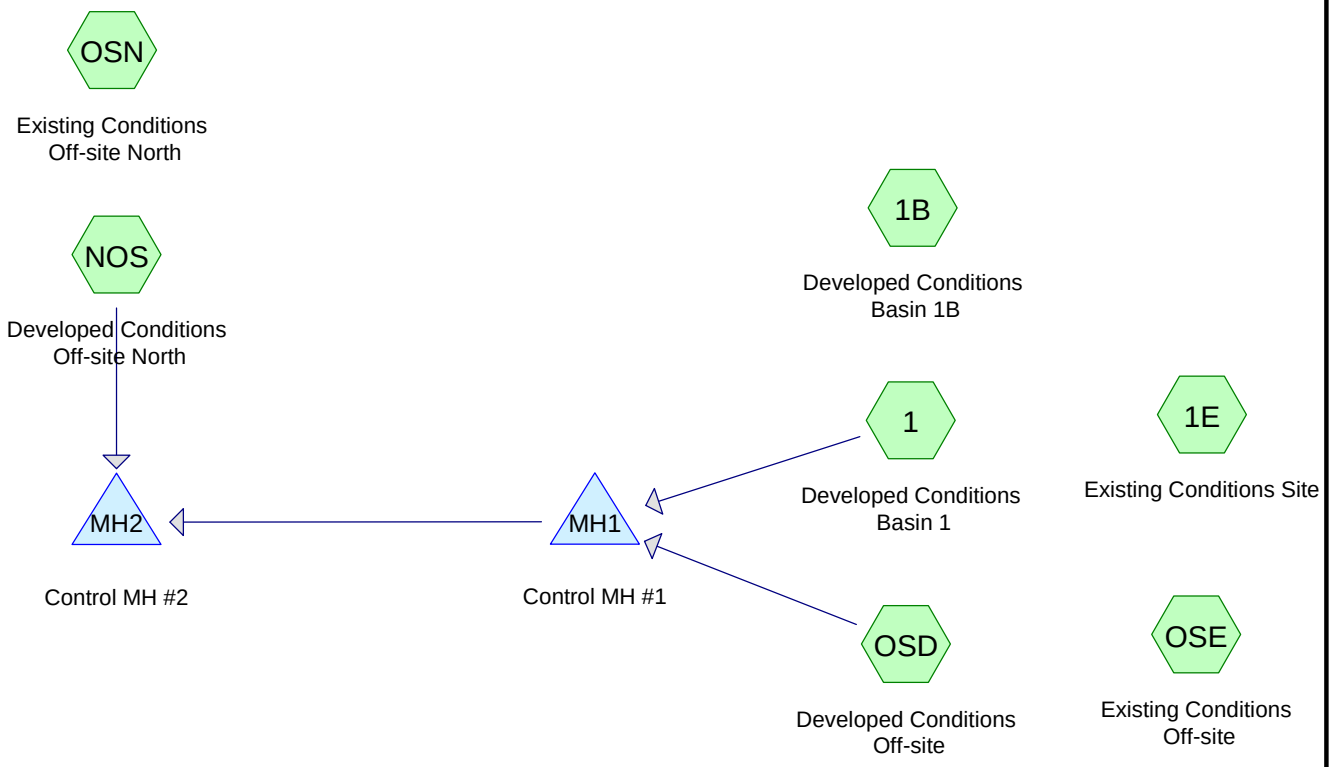
0.36 Hrs = 21.6 Minutes

Manning's Roughness Coefficients for Overland Sheet Flow	
Surface Types:	**n**
Impervious Areas	0.014
Gravel Pavement	0.02
Developed: Landscape Areas (Except Lawns)	0.08
Undeveloped: Meadow, Pasture, or Farm	0.15
Developed: Lawns	0.24
Pre-developed: Mixed	0.30
Pre-developed: Woodland and Forest	0.40
Development Types:	**n**
Commercial Development	0.015
Industrial Development, Heavy	0.04
Industrial Development, Light	0.05
Dense Residential (over 6 units/acre)	0.08
Normal Residential (3 to 6 units/acre)	0.20
Light Residential (1 to 3 units/acre)	0.30
Parks	0.40

Table 4D-4. Manning's Roughness Coefficients for Overland Sheet Flow


.1
.2
.3
.5
1
2
3
5
10
20
30
20
10
5
3
2
1
.5
.3
.2
.1
WATERCOURSE SLOPE IN PERCENT
FOREST WITH HEAVY GROUND LITTER AND MEADOW
FALLOW OR MINIMUM TILLAGE CULTIVATION
SHORT GRASS PASTURE & LAWNS
NEARLY BARE GROUND
GRASSED WATERWAY
PAVED AREA (SHEET FLOW) & SHALLOW GUTTER FLOW
VELOCITY , V (Ft. / Sec.)

Appendix D



Subcat Reach Pond Link

Routing Diagram for 20210112 Devon Estates Phase 1 Master
Prepared by Multitech Engineering Services, Inc., Printed 2/17/2021

Summary for Subcatchment 1E: Existing Conditions Site

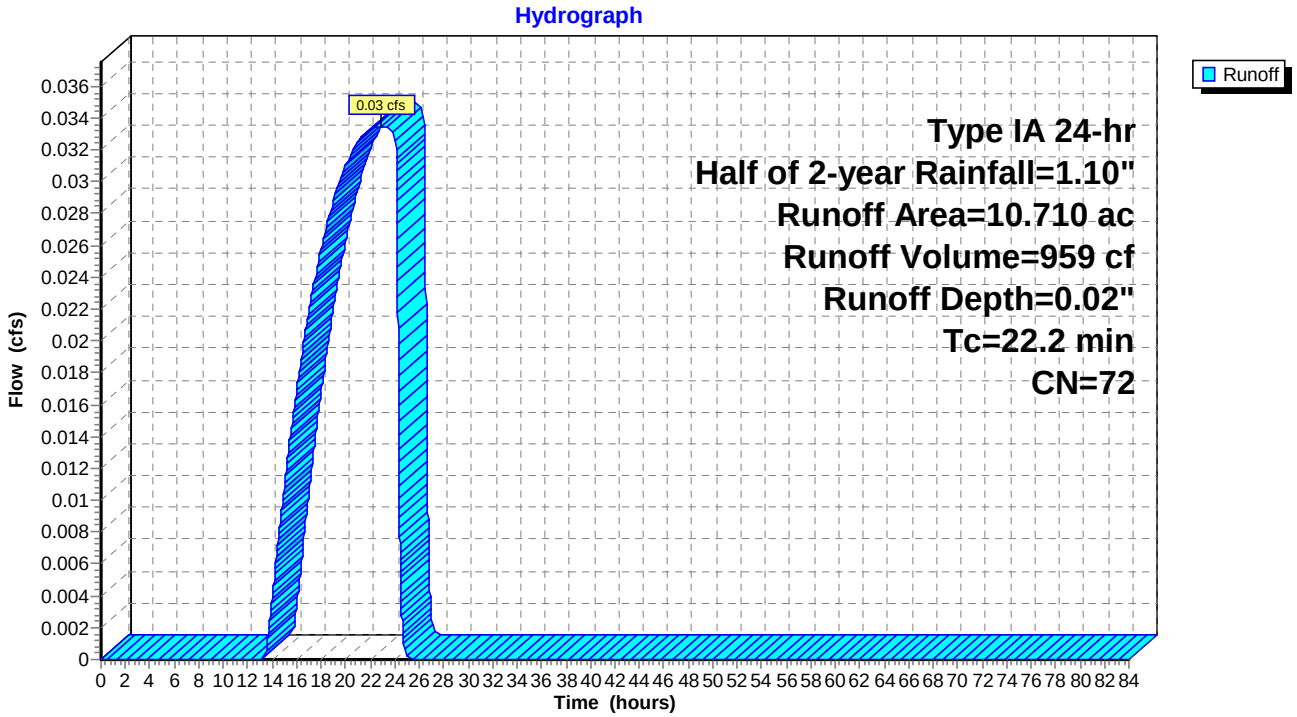
Runoff = 0.03 cfs @ 22.82 hrs, Volume= 959 cf, Depth= 0.02"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	10.710	72	City of Salem Pre-development, HSG C
	10.710	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment 1E: Existing Conditions Site



Summary for Subcatchment OSE: Existing Conditions Off-site

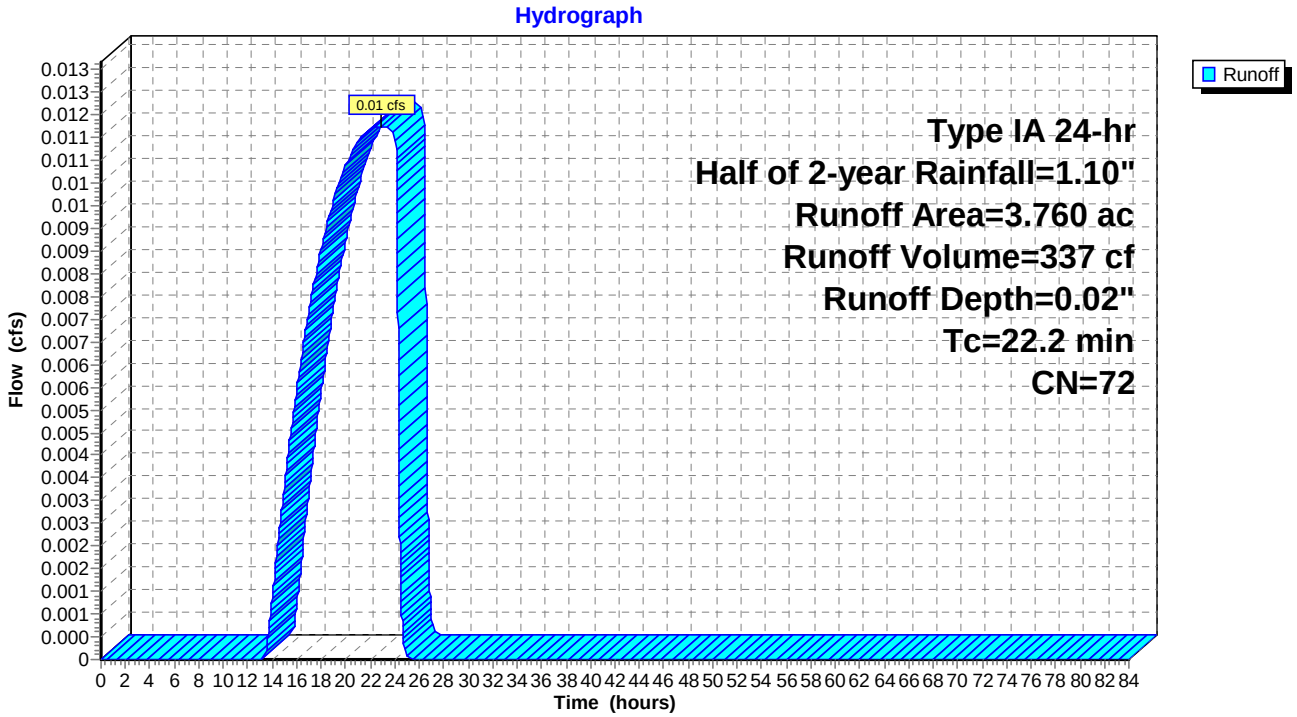
Runoff = 0.01 cfs @ 22.82 hrs, Volume= 337 cf, Depth= 0.02"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	3.760	72	City of Salem Pre-development, HSG C
	3.760	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment OSE: Existing Conditions Off-site



Summary for Subcatchment 1B: Developed Conditions Basin 1B

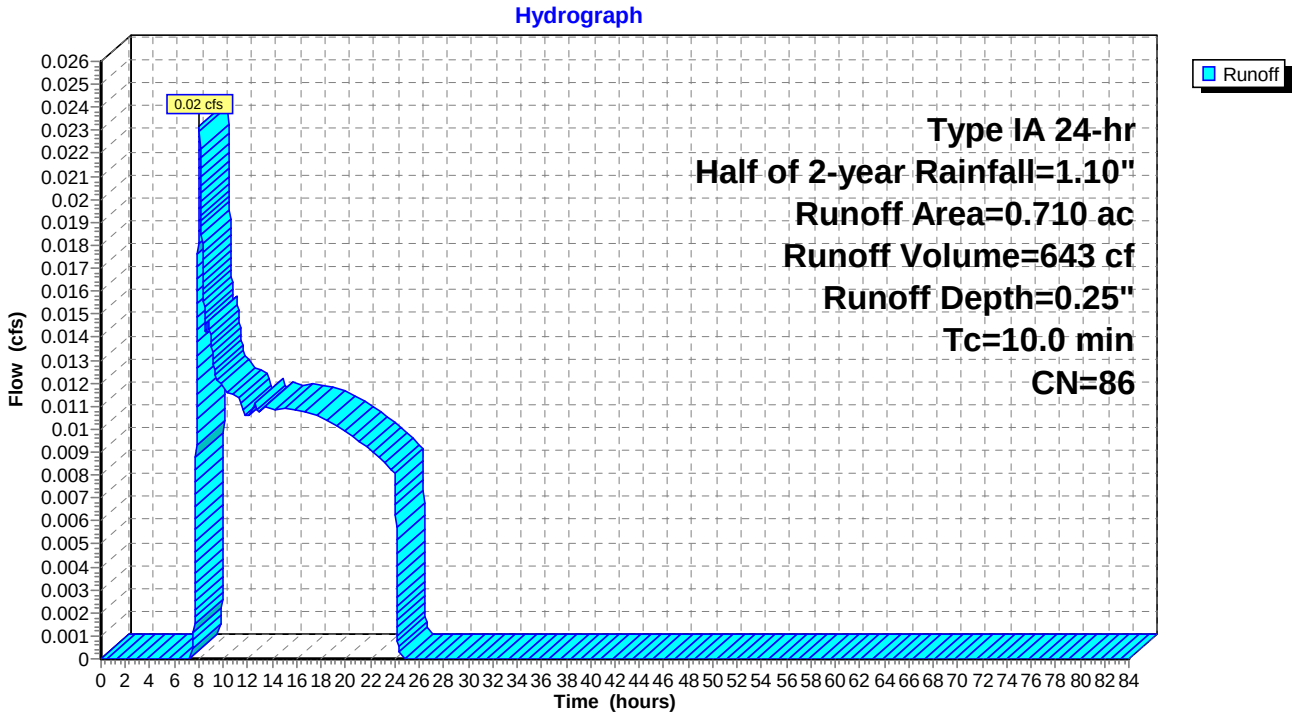
Runoff = 0.02 cfs @ 8.07 hrs, Volume= 643 cf, Depth= 0.25"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	0.350	98	Roofs,pavement, concrete, HSG C
	0.360	74	>75% Grass cover, Good, HSG C
	0.710	86	Weighted Average
	0.360	74	50.70% Pervious Area
	0.350	98	49.30% Impervious Area

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

Subcatchment 1B: Developed Conditions Basin 1B



Summary for Subcatchment 1: Developed Conditions Basin 1

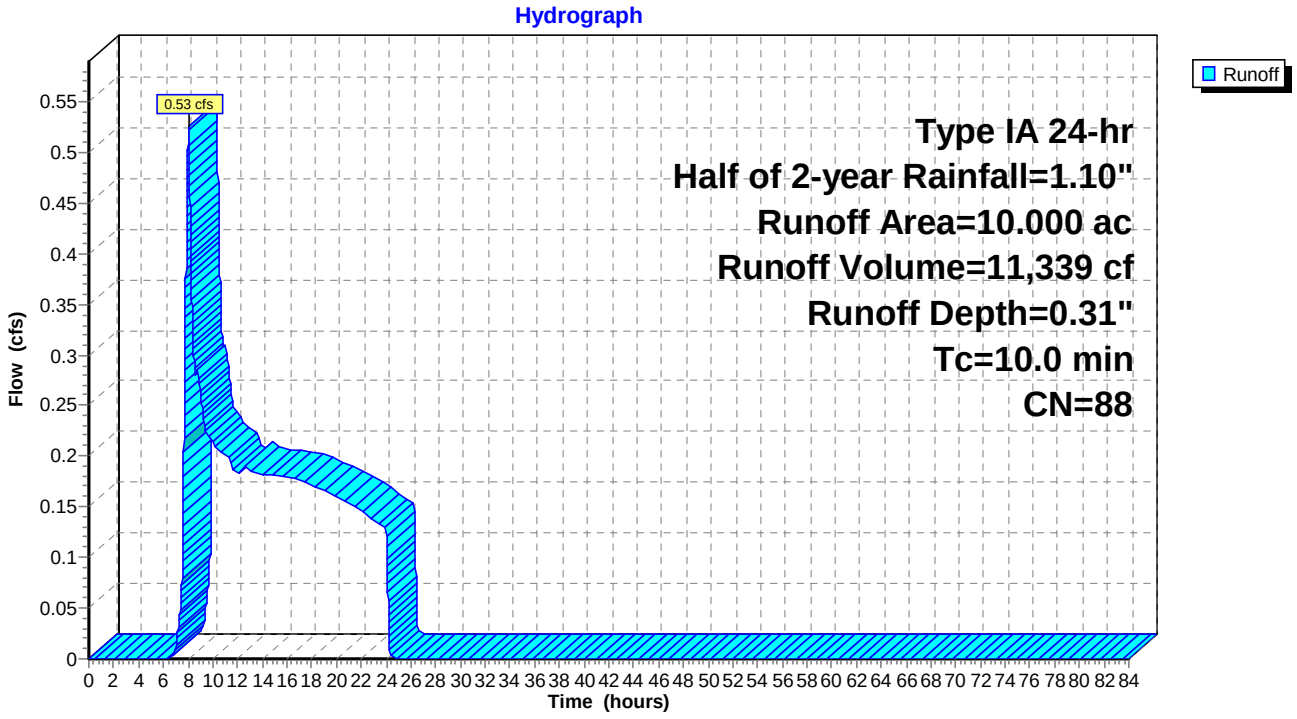
Runoff = 0.53 cfs @ 8.06 hrs, Volume= 11,339 cf, Depth= 0.31"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	6.000	98	Roofs,pavement, concrete, HSG C
	4.000	74	>75% Grass cover, Good, HSG C
	10.000	88	Weighted Average
	4.000	74	40.00% Pervious Area
	6.000	98	60.00% Impervious Area

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

Subcatchment 1: Developed Conditions Basin 1



Summary for Subcatchment OSD: Developed Conditions Off-site

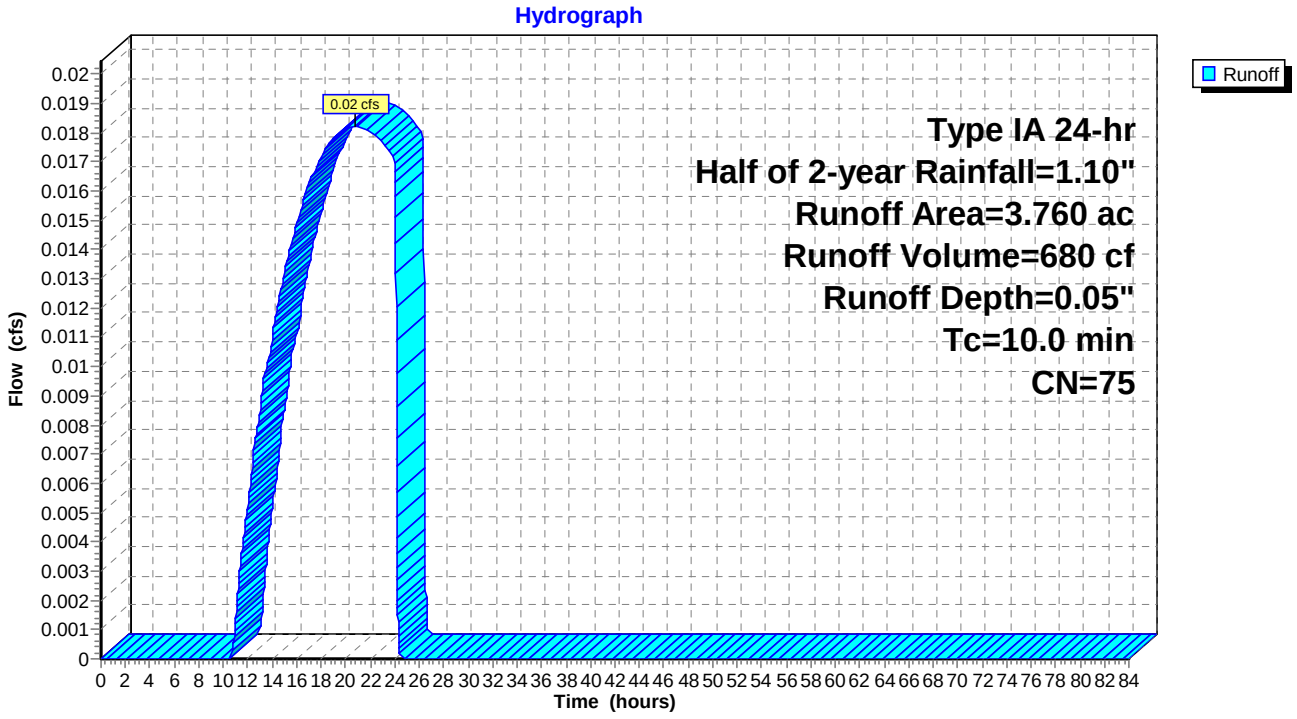
Runoff = 0.02 cfs @ 20.81 hrs, Volume= 680 cf, Depth= 0.05"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	3.260	72	City of Salem Pre-developed, HSG C
	0.500	98	Paved roads w/curbs & sewers, HSG C
	3.760	75	Weighted Average
	3.260	72	86.70% Pervious Area
	0.500	98	13.30% Impervious Area

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

Subcatchment OSD: Developed Conditions Off-site



Summary for Pond MH1: Control MH #1

Inflow Area = 599,386 sf, 47.24% Impervious, Inflow Depth = 0.24" for Half of 2-year event
Inflow = 0.53 cfs @ 8.06 hrs, Volume= 12,020 cf
Outflow = 0.02 cfs @ 24.25 hrs, Volume= 4,215 cf, Atten= 97%, Lag= 971.3 min
Primary = 0.02 cfs @ 24.25 hrs, Volume= 4,215 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 542.81' @ 24.25 hrs Surf.Area= 8,500 sf Storage= 11,141 cf
Flood Elev= 561.00' Surf.Area= 8,500 sf Storage= 55,229 cf

Plug-Flow detention time= 2,195.1 min calculated for 4,215 cf (35% of inflow)
Center-of-Mass det. time= 1,869.0 min (2,767.1 - 898.2)

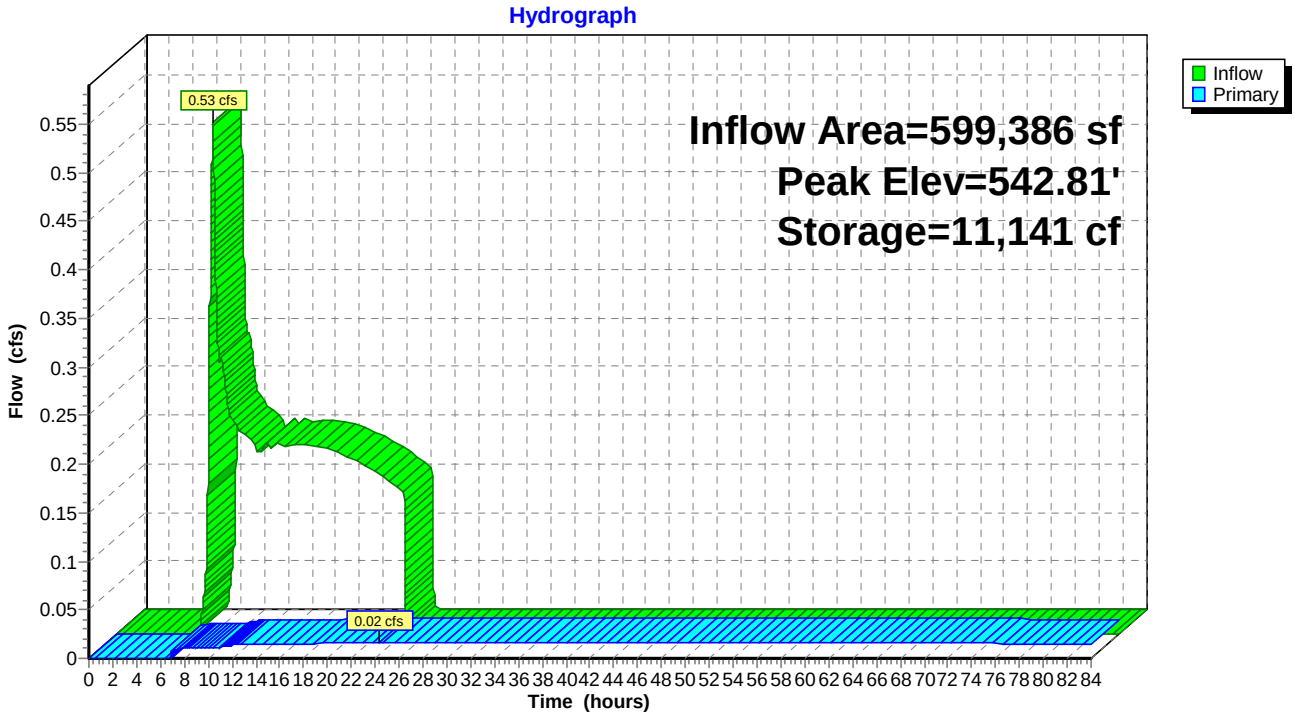
Volume	Invert	Avail.Storage	Storage Description
#1	537.00'	55,229 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
537.00	4	0.0	0	0
539.24	8,500	0.0	0	0
539.25	8,500	40.0	34	34
540.24	8,500	40.0	3,366	3,400
540.25	8,500	5.0	4	3,404
541.99	8,500	5.0	740	4,144
542.00	8,500	100.0	85	4,229
544.00	8,500	100.0	17,000	21,229
546.00	8,500	100.0	17,000	38,229
547.00	8,500	100.0	8,500	46,729
548.00	8,500	100.0	8,500	55,229

Device	Routing	Invert	Outlet Devices
#1	Primary	537.00'	**18.0" Round 18" Pipe** L= 62.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 537.00' / 535.00' S= 0.0323 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	537.00'	**0.5" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	543.00'	**7.5" Vert. Orifice #2** C= 0.600 Limited to weir flow at low heads
#4	Device 1	544.50'	**9.5" Vert. Orifice #3** C= 0.600 Limited to weir flow at low heads
#5	Device 1	547.00'	**24.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.02 cfs @ 24.25 hrs HW=542.81' (Free Discharge)
└**1=18" Pipe** (Passes 0.02 cfs of 23.84 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 0.02 cfs @ 11.59 fps)
  ├**3=Orifice #2** (Controls 0.00 cfs)
  ├**4=Orifice #3** (Controls 0.00 cfs)
  └**5=Overflow** (Controls 0.00 cfs)

Pond MH1: Control MH #1



Summary for Subcatchment OSN: Existing Conditions Off-site North

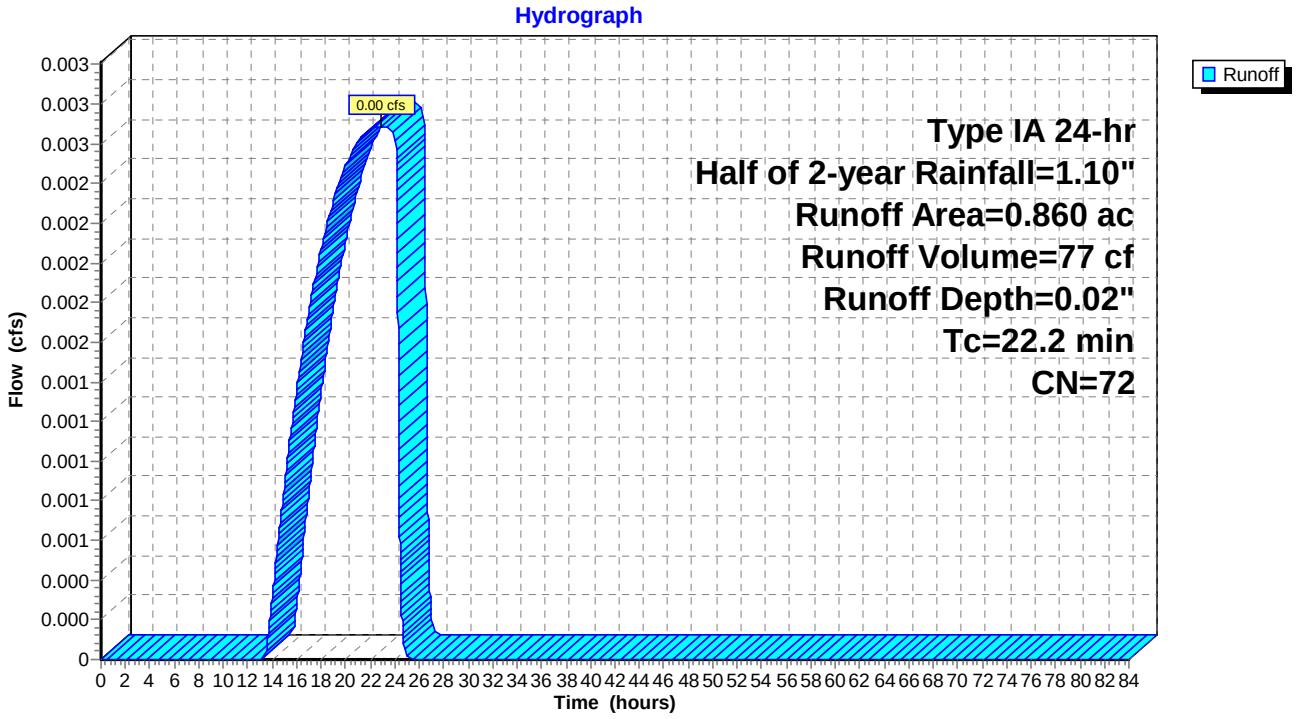
Runoff = 0.00 cfs @ 22.82 hrs, Volume= 77 cf, Depth= 0.02"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	0.860	72	City of Salem Pre-development, HSG C
	0.860	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment OSN: Existing Conditions Off-site North



Summary for Subcatchment NOS: Developed Conditions Off-site North

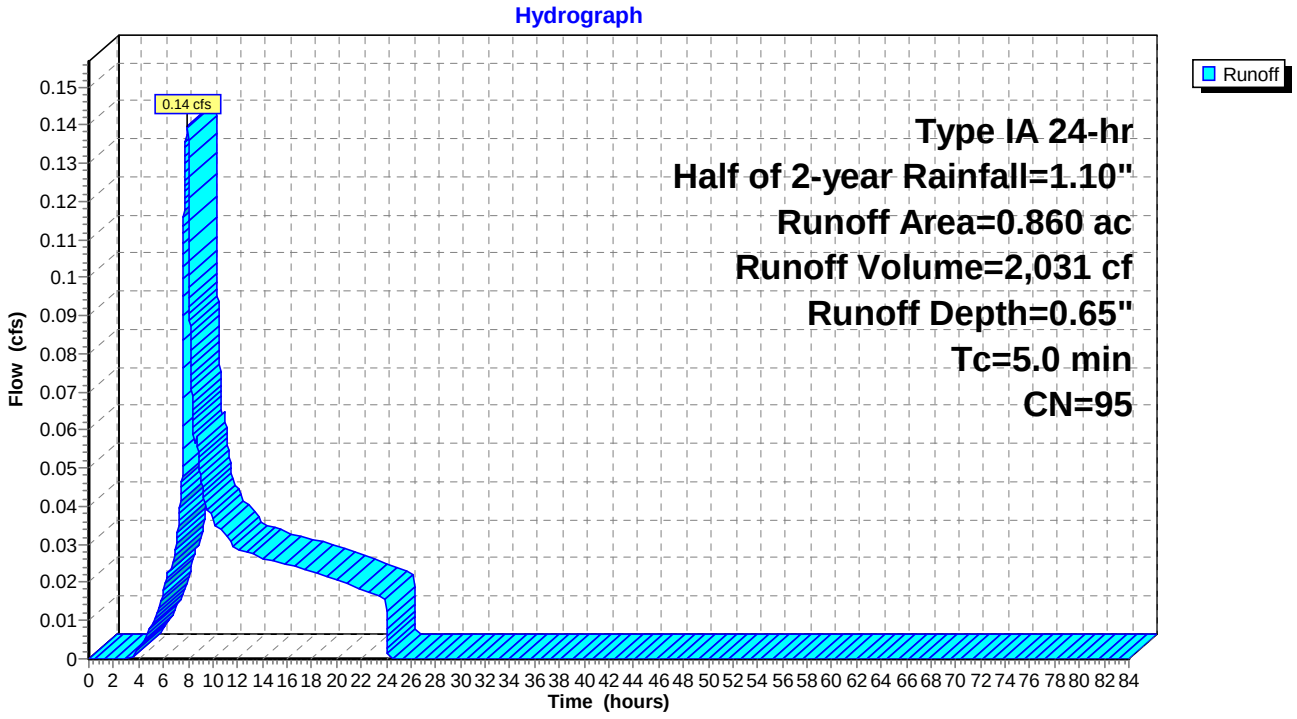
Runoff = 0.14 cfs @ 7.94 hrs, Volume= 2,031 cf, Depth= 0.65"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	0.760	98	Roofs,pavement, concrete, HSG C
	0.100	74	>75% Grass cover, Good, HSG C
	0.860	95	Weighted Average
	0.100	74	11.63% Pervious Area
	0.760	98	88.37% Impervious Area

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

Subcatchment NOS: Developed Conditions Off-site North



Summary for Pond MH2: Control MH #2

Inflow Area = 636,847 sf, 49.66% Impervious, Inflow Depth > 0.12" for Half of 2-year event
Inflow = 0.15 cfs @ 7.94 hrs, Volume= 6,246 cf
Outflow = 0.15 cfs @ 7.95 hrs, Volume= 6,246 cf, Atten= 0%, Lag= 0.6 min
Primary = 0.15 cfs @ 7.95 hrs, Volume= 6,246 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 502.16' @ 7.95 hrs Surf.Area= 51 sf Storage= 4 cf

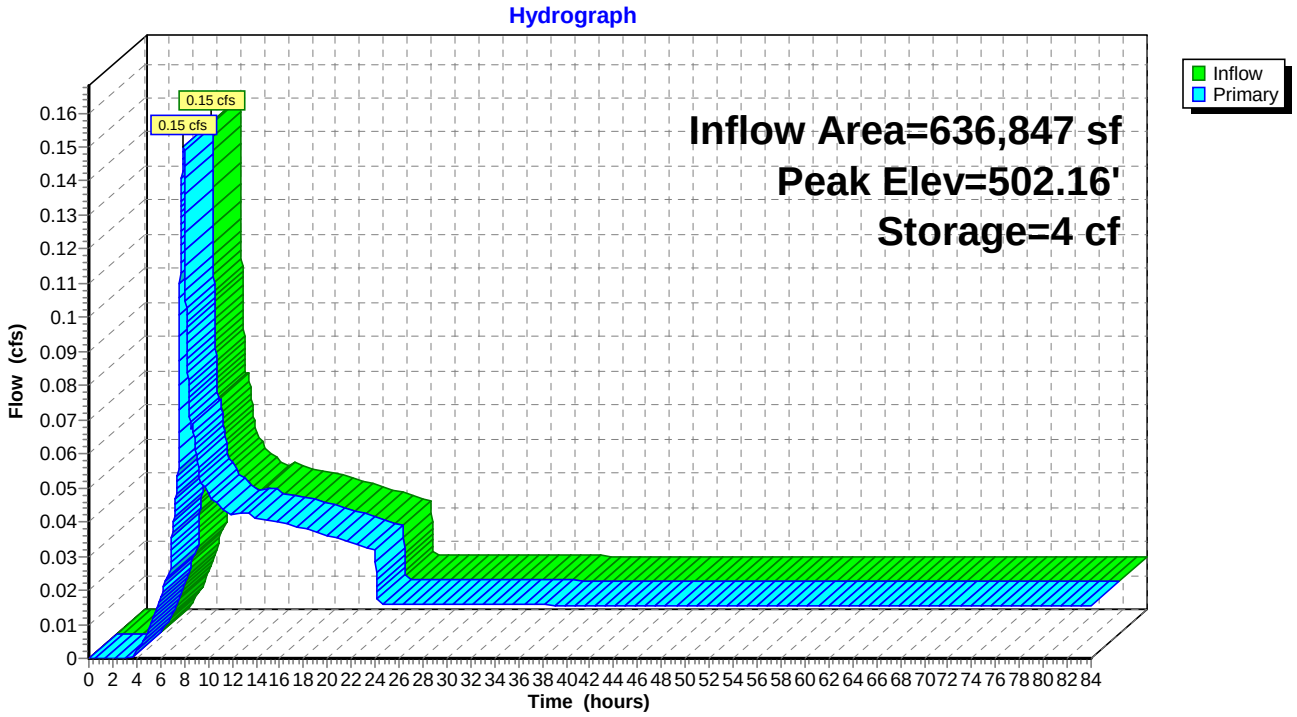
Plug-Flow detention time= 0.3 min calculated for 6,246 cf (100% of inflow)
Center-of-Mass det. time= 0.2 min (2,118.6 - 2,118.4)

Volume	Invert	Avail.Storage	Storage Description
#1	501.96'	2,356 cf	**60.0" Round 60" Detention Pipe** L= 120.0' S= 0.0050 '/'

Device	Routing	Invert	Outlet Devices
#1	Primary	499.77'	**18.0" Round Culvert** L= 110.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 499.77' / 484.72' S= 0.1368 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	501.96'	**10.0" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	507.50'	**18.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.15 cfs @ 7.95 hrs HW=502.16' (Free Discharge)
└**1=Culvert** (Passes 0.15 cfs of 14.84 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 0.15 cfs @ 1.51 fps)
  └**3=Overflow** (Controls 0.00 cfs)

Pond MH2: Control MH #2



Summary for Subcatchment 1E: Existing Conditions Site

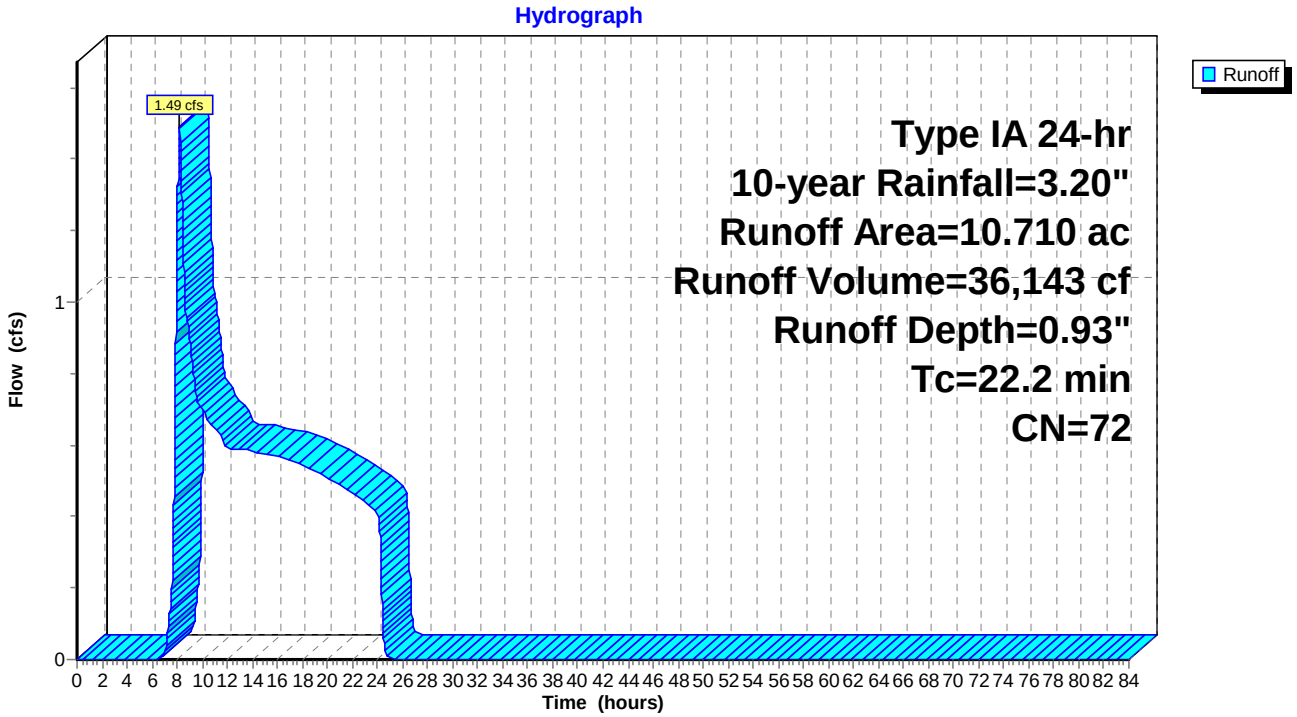
Runoff = 1.49 cfs @ 8.20 hrs, Volume= 36,143 cf, Depth= 0.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	10.710	72	City of Salem Pre-development, HSG C
	10.710	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment 1E: Existing Conditions Site



Summary for Subcatchment OSE: Existing Conditions Off-site

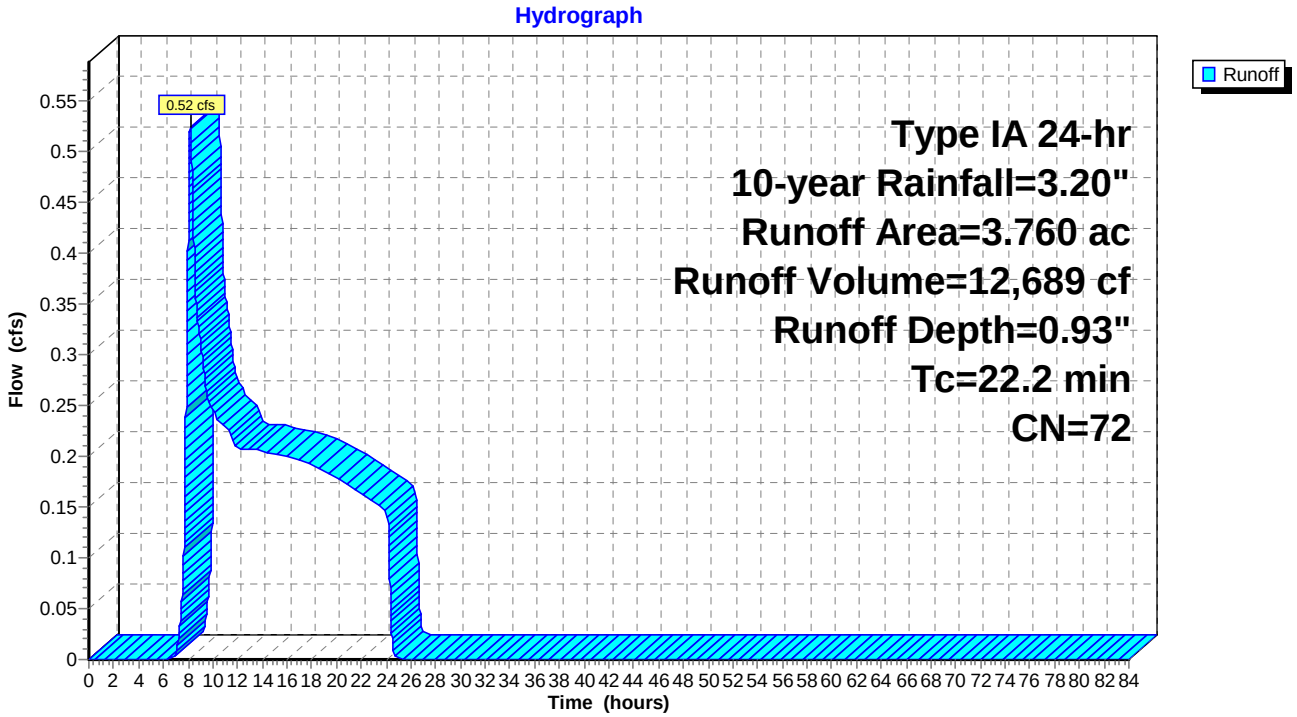
Runoff = 0.52 cfs @ 8.20 hrs, Volume= 12,689 cf, Depth= 0.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	3.760	72	City of Salem Pre-development, HSG C
	3.760	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment OSE: Existing Conditions Off-site



Summary for Subcatchment 1B: Developed Conditions Basin 1B

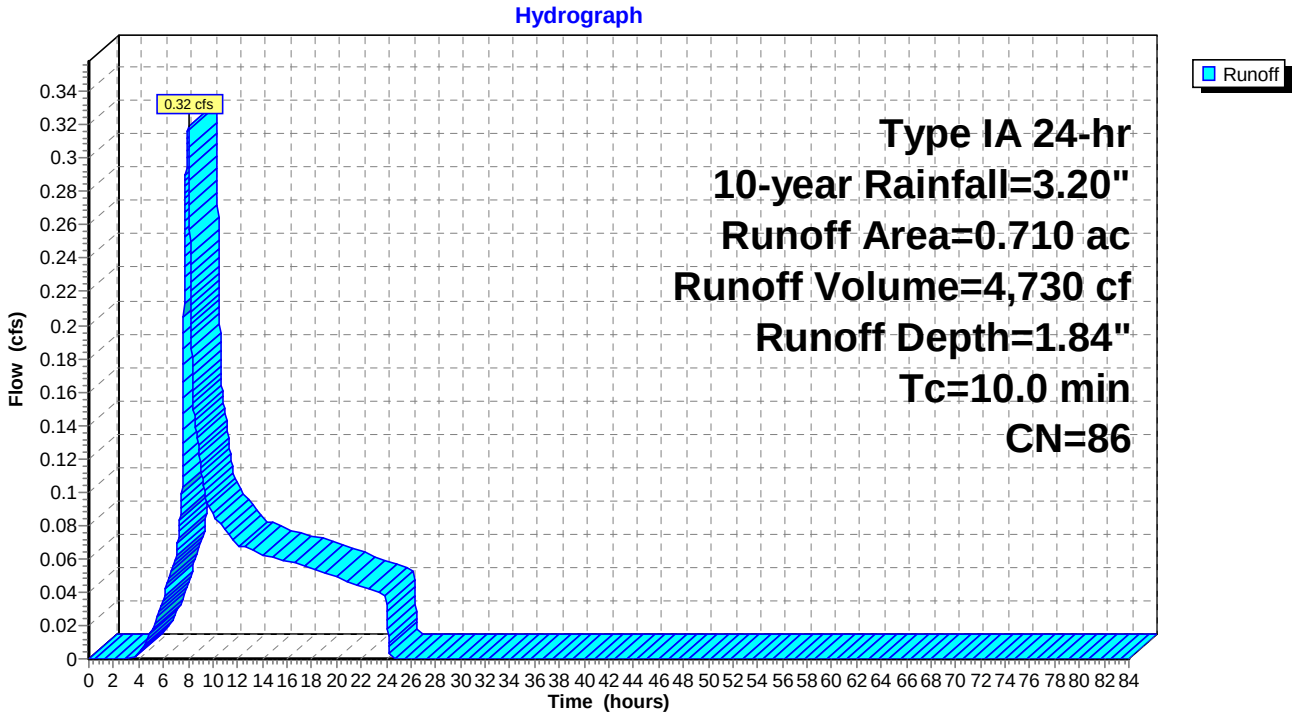
Runoff = 0.32 cfs @ 8.01 hrs, Volume= 4,730 cf, Depth= 1.84"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	0.350	98	Roofs,pavement, concrete, HSG C
	0.360	74	>75% Grass cover, Good, HSG C
	0.710	86	Weighted Average
	0.360	74	50.70% Pervious Area
	0.350	98	49.30% Impervious Area

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

Subcatchment 1B: Developed Conditions Basin 1B



Summary for Subcatchment 1: Developed Conditions Basin 1

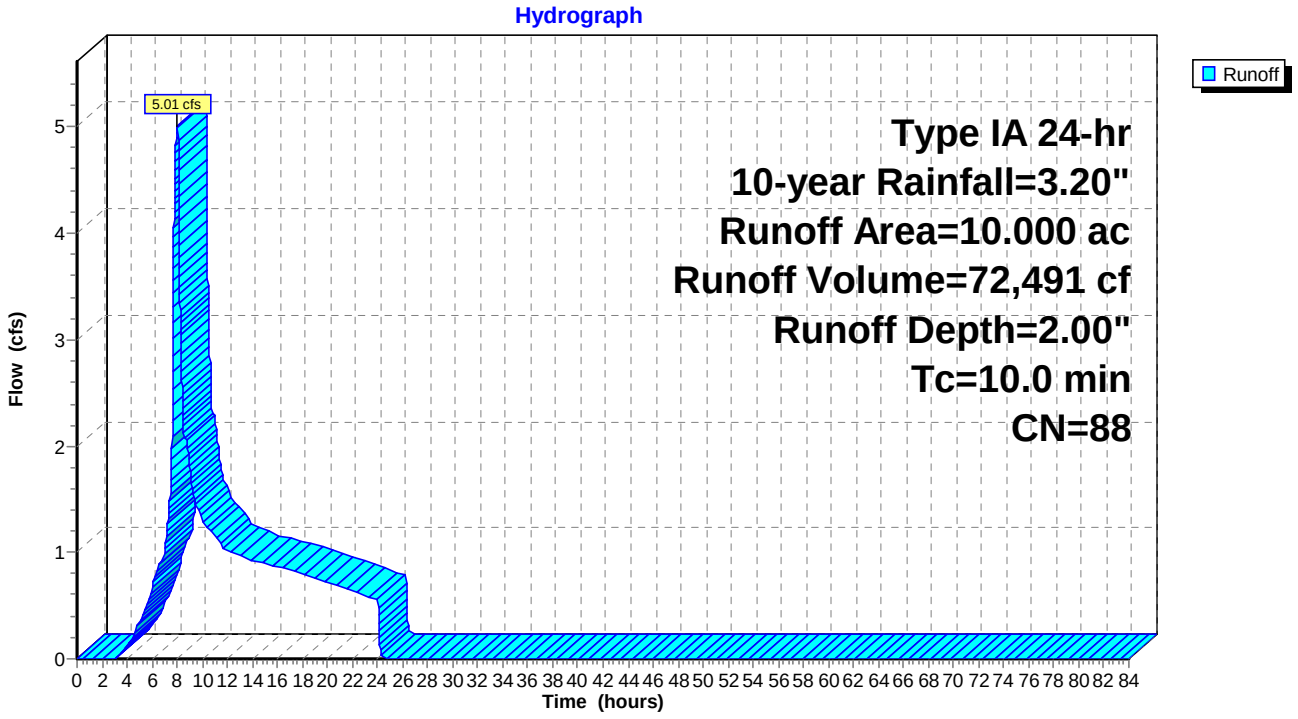
Runoff = 5.01 cfs @ 8.01 hrs, Volume= 72,491 cf, Depth= 2.00"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	6.000	98	Roofs,pavement, concrete, HSG C
	4.000	74	>75% Grass cover, Good, HSG C
	10.000	88	Weighted Average
	4.000	74	40.00% Pervious Area
	6.000	98	60.00% Impervious Area

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

Subcatchment 1: Developed Conditions Basin 1



Summary for Subcatchment OSD: Developed Conditions Off-site

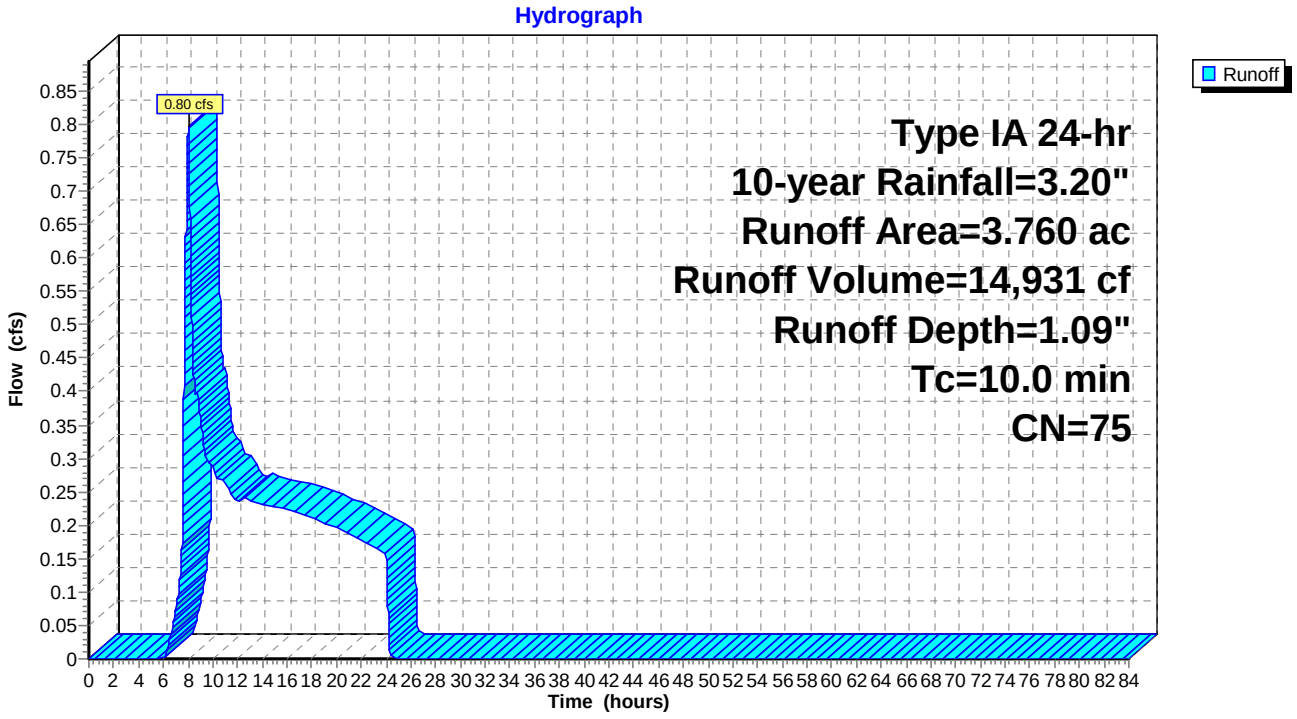
Runoff = 0.80 cfs @ 8.05 hrs, Volume= 14,931 cf, Depth= 1.09"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	3.260	72	City of Salem Pre-developed, HSG C
	0.500	98	Paved roads w/curbs & sewers, HSG C
	3.760	75	Weighted Average
	3.260	72	86.70% Pervious Area
	0.500	98	13.30% Impervious Area

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

Subcatchment OSD: Developed Conditions Off-site



Summary for Pond MH1: Control MH #1

Inflow Area = 599,386 sf, 47.24% Impervious, Inflow Depth = 1.75" for 10-year event
Inflow = 5.80 cfs @ 8.01 hrs, Volume= 87,422 cf
Outflow = 1.52 cfs @ 10.27 hrs, Volume= 77,544 cf, Atten= 74%, Lag= 135.5 min
Primary = 1.52 cfs @ 10.27 hrs, Volume= 77,544 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 544.35' @ 10.27 hrs Surf.Area= 8,500 sf Storage= 24,223 cf
Flood Elev= 561.00' Surf.Area= 8,500 sf Storage= 55,229 cf

Plug-Flow detention time= 345.4 min calculated for 77,535 cf (89% of inflow)
Center-of-Mass det. time= 274.7 min (1,057.5 - 782.8)

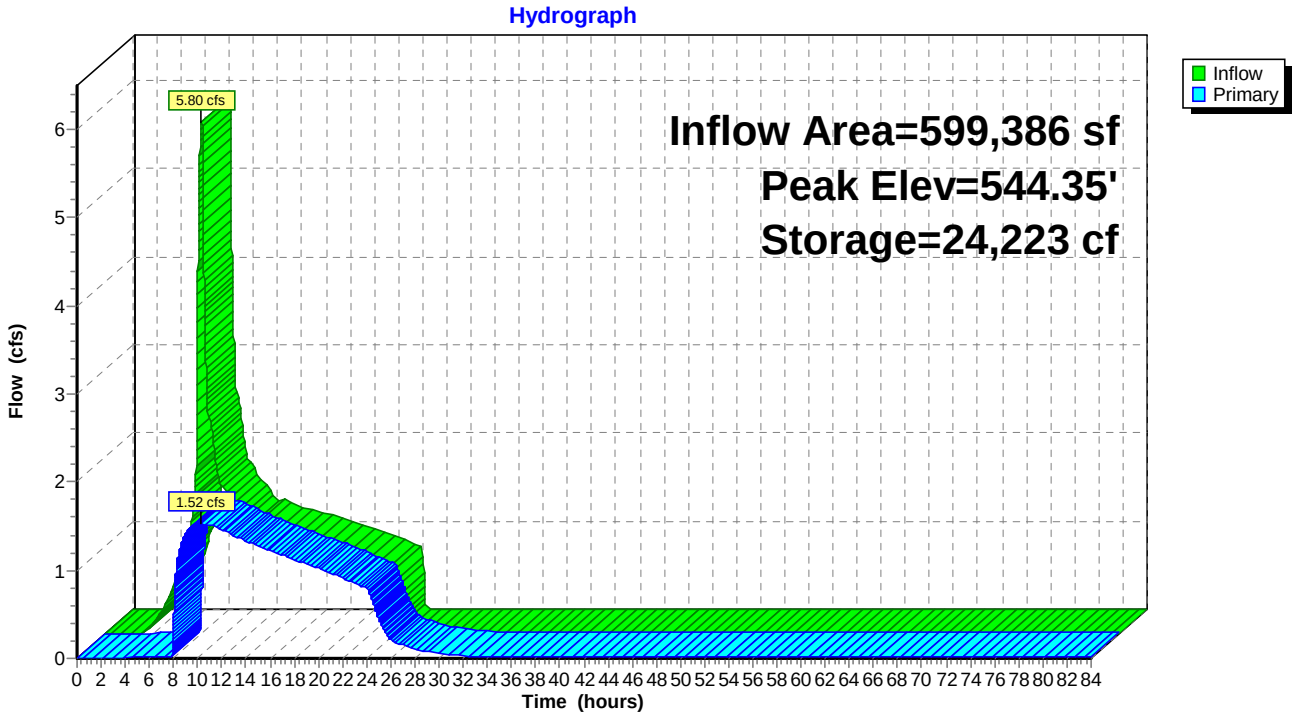
Volume	Invert	Avail.Storage	Storage Description
#1	537.00'	55,229 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
537.00	4	0.0	0	0
539.24	8,500	0.0	0	0
539.25	8,500	40.0	34	34
540.24	8,500	40.0	3,366	3,400
540.25	8,500	5.0	4	3,404
541.99	8,500	5.0	740	4,144
542.00	8,500	100.0	85	4,229
544.00	8,500	100.0	17,000	21,229
546.00	8,500	100.0	17,000	38,229
547.00	8,500	100.0	8,500	46,729
548.00	8,500	100.0	8,500	55,229

Device	Routing	Invert	Outlet Devices
#1	Primary	537.00'	**18.0" Round 18" Pipe** L= 62.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 537.00' / 535.00' S= 0.0323 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	537.00'	**0.5" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	543.00'	**7.5" Vert. Orifice #2** C= 0.600 Limited to weir flow at low heads
#4	Device 1	544.50'	**9.5" Vert. Orifice #3** C= 0.600 Limited to weir flow at low heads
#5	Device 1	547.00'	**24.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=1.52 cfs @ 10.27 hrs HW=544.35' (Free Discharge)
└**1=18" Pipe** (Passes 1.52 cfs of 26.59 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 0.02 cfs @ 13.04 fps)
  ├**3=Orifice #2** (Orifice Controls 1.51 cfs @ 4.91 fps)
  ├**4=Orifice #3** (Controls 0.00 cfs)
  └**5=Overflow** (Controls 0.00 cfs)

Pond MH1: Control MH #1



Summary for Subcatchment OSN: Existing Conditions Off-site North

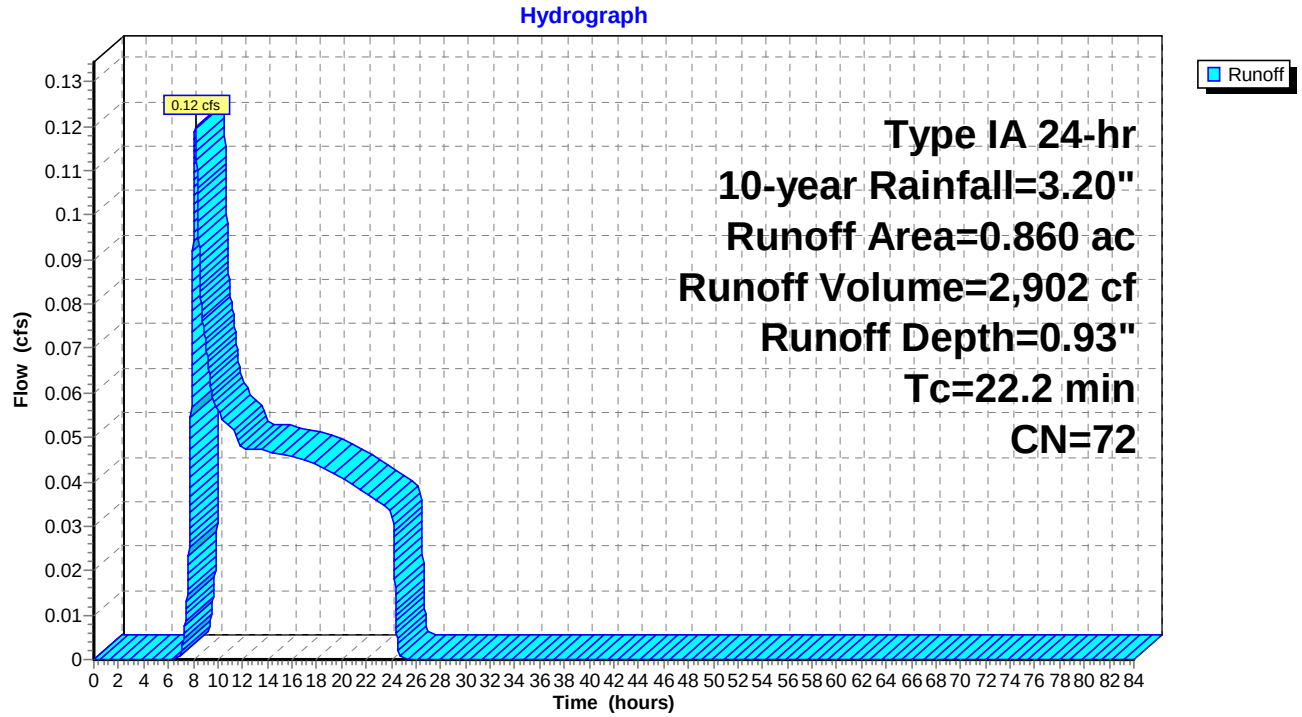
Runoff = 0.12 cfs @ 8.20 hrs, Volume= 2,902 cf, Depth= 0.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	0.860	72	City of Salem Pre-development, HSG C
	0.860	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment OSN: Existing Conditions Off-site North



Summary for Subcatchment NOS: Developed Conditions Off-site North

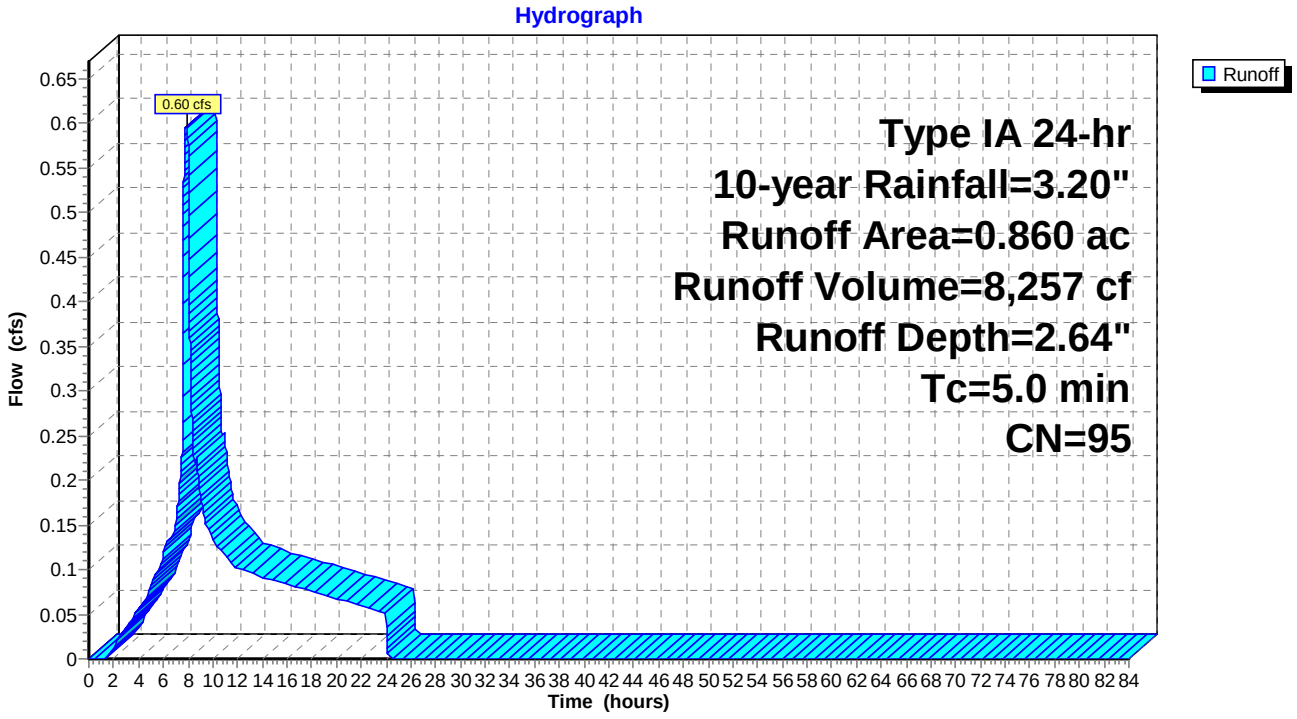
Runoff = 0.60 cfs @ 7.88 hrs, Volume= 8,257 cf, Depth= 2.64"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	0.760	98	Roofs,pavement, concrete, HSG C
	0.100	74	>75% Grass cover, Good, HSG C
	0.860	95	Weighted Average
	0.100	74	11.63% Pervious Area
	0.760	98	88.37% Impervious Area

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

Subcatchment NOS: Developed Conditions Off-site North



Summary for Pond MH2: Control MH #2

Inflow Area = 636,847 sf, 49.66% Impervious, Inflow Depth > 1.62" for 10-year event
Inflow = 1.65 cfs @ 10.01 hrs, Volume= 85,801 cf
Outflow = 1.65 cfs @ 10.04 hrs, Volume= 85,801 cf, Atten= 0%, Lag= 1.8 min
Primary = 1.65 cfs @ 10.04 hrs, Volume= 85,801 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 502.77' @ 10.04 hrs Surf.Area= 356 sf Storage= 131 cf

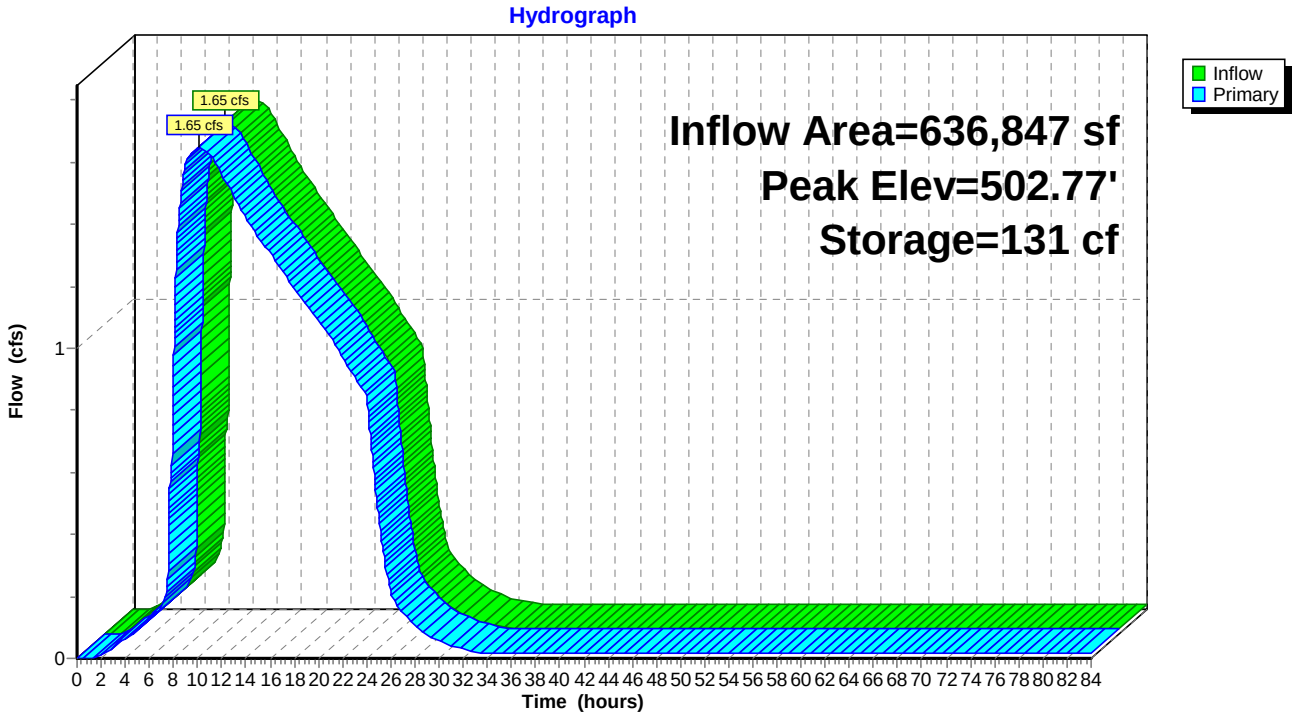
Plug-Flow detention time= 1.0 min calculated for 85,801 cf (100% of inflow)
Center-of-Mass det. time= 1.0 min (1,024.2 - 1,023.2)

Volume	Invert	Avail.Storage	Storage Description
#1	501.96'	2,356 cf	**60.0" Round 60" Detention Pipe** L= 120.0' S= 0.0050 '/'

Device	Routing	Invert	Outlet Devices
#1	Primary	499.77'	**18.0" Round Culvert** L= 110.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 499.77' / 484.72' S= 0.1368 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	501.96'	**10.0" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	507.50'	**18.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=1.66 cfs @ 10.04 hrs HW=502.77' (Free Discharge)
└**1=Culvert** (Passes 1.66 cfs of 17.41 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 1.66 cfs @ 3.06 fps)
  └**3=Overflow** (Controls 0.00 cfs)

Pond MH2: Control MH #2



Summary for Subcatchment 1E: Existing Conditions Site

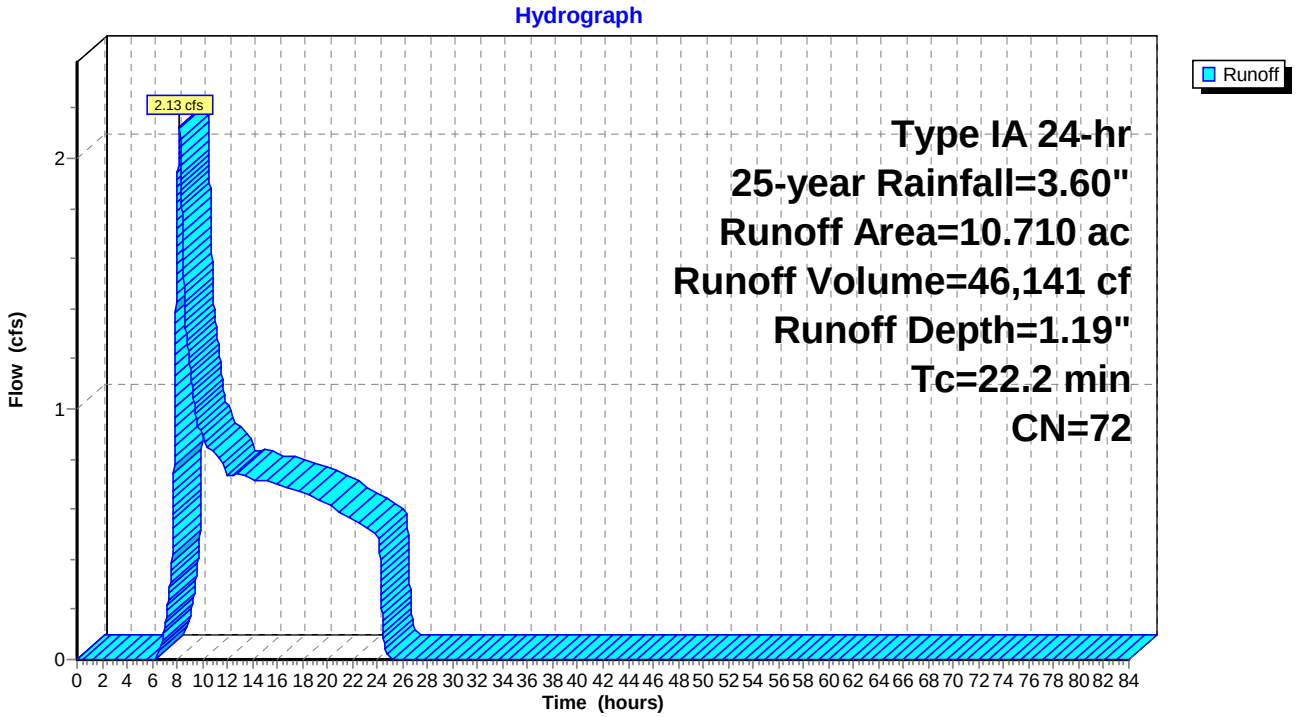
Runoff = 2.13 cfs @ 8.17 hrs, Volume= 46,141 cf, Depth= 1.19"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	10.710	72	City of Salem Pre-development, HSG C
	10.710	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment 1E: Existing Conditions Site



Summary for Subcatchment OSE: Existing Conditions Off-site

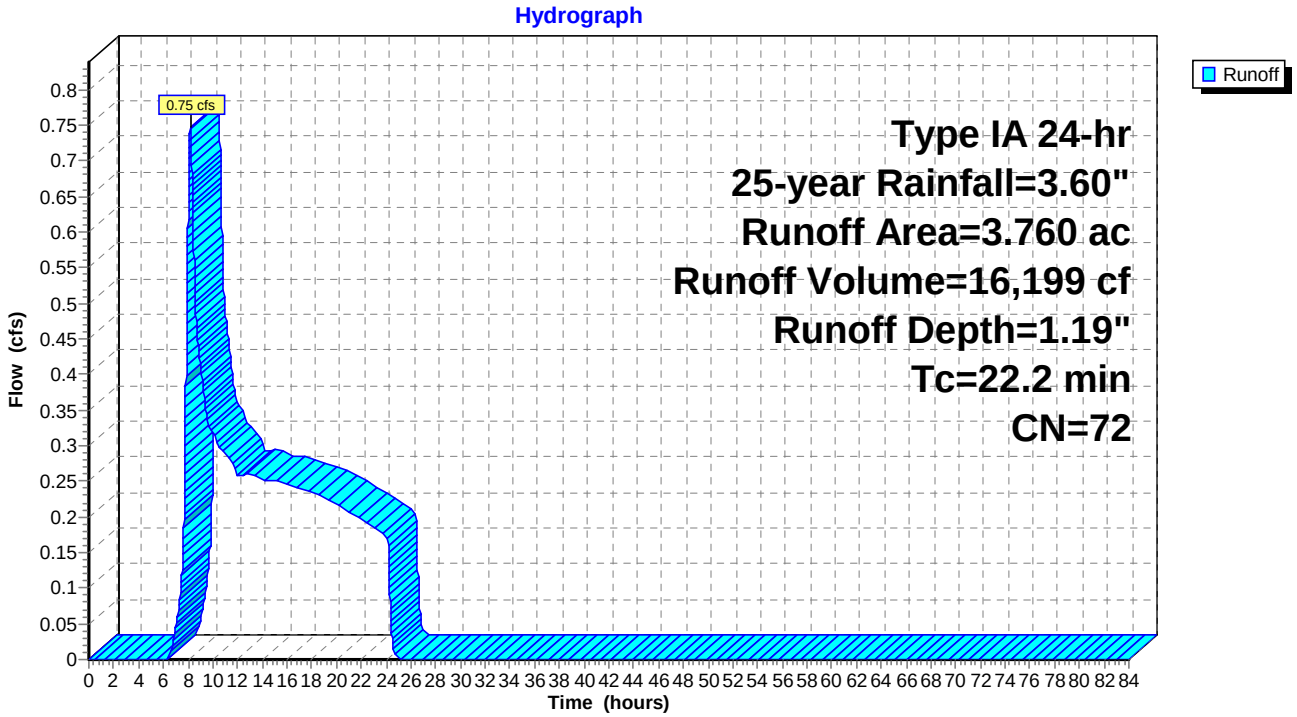
Runoff = 0.75 cfs @ 8.17 hrs, Volume= 16,199 cf, Depth= 1.19"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	3.760	72	City of Salem Pre-development, HSG C
	3.760	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment OSE: Existing Conditions Off-site



Summary for Subcatchment 1B: Developed Conditions Basin 1B

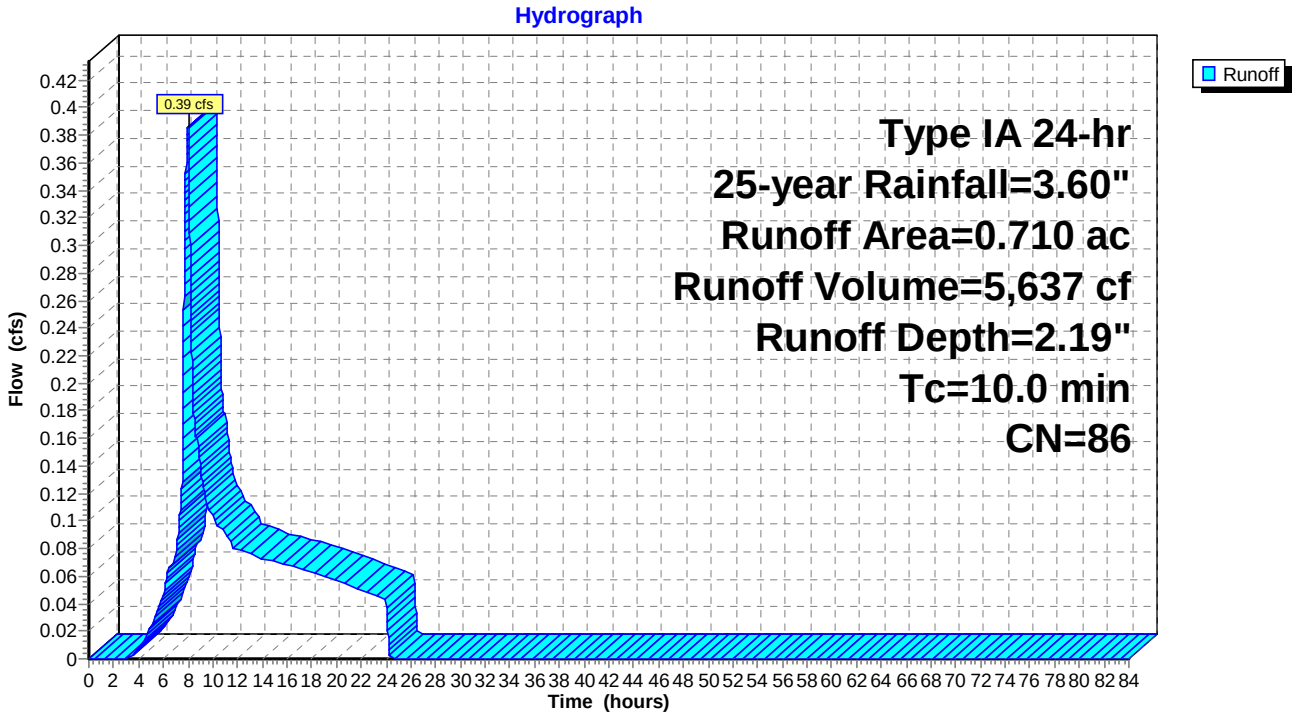
Runoff = 0.39 cfs @ 8.01 hrs, Volume= 5,637 cf, Depth= 2.19"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	0.350	98	Roofs,pavement, concrete, HSG C
	0.360	74	>75% Grass cover, Good, HSG C
	0.710	86	Weighted Average
	0.360	74	50.70% Pervious Area
	0.350	98	49.30% Impervious Area

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

Subcatchment 1B: Developed Conditions Basin 1B



Summary for Subcatchment 1: Developed Conditions Basin 1

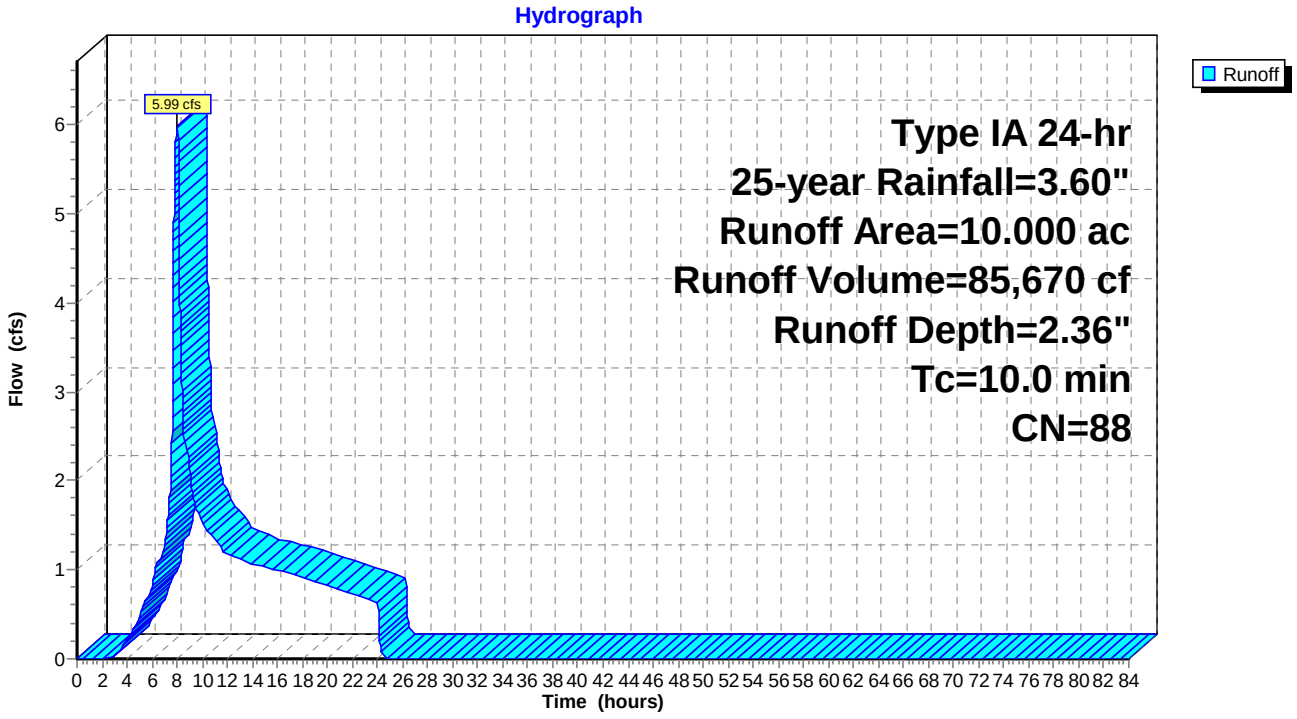
Runoff = 5.99 cfs @ 7.99 hrs, Volume= 85,670 cf, Depth= 2.36"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	6.000	98	Roofs,pavement, concrete, HSG C
	4.000	74	>75% Grass cover, Good, HSG C
	10.000	88	Weighted Average
	4.000	74	40.00% Pervious Area
	6.000	98	60.00% Impervious Area

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

Subcatchment 1: Developed Conditions Basin 1



Summary for Subcatchment OSD: Developed Conditions Off-site

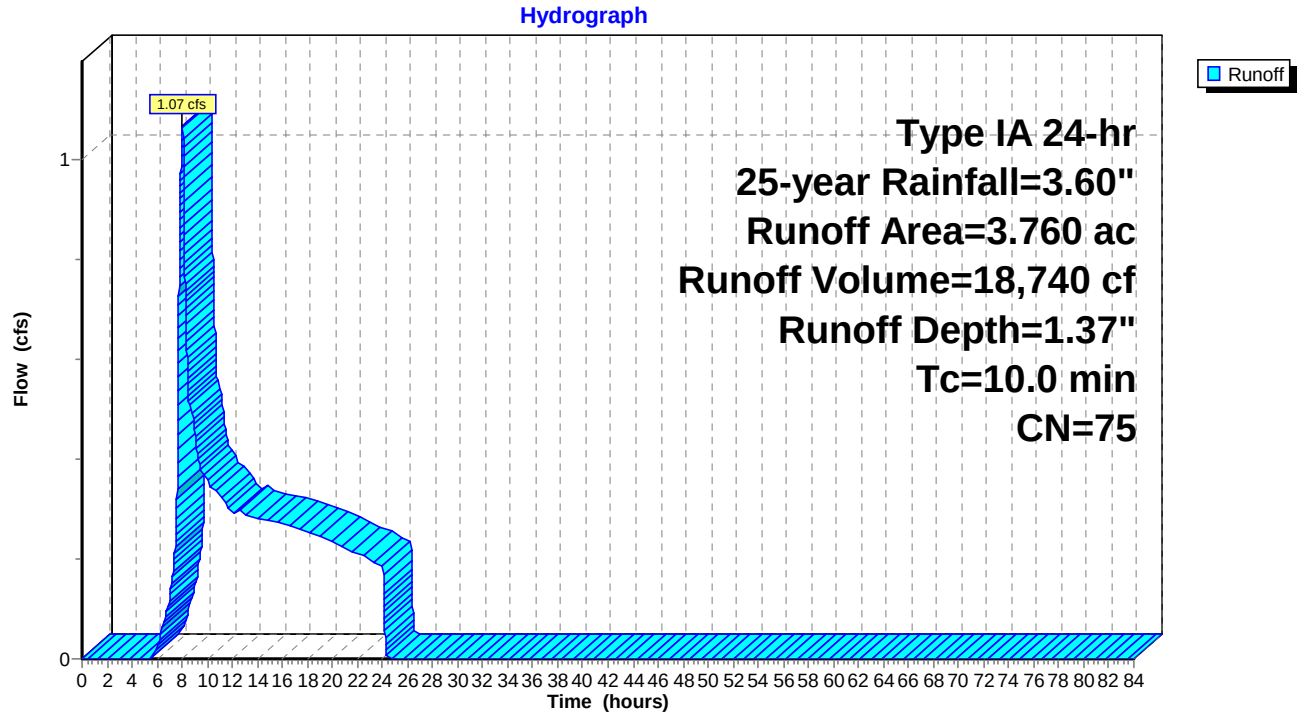
Runoff = 1.07 cfs @ 8.04 hrs, Volume= 18,740 cf, Depth= 1.37"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	3.260	72	City of Salem Pre-developed, HSG C
	0.500	98	Paved roads w/curbs & sewers, HSG C
	3.760	75	Weighted Average
	3.260	72	86.70% Pervious Area
	0.500	98	13.30% Impervious Area

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

Subcatchment OSD: Developed Conditions Off-site



Summary for Pond MH1: Control MH #1

Inflow Area = 599,386 sf, 47.24% Impervious, Inflow Depth = 2.09" for 25-year event
Inflow = 7.05 cfs @ 8.01 hrs, Volume= 104,410 cf
Outflow = 2.14 cfs @ 9.37 hrs, Volume= 94,516 cf, Atten= 70%, Lag= 81.6 min
Primary = 2.14 cfs @ 9.37 hrs, Volume= 94,516 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 544.79' @ 9.37 hrs Surf.Area= 8,500 sf Storage= 27,908 cf
Flood Elev= 561.00' Surf.Area= 8,500 sf Storage= 55,229 cf

Plug-Flow detention time= 312.9 min calculated for 94,504 cf (91% of inflow)
Center-of-Mass det. time= 252.4 min (1,025.7 - 773.2)

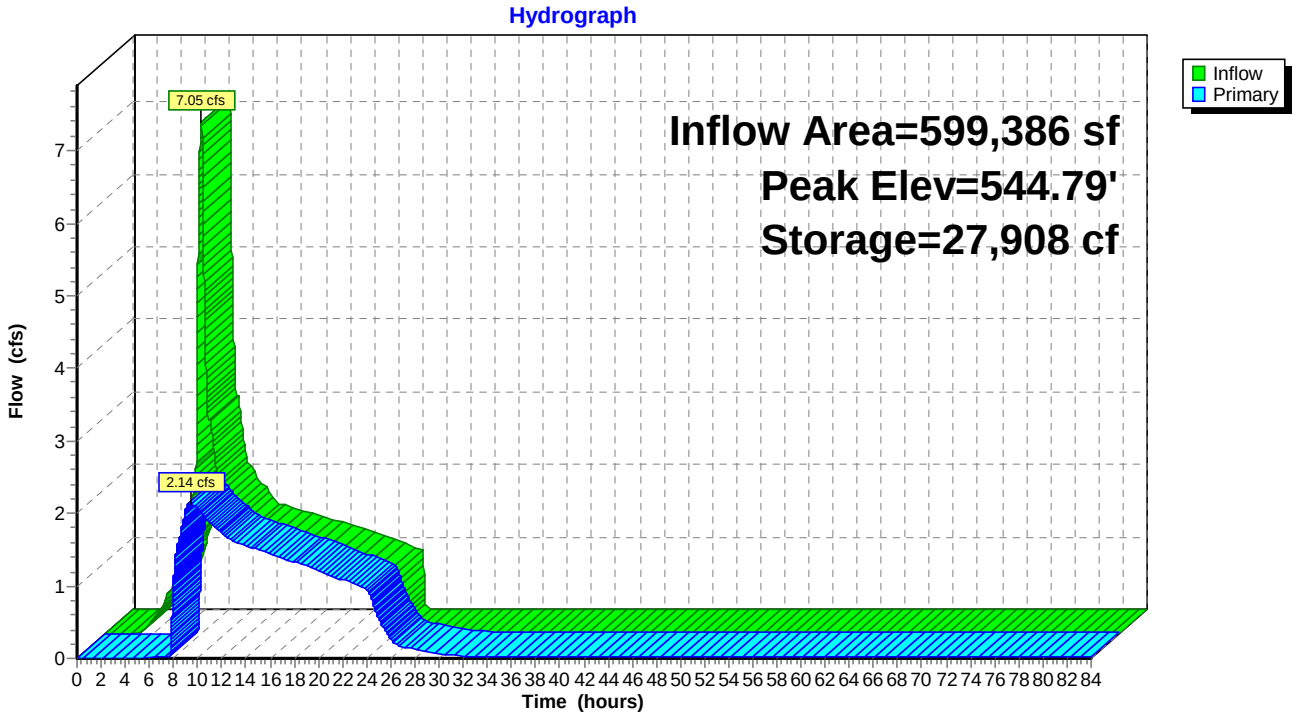
Volume	Invert	Avail.Storage	Storage Description
#1	537.00'	55,229 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
537.00	4	0.0	0	0
539.24	8,500	0.0	0	0
539.25	8,500	40.0	34	34
540.24	8,500	40.0	3,366	3,400
540.25	8,500	5.0	4	3,404
541.99	8,500	5.0	740	4,144
542.00	8,500	100.0	85	4,229
544.00	8,500	100.0	17,000	21,229
546.00	8,500	100.0	17,000	38,229
547.00	8,500	100.0	8,500	46,729
548.00	8,500	100.0	8,500	55,229

Device	Routing	Invert	Outlet Devices
#1	Primary	537.00'	**18.0" Round 18" Pipe** L= 62.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 537.00' / 535.00' S= 0.0323 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	537.00'	**0.5" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	543.00'	**7.5" Vert. Orifice #2** C= 0.600 Limited to weir flow at low heads
#4	Device 1	544.50'	**9.5" Vert. Orifice #3** C= 0.600 Limited to weir flow at low heads
#5	Device 1	547.00'	**24.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=2.10 cfs @ 9.37 hrs HW=544.79' (Free Discharge)
└**1=18" Pipe** (Passes 2.10 cfs of 27.31 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 0.02 cfs @ 13.42 fps)
  ├**3=Orifice #2** (Orifice Controls 1.79 cfs @ 5.84 fps)
  ├**4=Orifice #3** (Orifice Controls 0.29 cfs @ 1.82 fps)
  └**5=Overflow** (Controls 0.00 cfs)

Pond MH1: Control MH #1



Summary for Subcatchment OSN: Existing Conditions Off-site North

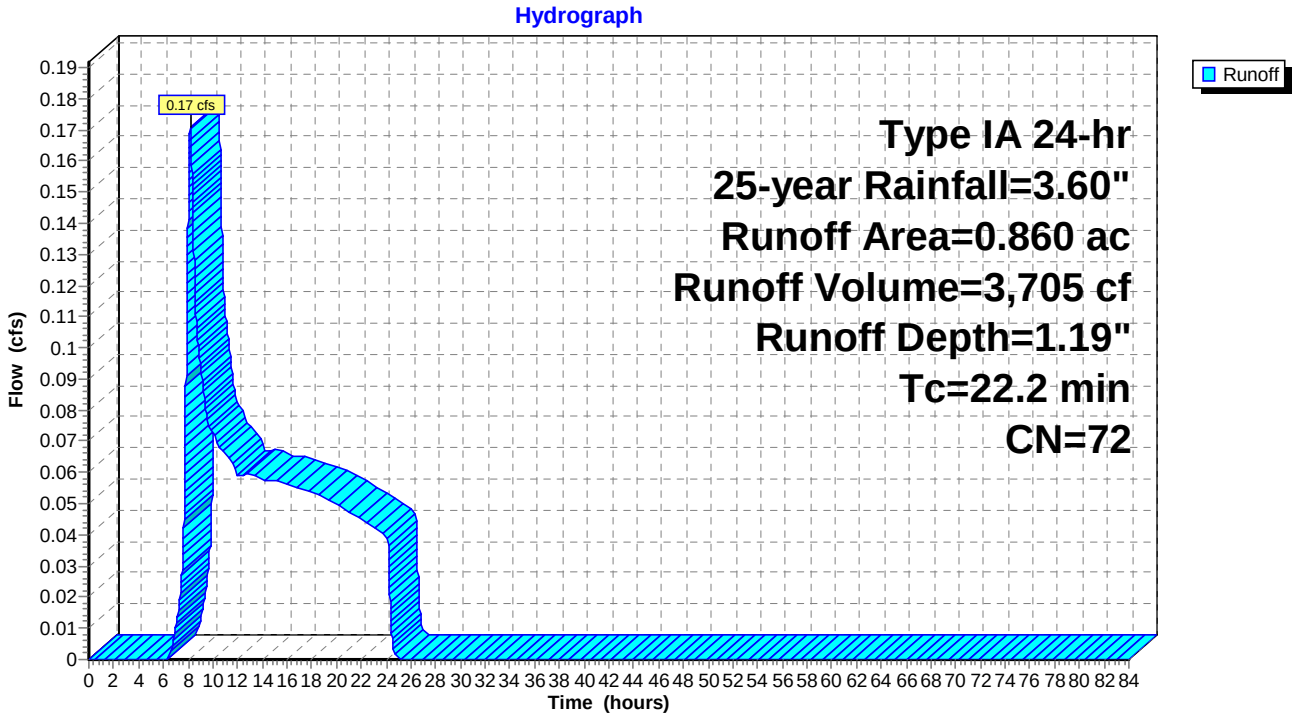
Runoff = 0.17 cfs @ 8.17 hrs, Volume= 3,705 cf, Depth= 1.19"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	0.860	72	City of Salem Pre-development, HSG C
	0.860	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment OSN: Existing Conditions Off-site North



Summary for Subcatchment NOS: Developed Conditions Off-site North

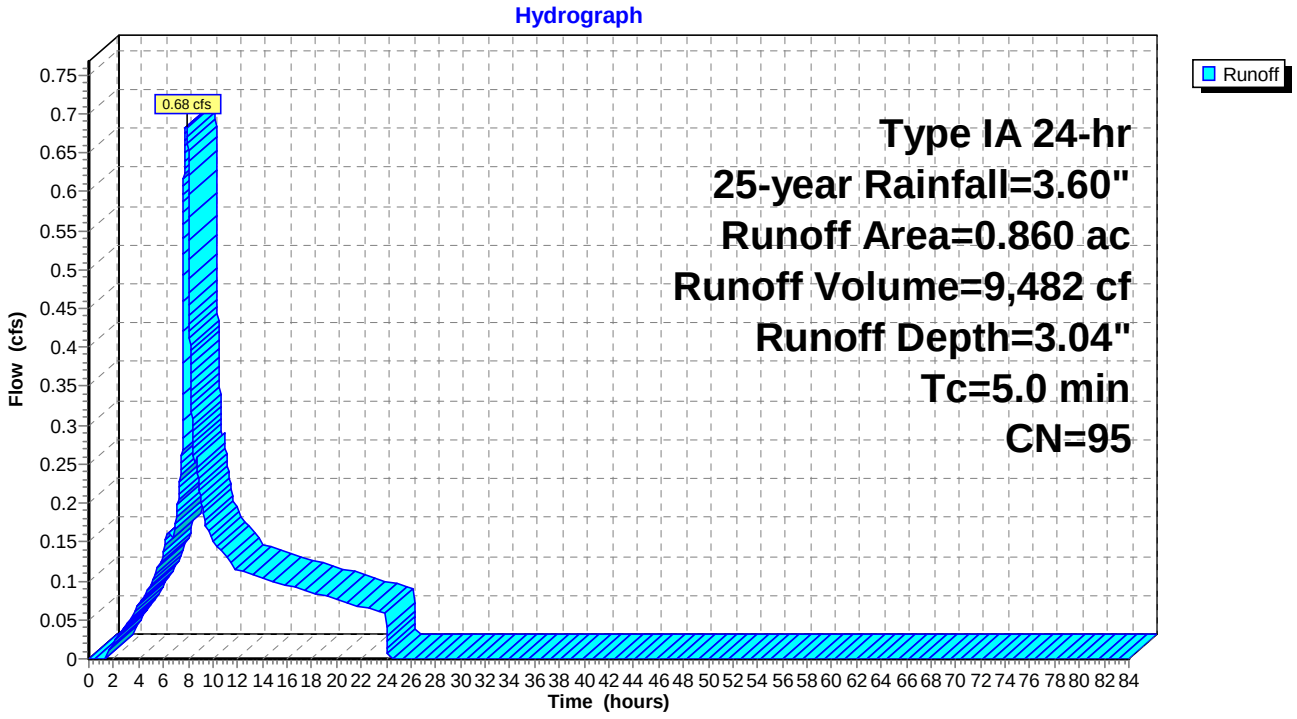
Runoff = 0.68 cfs @ 7.87 hrs, Volume= 9,482 cf, Depth= 3.04"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	0.760	98	Roofs,pavement, concrete, HSG C
	0.100	74	>75% Grass cover, Good, HSG C
	0.860	95	Weighted Average
	0.100	74	11.63% Pervious Area
	0.760	98	88.37% Impervious Area

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

Subcatchment NOS: Developed Conditions Off-site North



Summary for Pond MH2: Control MH #2

Inflow Area = 636,847 sf, 49.66% Impervious, Inflow Depth > 1.96" for 25-year event
Inflow = 2.31 cfs @ 9.29 hrs, Volume= 103,998 cf
Outflow = 2.31 cfs @ 9.39 hrs, Volume= 103,997 cf, Atten= 0%, Lag= 5.7 min
Primary = 2.31 cfs @ 9.39 hrs, Volume= 103,997 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 503.15' @ 9.39 hrs Surf.Area= 456 sf Storage= 286 cf

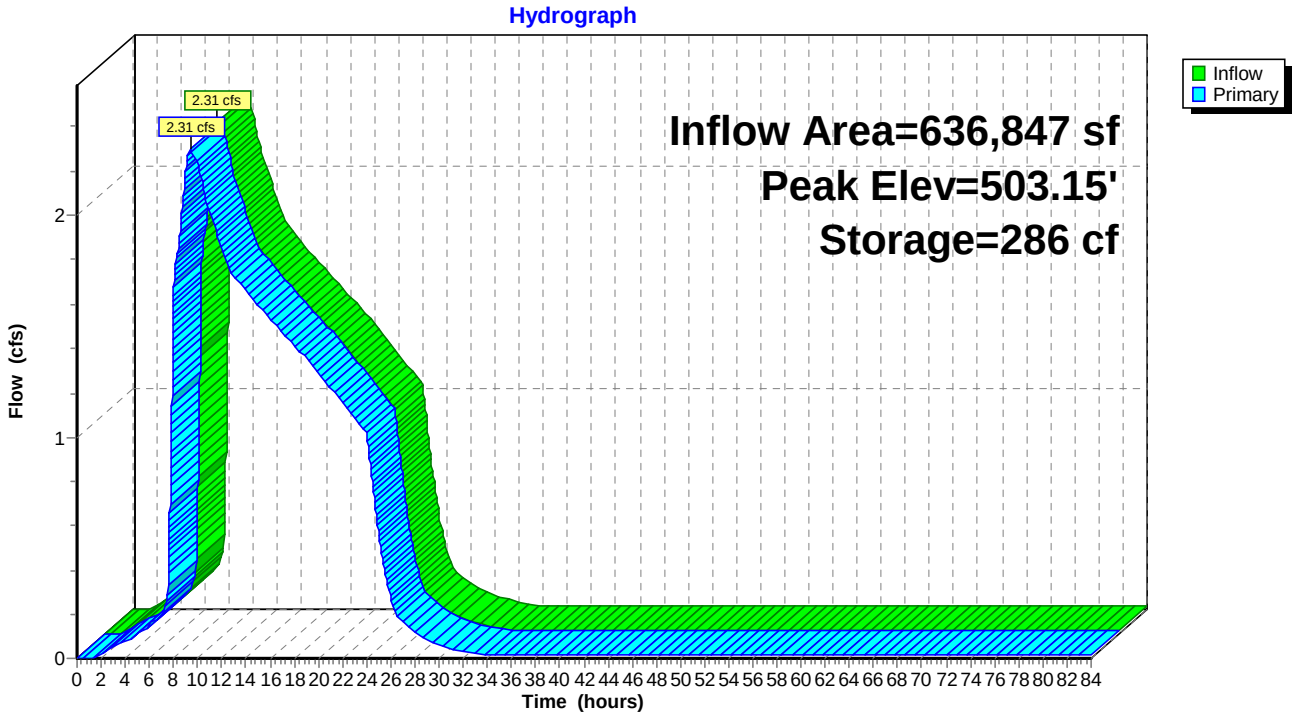
Plug-Flow detention time= 1.3 min calculated for 103,997 cf (100% of inflow)
Center-of-Mass det. time= 1.3 min (996.8 - 995.5)

Volume	Invert	Avail.Storage	Storage Description
#1	501.96'	2,356 cf	**60.0" Round 60" Detention Pipe** L= 120.0' S= 0.0050 '/'

Device	Routing	Invert	Outlet Devices
#1	Primary	499.77'	**18.0" Round Culvert** L= 110.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 499.77' / 484.72' S= 0.1368 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	501.96'	**10.0" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	507.50'	**18.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=2.31 cfs @ 9.39 hrs HW=503.15' (Free Discharge)
└**1=Culvert** (Passes 2.31 cfs of 18.81 cfs potential flow)
└**2=Orifice #1** (Orifice Controls 2.31 cfs @ 4.23 fps)
└**3=Overflow** (Controls 0.00 cfs)

Pond MH2: Control MH #2



Summary for Subcatchment 1E: Existing Conditions Site

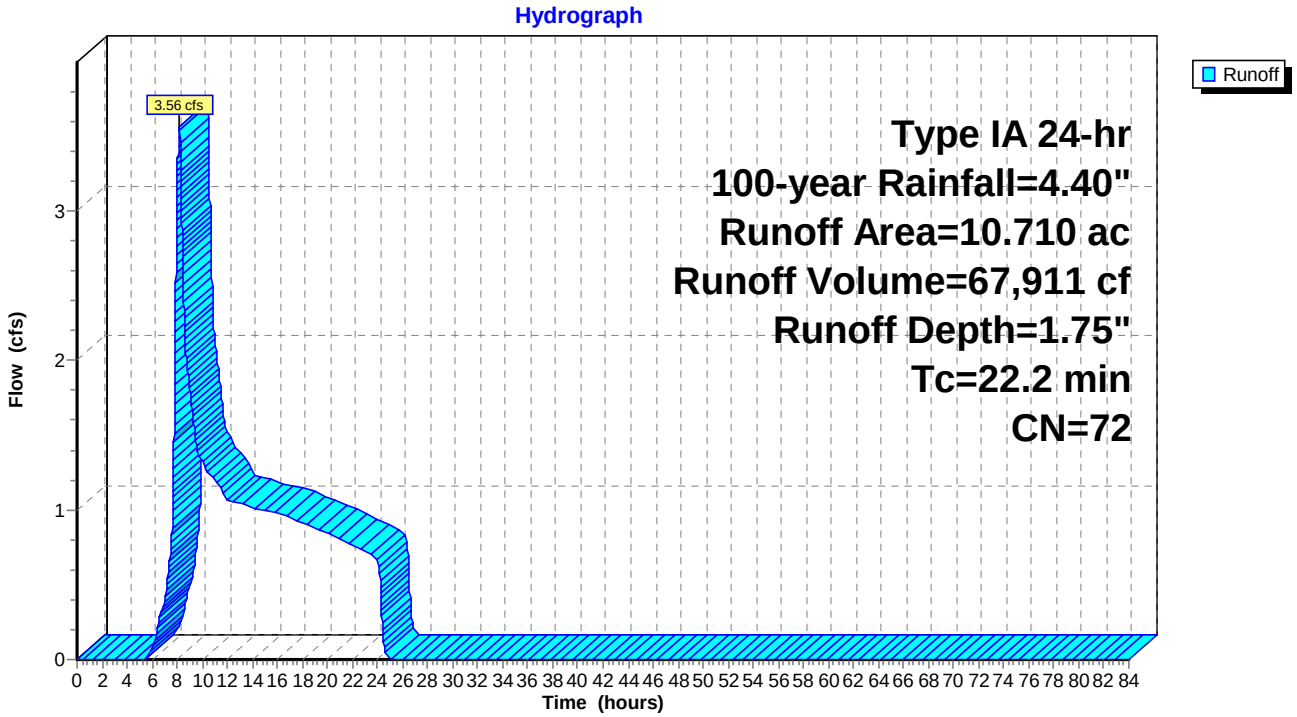
Runoff = 3.56 cfs @ 8.16 hrs, Volume= 67,911 cf, Depth= 1.75"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	10.710	72	City of Salem Pre-development, HSG C
	10.710	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment 1E: Existing Conditions Site



Summary for Subcatchment OSE: Existing Conditions Off-site

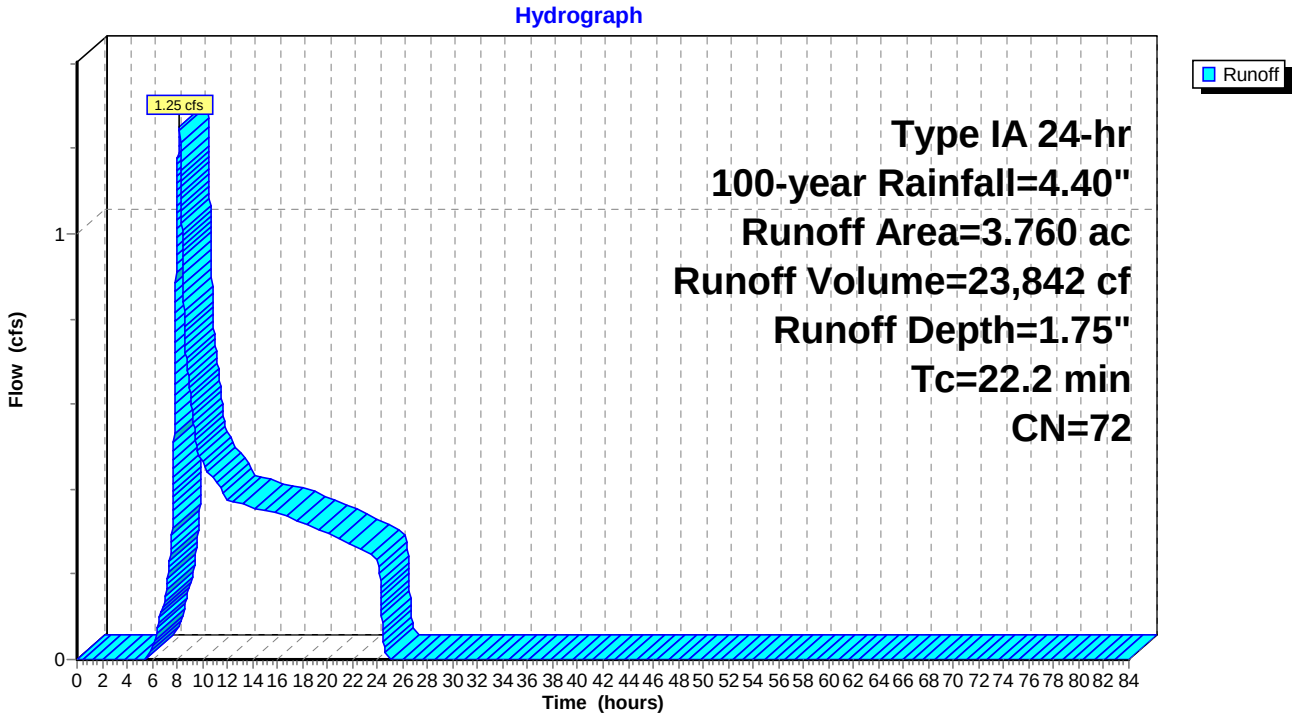
Runoff = 1.25 cfs @ 8.16 hrs, Volume= 23,842 cf, Depth= 1.75"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	3.760	72	City of Salem Pre-development, HSG C
	3.760	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment OSE: Existing Conditions Off-site



Summary for Subcatchment 1B: Developed Conditions Basin 1B

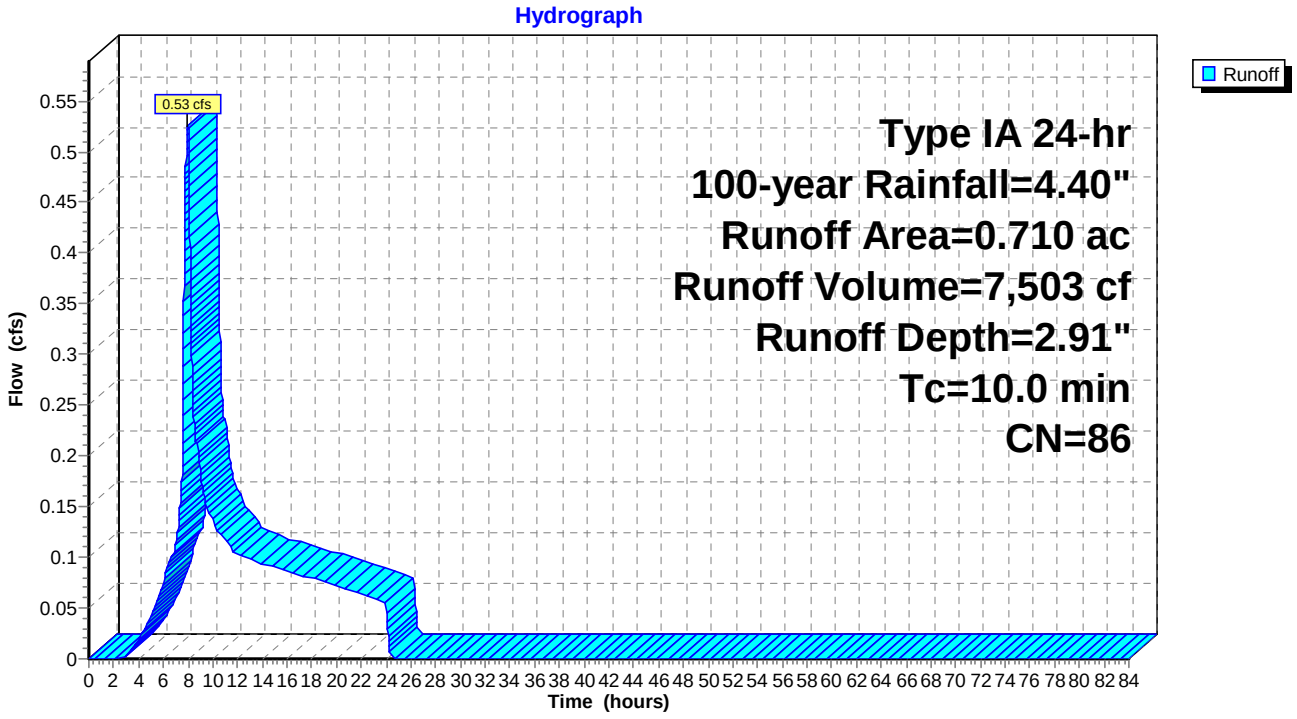
Runoff = 0.53 cfs @ 7.99 hrs, Volume= 7,503 cf, Depth= 2.91"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	0.350	98	Roofs,pavement, concrete, HSG C
	0.360	74	>75% Grass cover, Good, HSG C
	0.710	86	Weighted Average
	0.360	74	50.70% Pervious Area
	0.350	98	49.30% Impervious Area

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

Subcatchment 1B: Developed Conditions Basin 1B



Summary for Subcatchment 1: Developed Conditions Basin 1

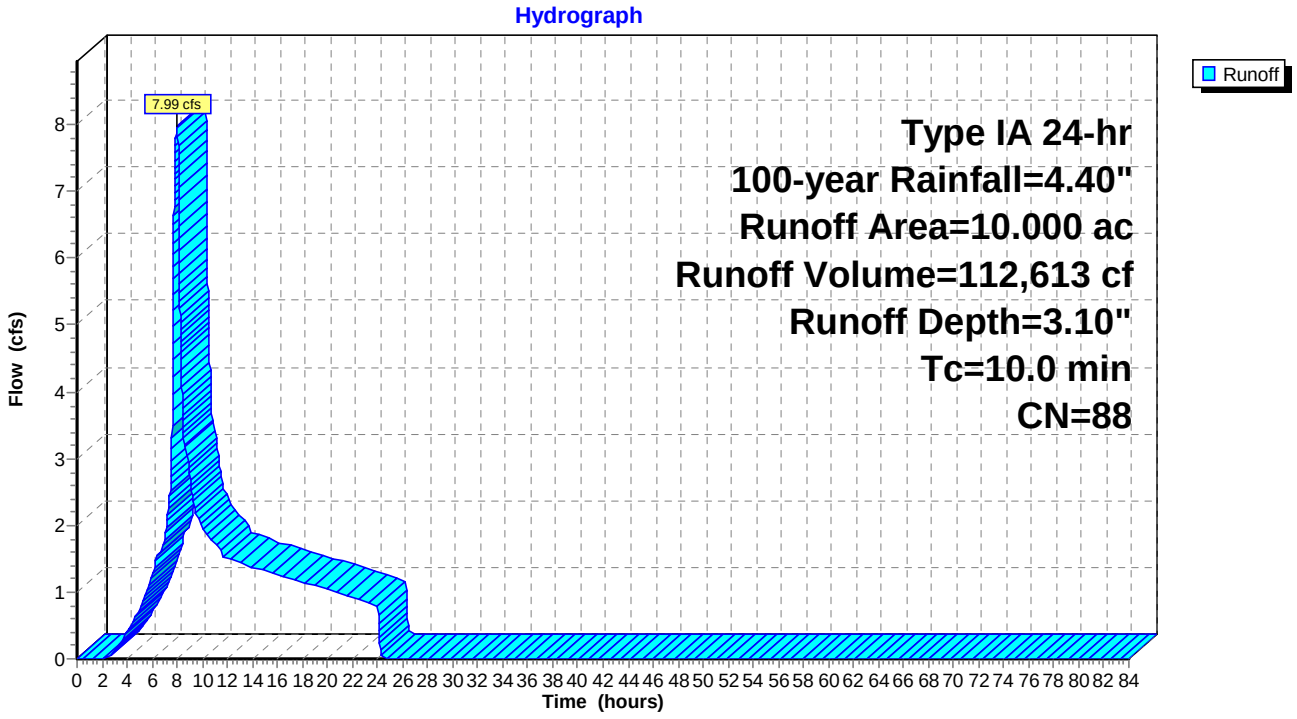
Runoff = 7.99 cfs @ 7.99 hrs, Volume= 112,613 cf, Depth= 3.10"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	6.000	98	Roofs,pavement, concrete, HSG C
	4.000	74	>75% Grass cover, Good, HSG C
	10.000	88	Weighted Average
	4.000	74	40.00% Pervious Area
	6.000	98	60.00% Impervious Area

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

Subcatchment 1: Developed Conditions Basin 1



Summary for Subcatchment OSD: Developed Conditions Off-site

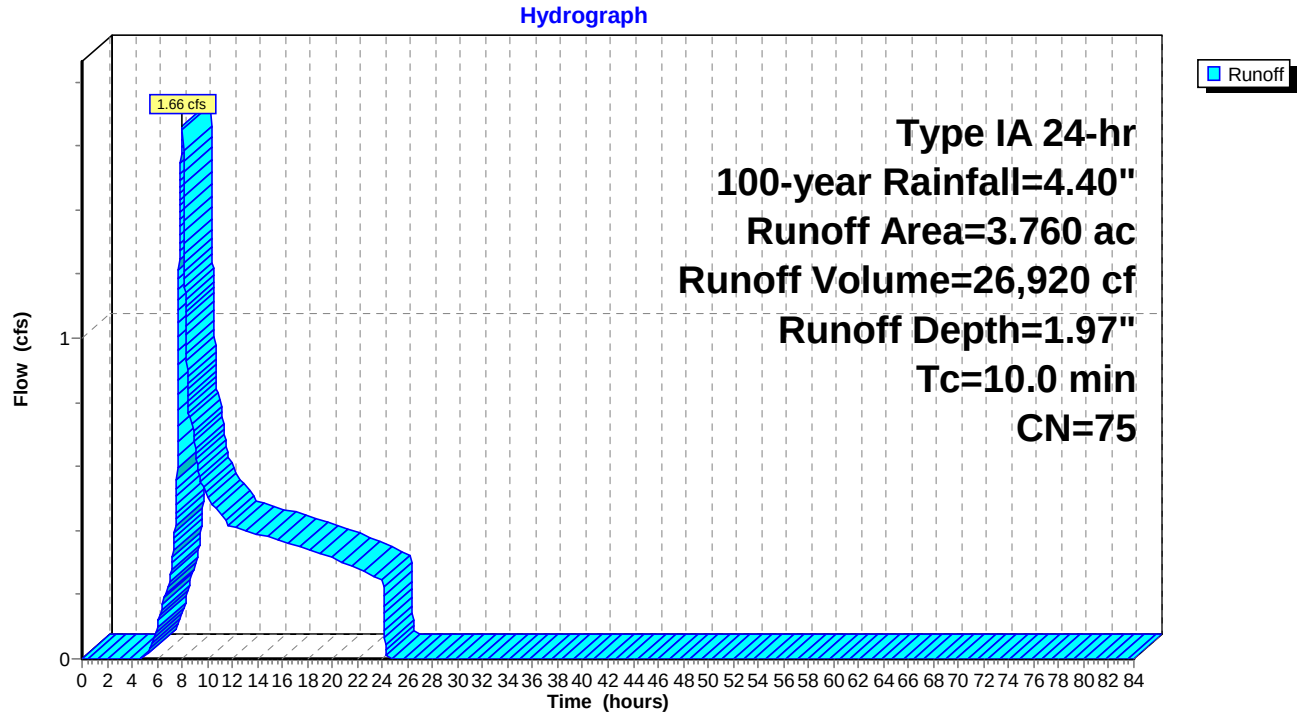
Runoff = 1.66 cfs @ 8.03 hrs, Volume= 26,920 cf, Depth= 1.97"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	3.260	72	City of Salem Pre-developed, HSG C
	0.500	98	Paved roads w/curbs & sewers, HSG C
	3.760	75	Weighted Average
	3.260	72	86.70% Pervious Area
	0.500	98	13.30% Impervious Area

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

Subcatchment OSD: Developed Conditions Off-site



Summary for Pond MH1: Control MH #1

Inflow Area = 599,386 sf, 47.24% Impervious, Inflow Depth = 2.79" for 100-year event
Inflow = 9.63 cfs @ 7.99 hrs, Volume= 139,533 cf
Outflow = 3.90 cfs @ 8.76 hrs, Volume= 129,604 cf, Atten= 60%, Lag= 46.3 min
Primary = 3.90 cfs @ 8.76 hrs, Volume= 129,604 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 545.43' @ 8.76 hrs Surf.Area= 8,500 sf Storage= 33,378 cf
Flood Elev= 561.00' Surf.Area= 8,500 sf Storage= 55,229 cf

Plug-Flow detention time= 261.0 min calculated for 129,588 cf (93% of inflow)
Center-of-Mass det. time= 214.2 min (972.2 - 758.0)

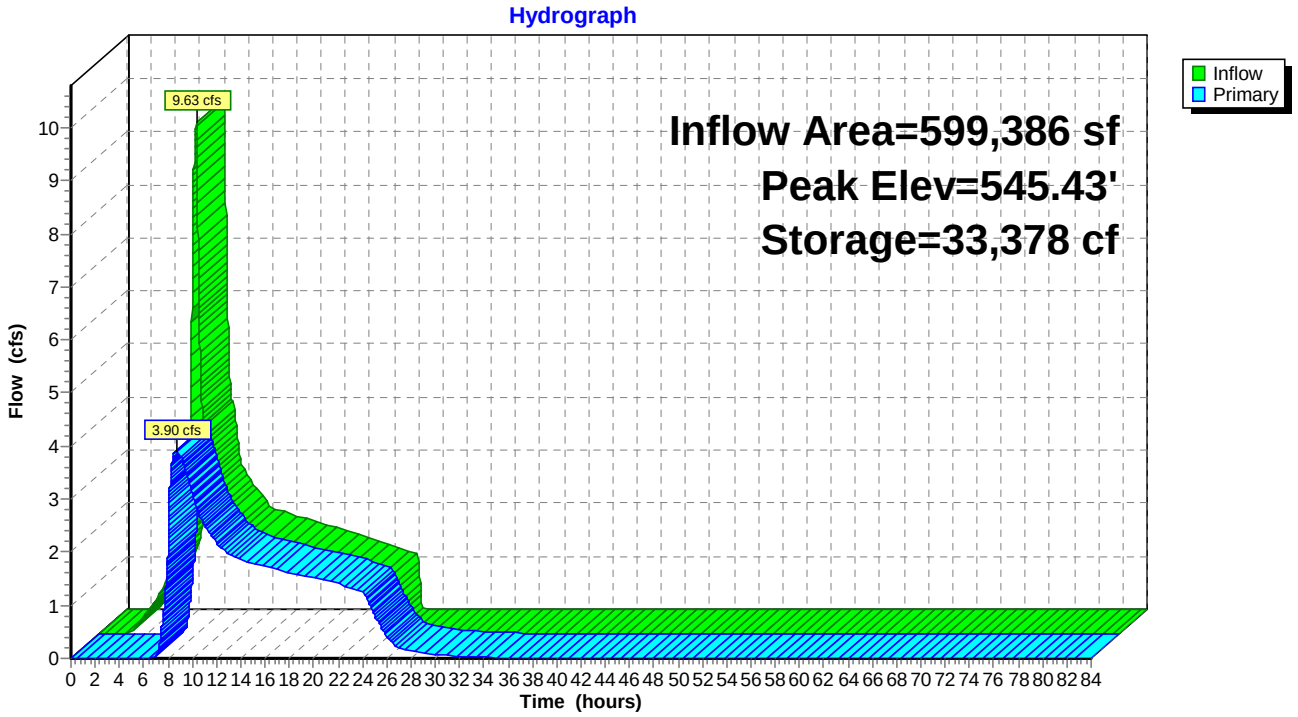
Volume	Invert	Avail.Storage	Storage Description
#1	537.00'	55,229 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
537.00	4	0.0	0	0
539.24	8,500	0.0	0	0
539.25	8,500	40.0	34	34
540.24	8,500	40.0	3,366	3,400
540.25	8,500	5.0	4	3,404
541.99	8,500	5.0	740	4,144
542.00	8,500	100.0	85	4,229
544.00	8,500	100.0	17,000	21,229
546.00	8,500	100.0	17,000	38,229
547.00	8,500	100.0	8,500	46,729
548.00	8,500	100.0	8,500	55,229

Device	Routing	Invert	Outlet Devices
#1	Primary	537.00'	**18.0" Round 18" Pipe** L= 62.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 537.00' / 535.00' S= 0.0323 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	537.00'	**0.5" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	543.00'	**7.5" Vert. Orifice #2** C= 0.600 Limited to weir flow at low heads
#4	Device 1	544.50'	**9.5" Vert. Orifice #3** C= 0.600 Limited to weir flow at low heads
#5	Device 1	547.00'	**24.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=3.90 cfs @ 8.76 hrs HW=545.43' (Free Discharge)
└**1=18" Pipe** (Passes 3.90 cfs of 28.35 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 0.02 cfs @ 13.96 fps)
  ├**3=Orifice #2** (Orifice Controls 2.15 cfs @ 7.01 fps)
  ├**4=Orifice #3** (Orifice Controls 1.73 cfs @ 3.52 fps)
  └**5=Overflow** (Controls 0.00 cfs)

Pond MH1: Control MH #1



Summary for Subcatchment OSN: Existing Conditions Off-site North

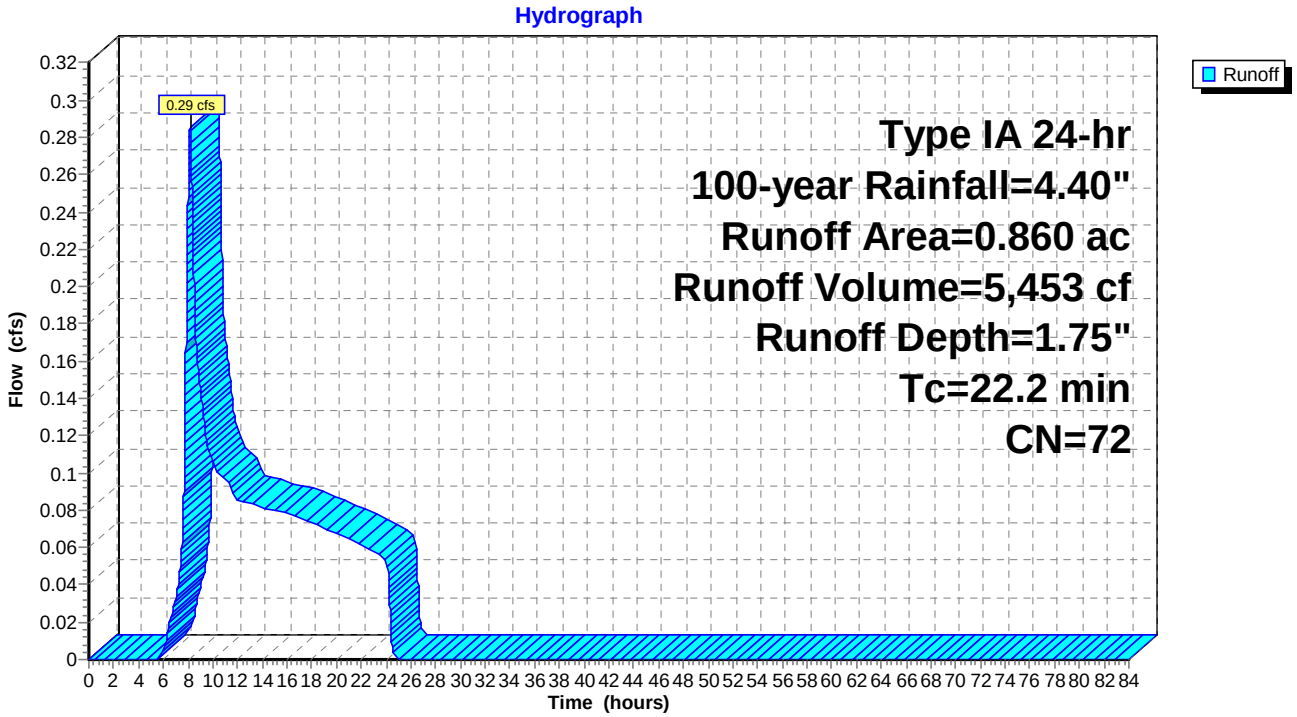
Runoff = 0.29 cfs @ 8.16 hrs, Volume= 5,453 cf, Depth= 1.75"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	0.860	72	City of Salem Pre-development, HSG C
	0.860	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment OSN: Existing Conditions Off-site North



Summary for Subcatchment NOS: Developed Conditions Off-site North

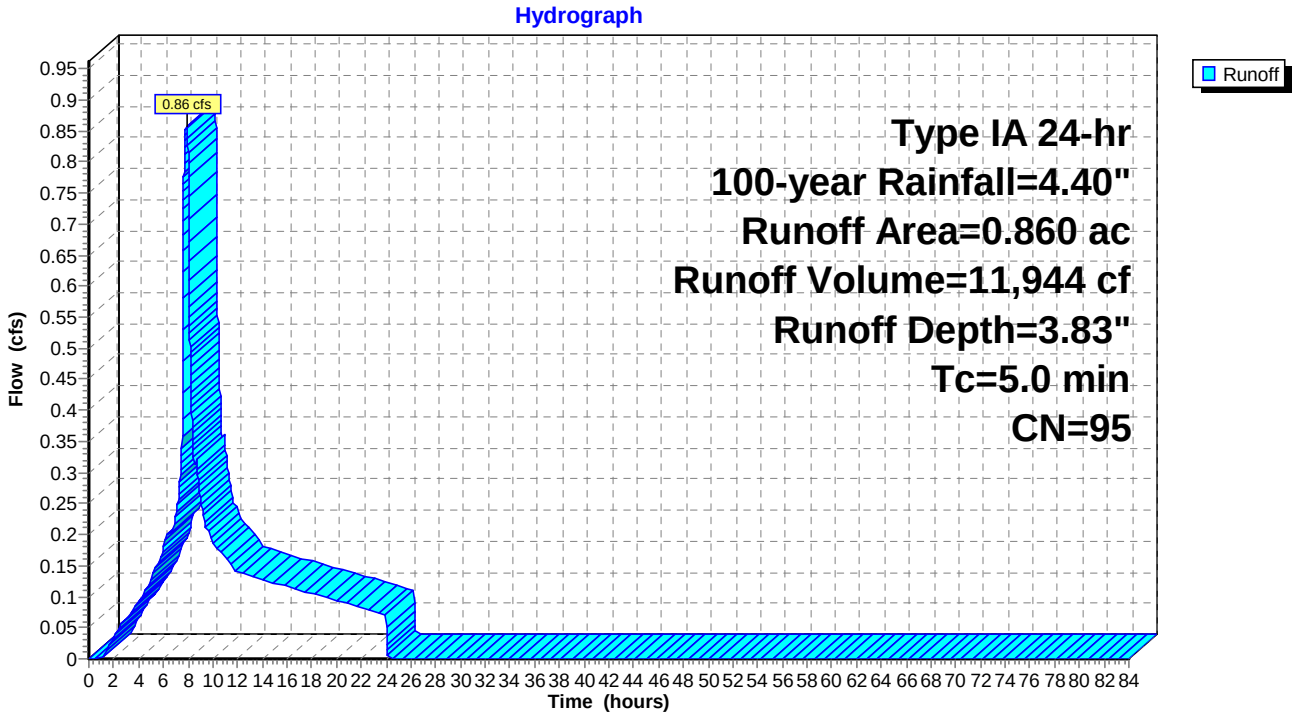
Runoff = 0.86 cfs @ 7.87 hrs, Volume= 11,944 cf, Depth= 3.83"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	0.760	98	Roofs,pavement, concrete, HSG C
	0.100	74	>75% Grass cover, Good, HSG C
	0.860	95	Weighted Average
	0.100	74	11.63% Pervious Area
	0.760	98	88.37% Impervious Area

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

Subcatchment NOS: Developed Conditions Off-site North



Summary for Pond MH2: Control MH #2

Inflow Area = 636,847 sf, 49.66% Impervious, Inflow Depth > 2.67" for 100-year event
Inflow = 4.21 cfs @ 8.62 hrs, Volume= 141,547 cf
Outflow = 4.16 cfs @ 8.89 hrs, Volume= 141,547 cf, Atten= 1%, Lag= 16.2 min
Primary = 4.16 cfs @ 8.89 hrs, Volume= 141,547 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 504.89' @ 8.89 hrs Surf.Area= 598 sf Storage= 1,256 cf

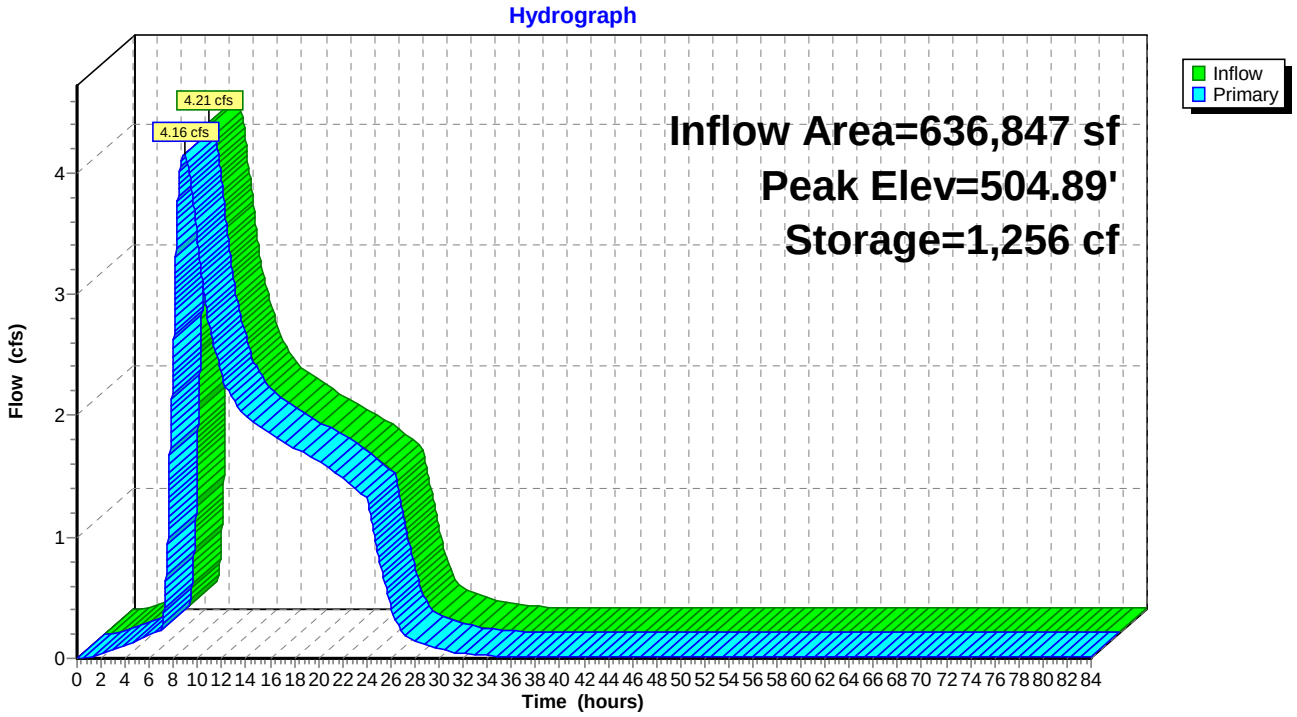
Plug-Flow detention time= 2.2 min calculated for 141,547 cf (100% of inflow)
Center-of-Mass det. time= 2.1 min (950.2 - 948.1)

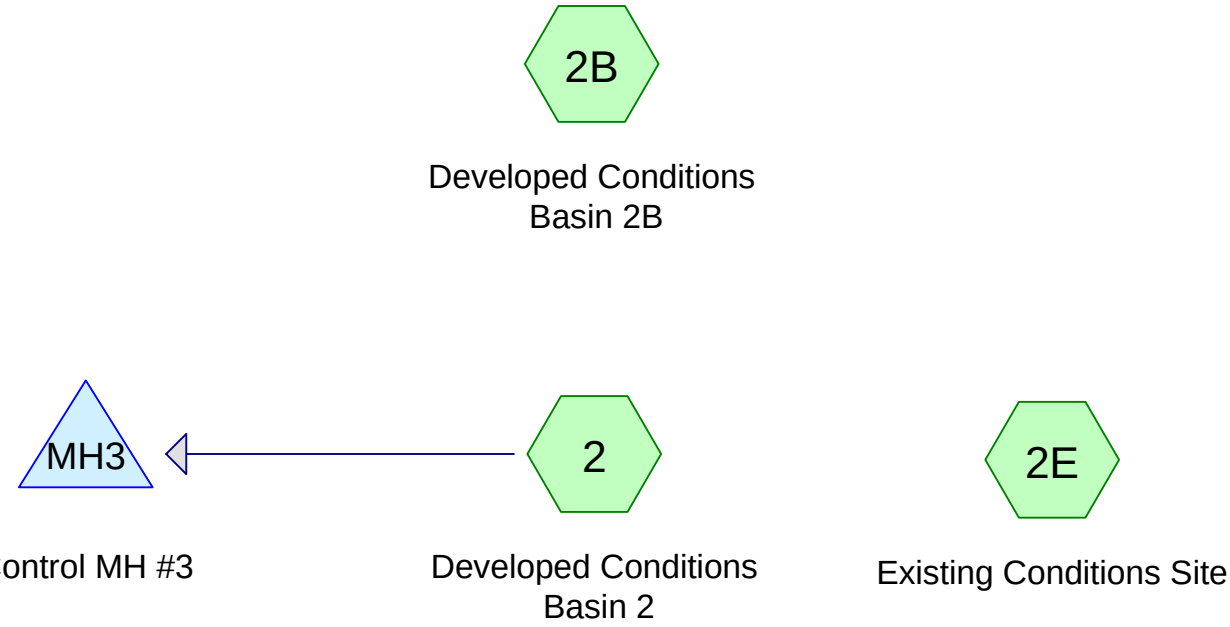
Volume	Invert	Avail.Storage	Storage Description
#1	501.96'	2,356 cf	**60.0" Round 60" Detention Pipe** L= 120.0' S= 0.0050 '/'

Device	Routing	Invert	Outlet Devices
#1	Primary	499.77'	**18.0" Round Culvert** L= 110.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 499.77' / 484.72' S= 0.1368 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	501.96'	**10.0" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	507.50'	**18.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=4.16 cfs @ 8.89 hrs HW=504.89' (Free Discharge)
└**1=Culvert** (Passes 4.16 cfs of 24.26 cfs potential flow)
  └**2=Orifice #1** (Orifice Controls 4.16 cfs @ 7.63 fps)
  └**3=Overflow** (Controls 0.00 cfs)

Pond MH2: Control MH #2





Subcat Reach Pond Link

Routing Diagram for 20210112 Devon Estates Phase 2 Master
Prepared by Multitech Engineering Services, Inc., Printed 2/17/2021

Summary for Subcatchment 2E: Existing Conditions Site

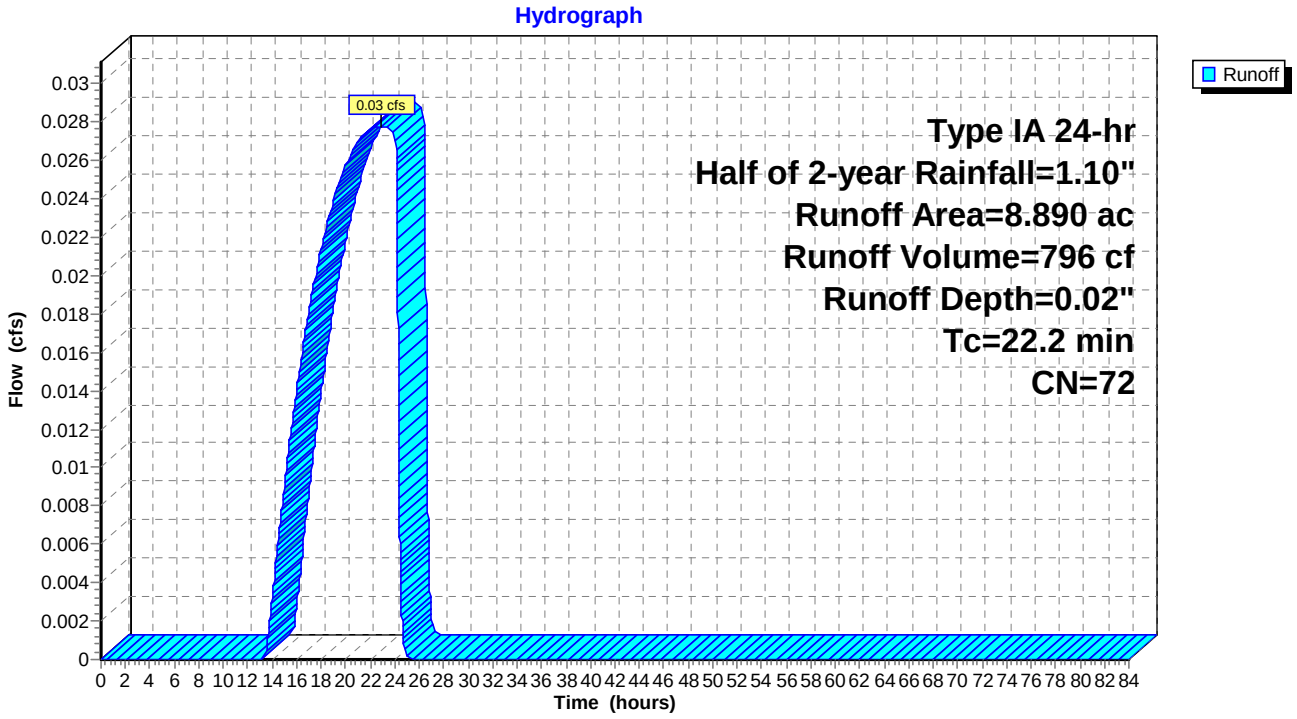
Runoff = 0.03 cfs @ 22.82 hrs, Volume= 796 cf, Depth= 0.02"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	8.890	72	City of Salem Pre-development, HSG C
	8.890	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment 2E: Existing Conditions Site



Summary for Subcatchment 2B: Developed Conditions Basin 2B

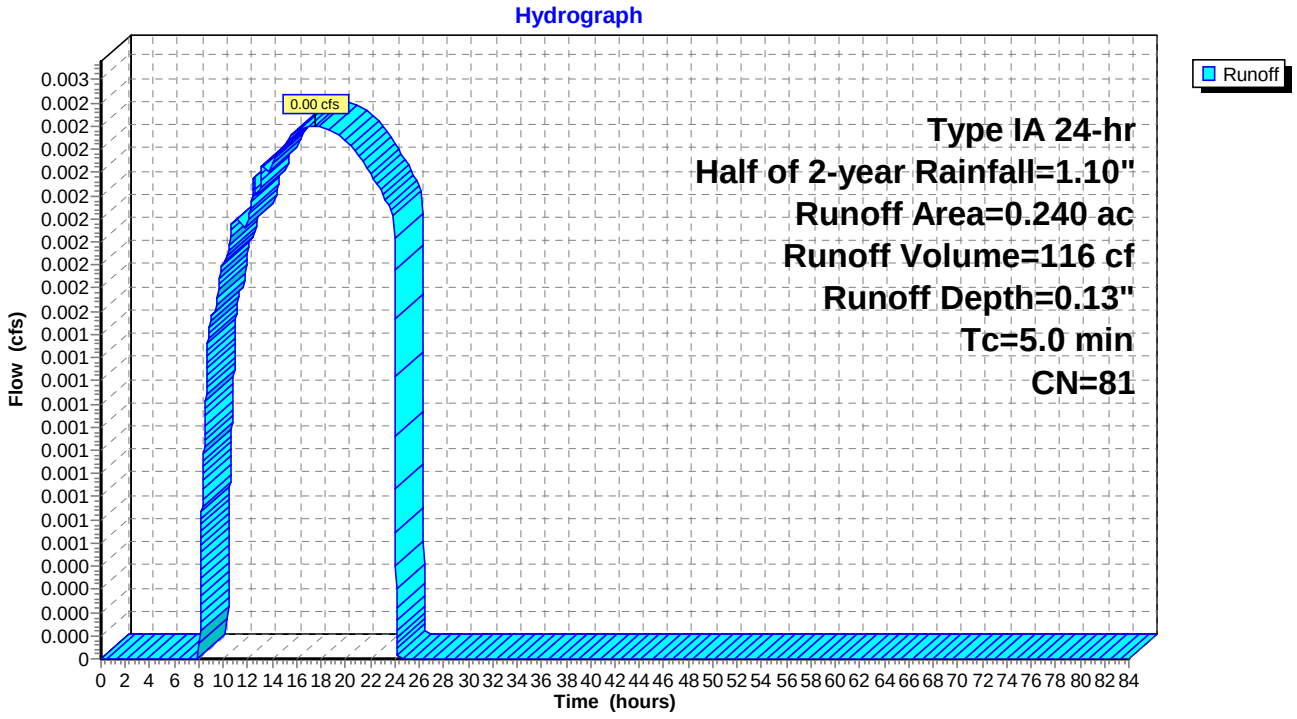
Runoff = 0.00 cfs @ 17.45 hrs, Volume= 116 cf, Depth= 0.13"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	0.070	98	Roofs,pavement, concrete, HSG C
	0.170	74	>75% Grass cover, Good, HSG C
	0.240	81	Weighted Average
	0.170	74	70.83% Pervious Area
	0.070	98	29.17% Impervious Area

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

Subcatchment 2B: Developed Conditions Basin 2B



Summary for Subcatchment 2: Developed Conditions Basin 2

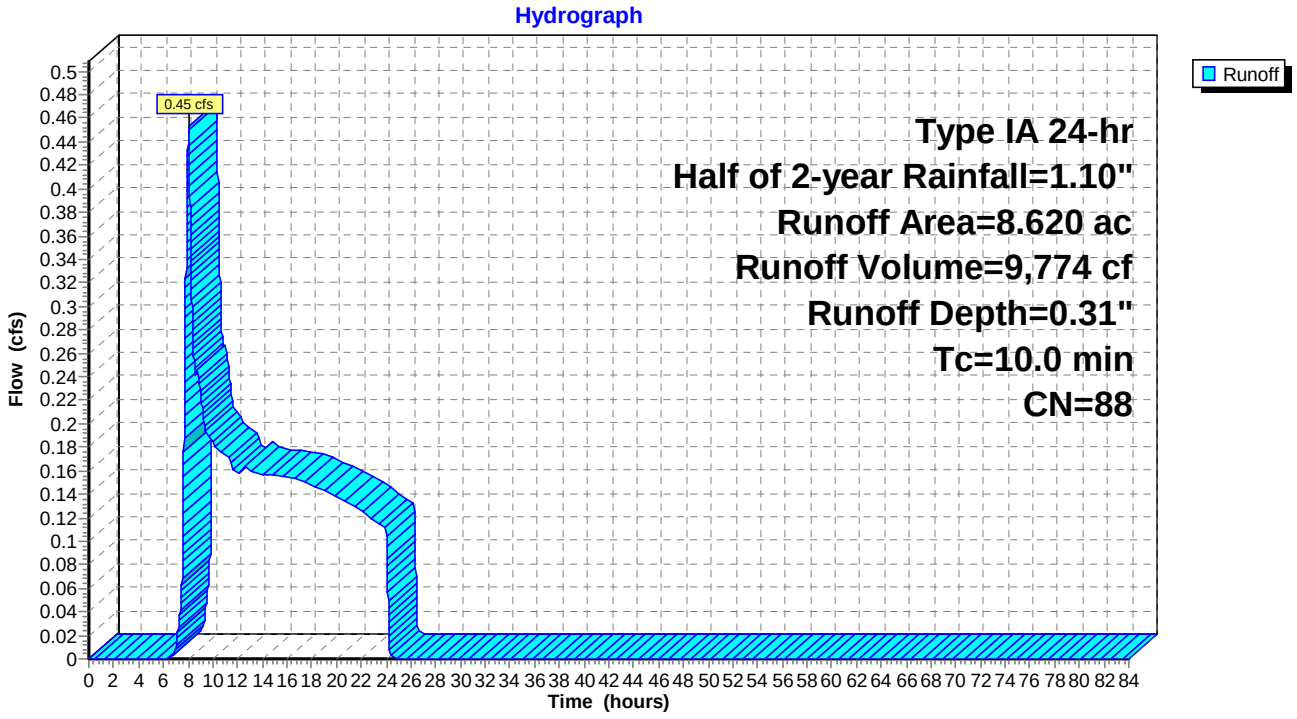
Runoff = 0.45 cfs @ 8.06 hrs, Volume= 9,774 cf, Depth= 0.31"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	5.170	98	Roofs,pavement, concrete, HSG C
	3.450	74	>75% Grass cover, Good, HSG C
	8.620	88	Weighted Average
	3.450	74	40.02% Pervious Area
	5.170	98	59.98% Impervious Area

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

Subcatchment 2: Developed Conditions Basin 2



Summary for Pond MH3: Control MH #3

Inflow Area = 375,487 sf, 59.98% Impervious, Inflow Depth = 0.31" for Half of 2-year event
Inflow = 0.45 cfs @ 8.06 hrs, Volume= 9,774 cf
Outflow = 0.03 cfs @ 24.18 hrs, Volume= 7,061 cf, Atten= 93%, Lag= 967.6 min
Primary = 0.03 cfs @ 24.18 hrs, Volume= 7,061 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 600.54' @ 24.18 hrs Surf.Area= 8,000 sf Storage= 8,322 cf
Flood Elev= 606.00' Surf.Area= 8,000 sf Storage= 51,980 cf

Plug-Flow detention time= 1,918.4 min calculated for 7,060 cf (72% of inflow)
Center-of-Mass det. time= 1,771.9 min (2,656.8 - 884.9)

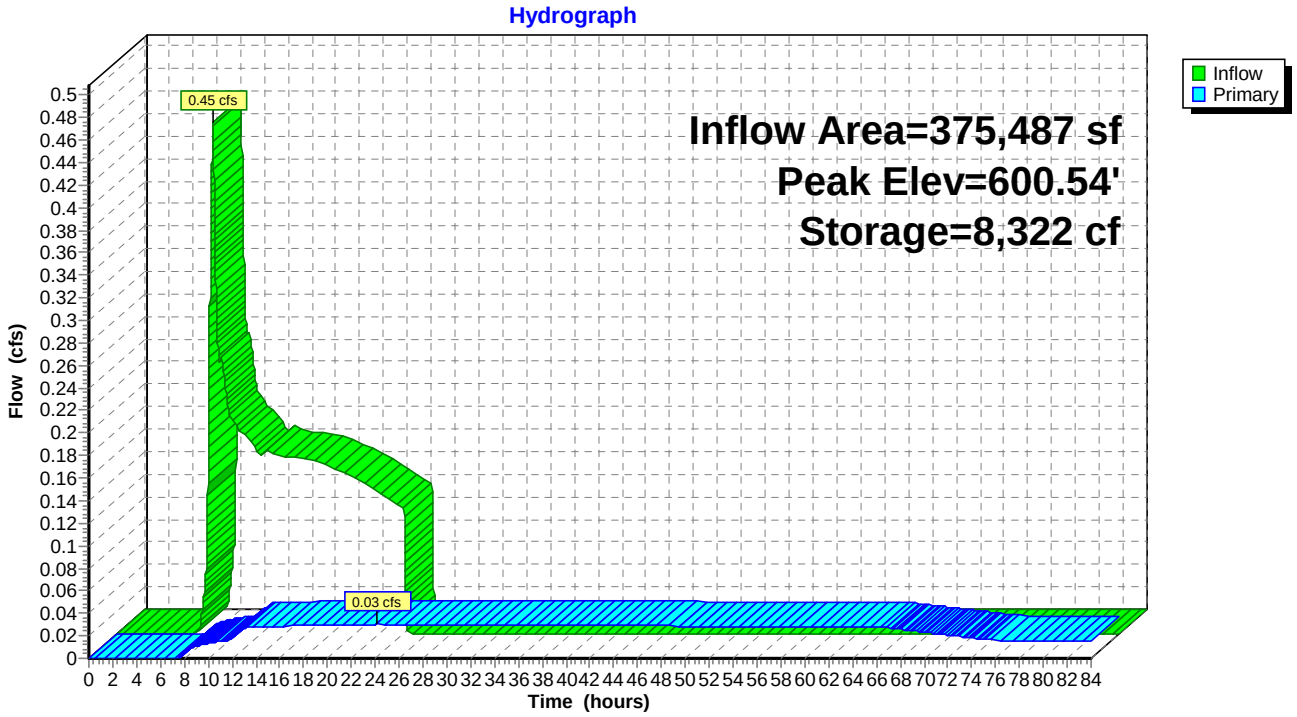
Volume	Invert	Avail.Storage	Storage Description
#1	597.24'	59,980 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
597.24	8,000	0.0	0	0
597.25	8,000	40.0	32	32
598.24	8,000	40.0	3,168	3,200
598.25	8,000	5.0	4	3,204
599.99	8,000	5.0	696	3,900
600.00	8,000	100.0	80	3,980
602.00	8,000	100.0	16,000	19,980
604.00	8,000	100.0	16,000	35,980
606.00	8,000	100.0	16,000	51,980
607.00	8,000	100.0	8,000	59,980

Device	Routing	Invert	Outlet Devices
#1	Primary	597.25'	**18.0" Round 18" Pipe** L= 50.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 597.25' / 597.00' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	597.25'	**0.8" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	600.75'	**6.5" Vert. Orifice #2** C= 0.600 Limited to weir flow at low heads
#4	Device 1	602.00'	**8.0" Vert. Orifice #3** C= 0.600 Limited to weir flow at low heads
#5	Device 1	606.00'	**18.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.03 cfs @ 24.18 hrs HW=600.54' (Free Discharge)
└**1=18" Pipe** (Passes 0.03 cfs of 14.28 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 0.03 cfs @ 8.69 fps)
  ├**3=Orifice #2** (Controls 0.00 cfs)
  ├**4=Orifice #3** (Controls 0.00 cfs)
  └**5=Overflow** (Controls 0.00 cfs)

Pond MH3: Control MH #3



Summary for Subcatchment 2E: Existing Conditions Site

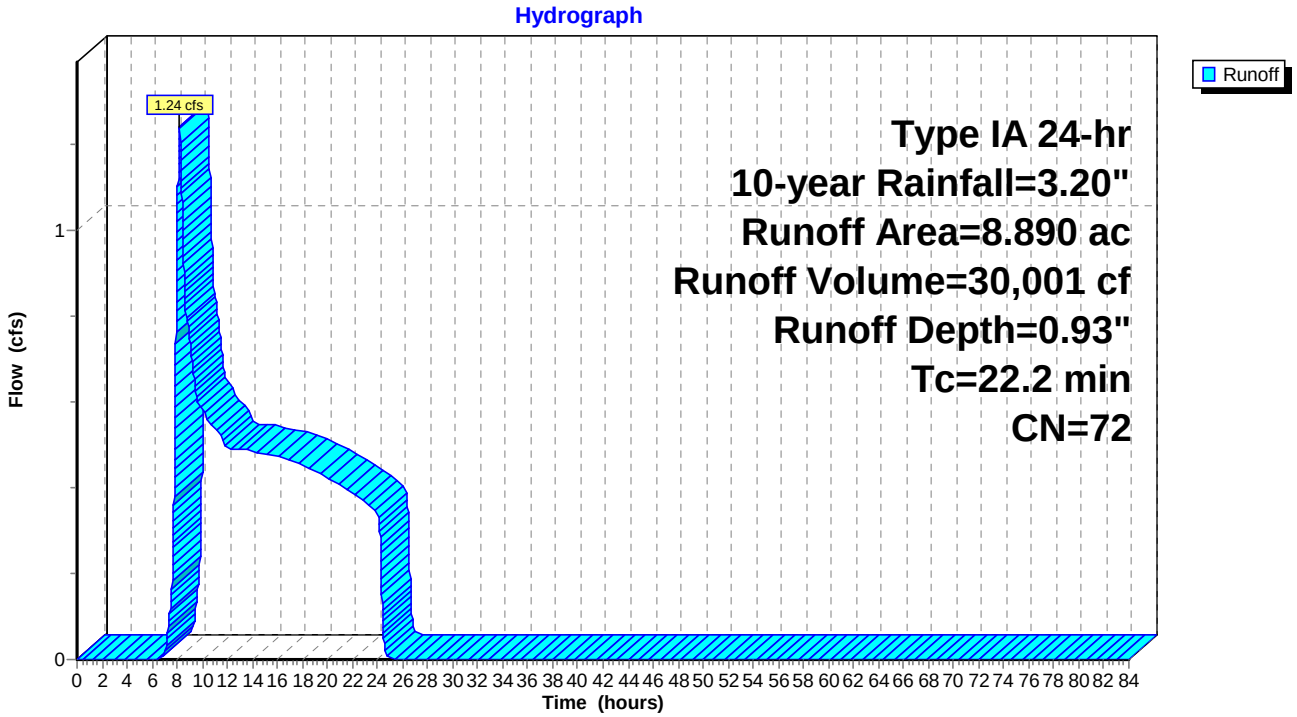
Runoff = 1.24 cfs @ 8.20 hrs, Volume= 30,001 cf, Depth= 0.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	8.890	72	City of Salem Pre-development, HSG C
	8.890	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment 2E: Existing Conditions Site



Summary for Subcatchment 2B: Developed Conditions Basin 2B

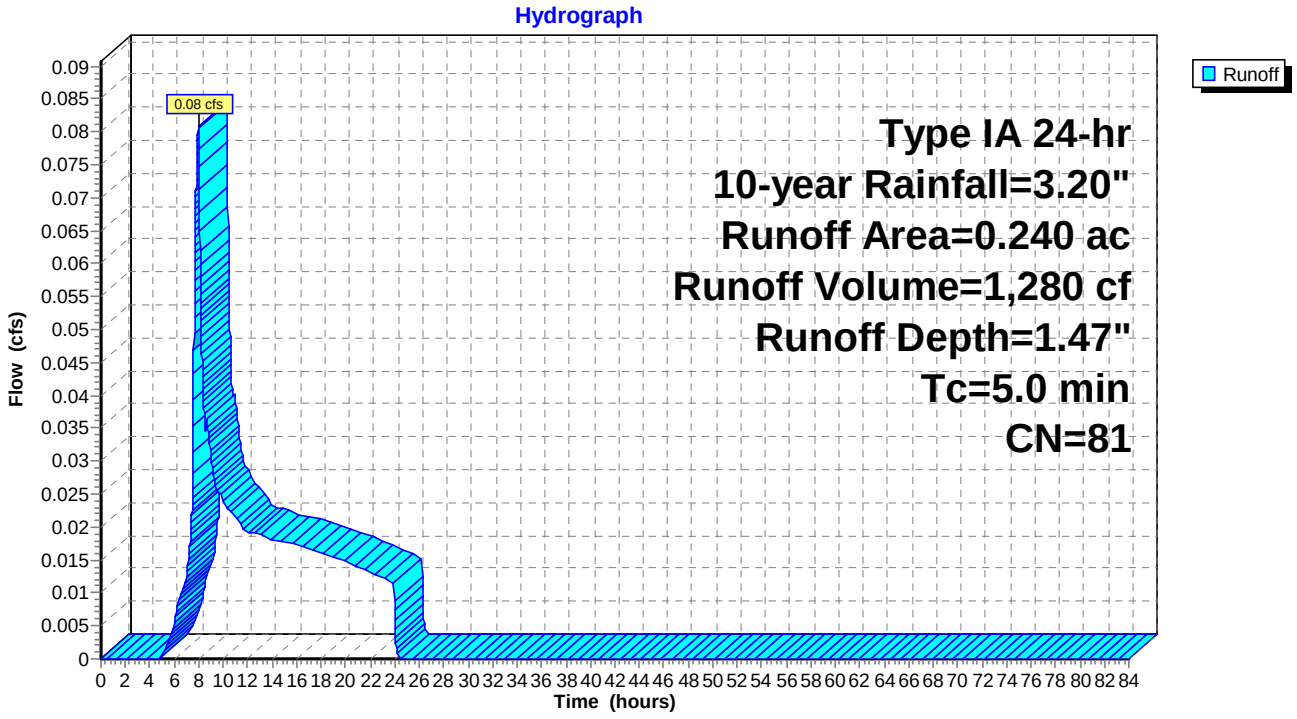
Runoff = 0.08 cfs @ 7.98 hrs, Volume= 1,280 cf, Depth= 1.47"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	0.070	98	Roofs,pavement, concrete, HSG C
	0.170	74	>75% Grass cover, Good, HSG C
	0.240	81	Weighted Average
	0.170	74	70.83% Pervious Area
	0.070	98	29.17% Impervious Area

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

Subcatchment 2B: Developed Conditions Basin 2B



Summary for Subcatchment 2: Developed Conditions Basin 2

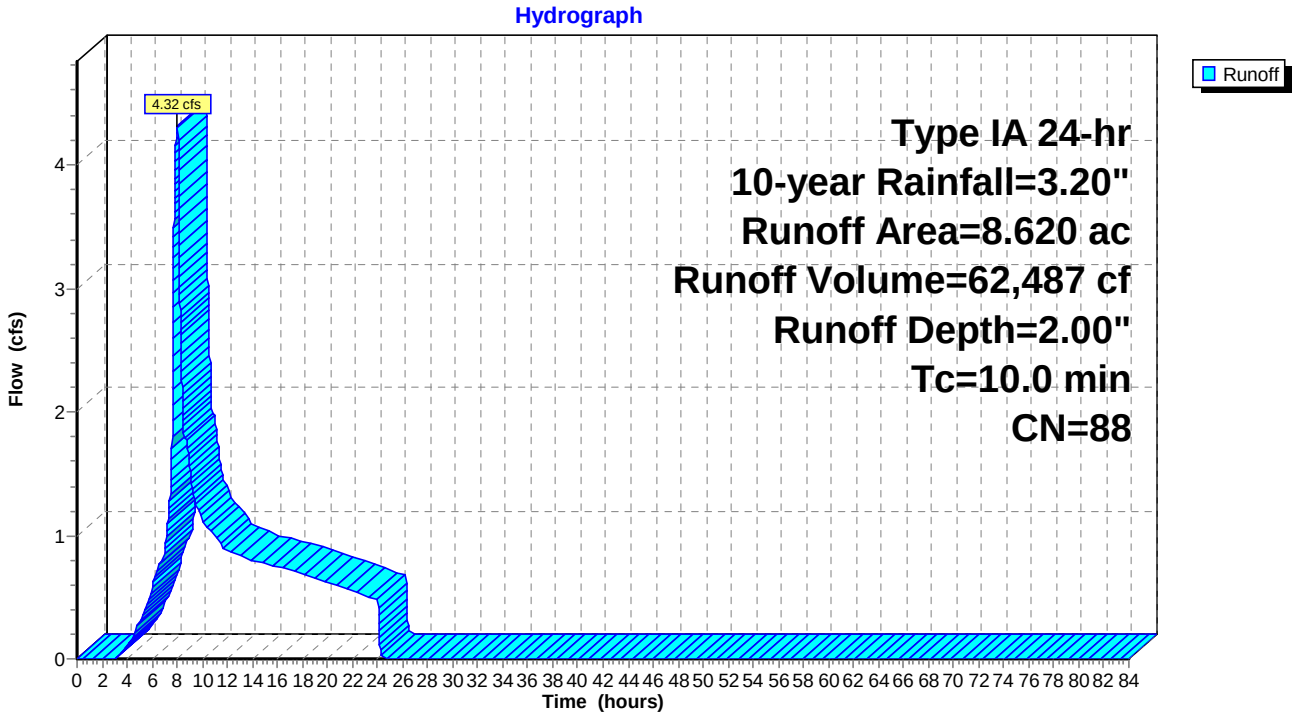
Runoff = 4.32 cfs @ 8.01 hrs, Volume= 62,487 cf, Depth= 2.00"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	5.170	98	Roofs,pavement, concrete, HSG C
	3.450	74	>75% Grass cover, Good, HSG C
	8.620	88	Weighted Average
	3.450	74	40.02% Pervious Area
	5.170	98	59.98% Impervious Area

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

Subcatchment 2: Developed Conditions Basin 2



Summary for Pond MH3: Control MH #3

Inflow Area = 375,487 sf, 59.98% Impervious, Inflow Depth = 2.00" for 10-year event
Inflow = 4.32 cfs @ 8.01 hrs, Volume= 62,487 cf
Outflow = 1.06 cfs @ 10.37 hrs, Volume= 57,912 cf, Atten= 76%, Lag= 141.7 min
Primary = 1.06 cfs @ 10.37 hrs, Volume= 57,912 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 601.87' @ 10.37 hrs Surf.Area= 8,000 sf Storage= 18,934 cf
Flood Elev= 606.00' Surf.Area= 8,000 sf Storage= 51,980 cf

Plug-Flow detention time= 483.3 min calculated for 57,912 cf (93% of inflow)
Center-of-Mass det. time= 435.4 min (1,202.4 - 766.9)

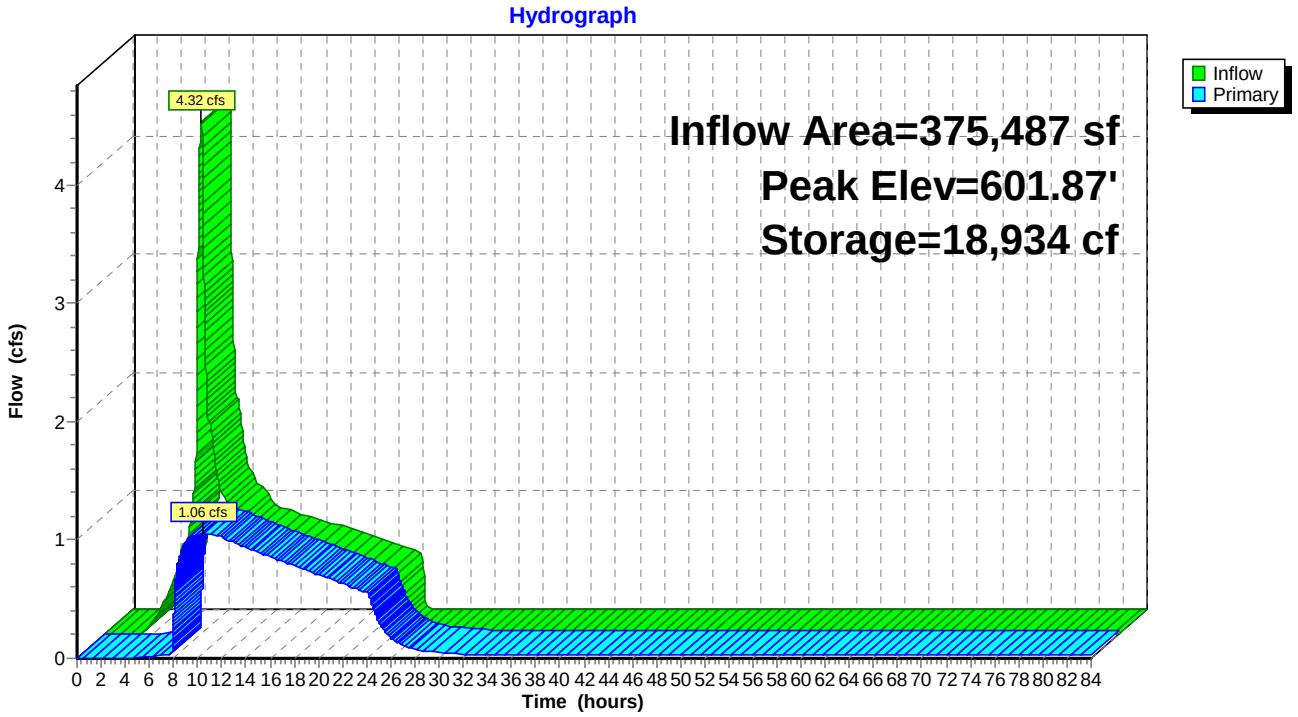
Volume	Invert	Avail.Storage	Storage Description
#1	597.24'	59,980 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
597.24	8,000	0.0	0	0
597.25	8,000	40.0	32	32
598.24	8,000	40.0	3,168	3,200
598.25	8,000	5.0	4	3,204
599.99	8,000	5.0	696	3,900
600.00	8,000	100.0	80	3,980
602.00	8,000	100.0	16,000	19,980
604.00	8,000	100.0	16,000	35,980
606.00	8,000	100.0	16,000	51,980
607.00	8,000	100.0	8,000	59,980

Device	Routing	Invert	Outlet Devices
#1	Primary	597.25'	**18.0" Round 18" Pipe** L= 50.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 597.25' / 597.00' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	597.25'	**0.8" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	600.75'	**6.5" Vert. Orifice #2** C= 0.600 Limited to weir flow at low heads
#4	Device 1	602.00'	**8.0" Vert. Orifice #3** C= 0.600 Limited to weir flow at low heads
#5	Device 1	606.00'	**18.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=1.06 cfs @ 10.37 hrs HW=601.87' (Free Discharge)
└**1=18" Pipe** (Passes 1.06 cfs of 18.34 cfs potential flow)
├**2=Orifice #1** (Orifice Controls 0.04 cfs @ 10.31 fps)
├**3=Orifice #2** (Orifice Controls 1.02 cfs @ 4.44 fps)
├**4=Orifice #3** (Controls 0.00 cfs)
└**5=Overflow** (Controls 0.00 cfs)

Pond MH3: Control MH #3



Summary for Subcatchment 2E: Existing Conditions Site

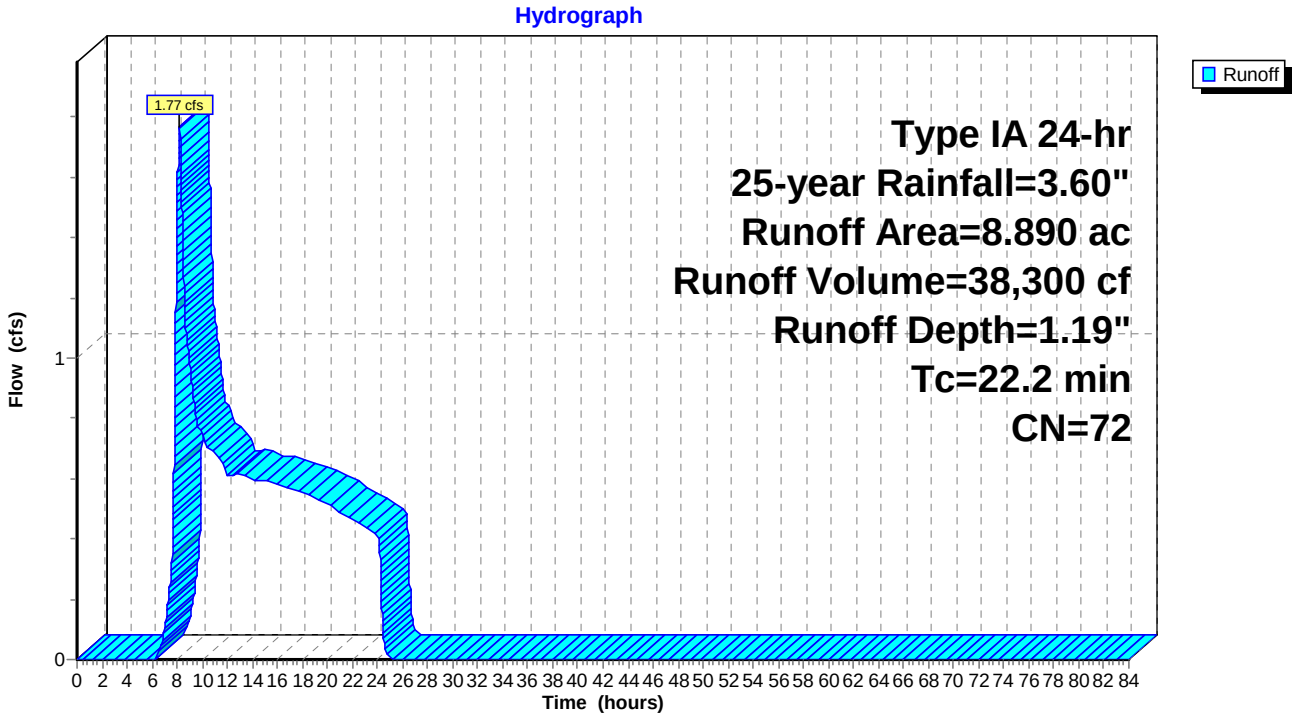
Runoff = 1.77 cfs @ 8.17 hrs, Volume= 38,300 cf, Depth= 1.19"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	8.890	72	City of Salem Pre-development, HSG C
	8.890	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment 2E: Existing Conditions Site



Summary for Subcatchment 2B: Developed Conditions Basin 2B

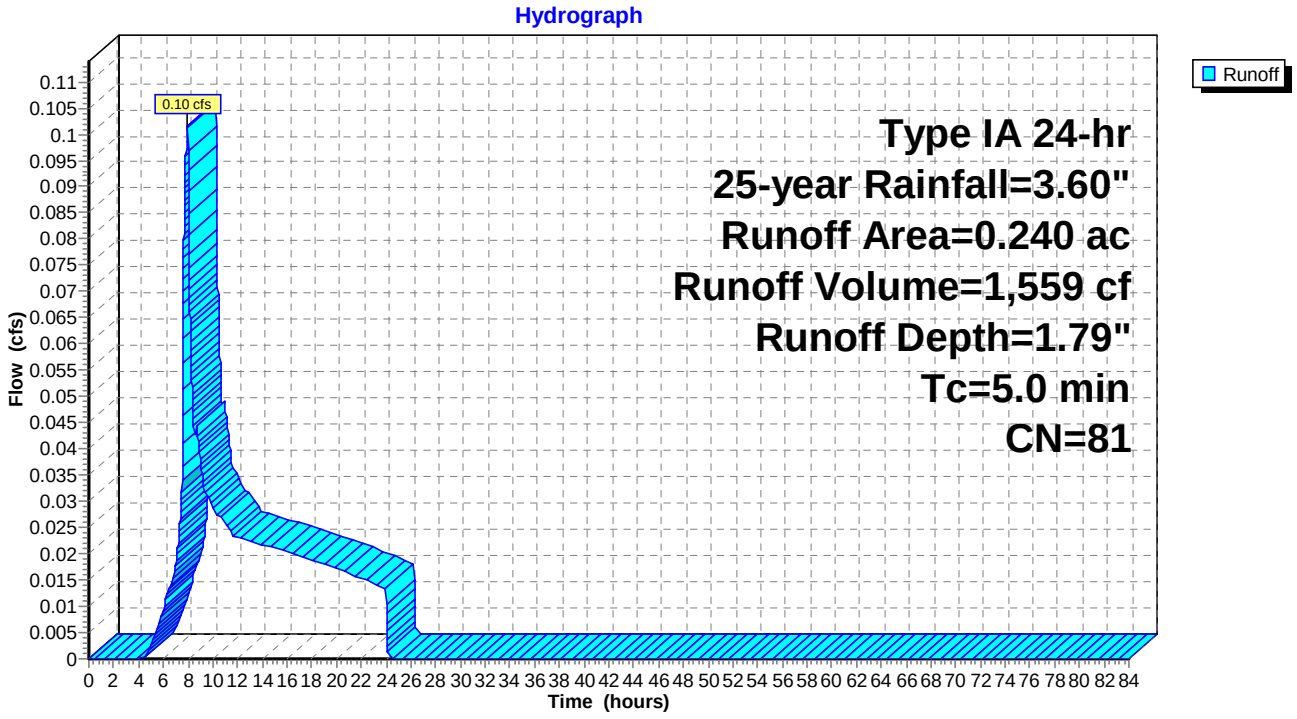
Runoff = 0.10 cfs @ 7.96 hrs, Volume= 1,559 cf, Depth= 1.79"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	0.070	98	Roofs,pavement, concrete, HSG C
	0.170	74	>75% Grass cover, Good, HSG C
	0.240	81	Weighted Average
	0.170	74	70.83% Pervious Area
	0.070	98	29.17% Impervious Area

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

Subcatchment 2B: Developed Conditions Basin 2B



Summary for Subcatchment 2: Developed Conditions Basin 2

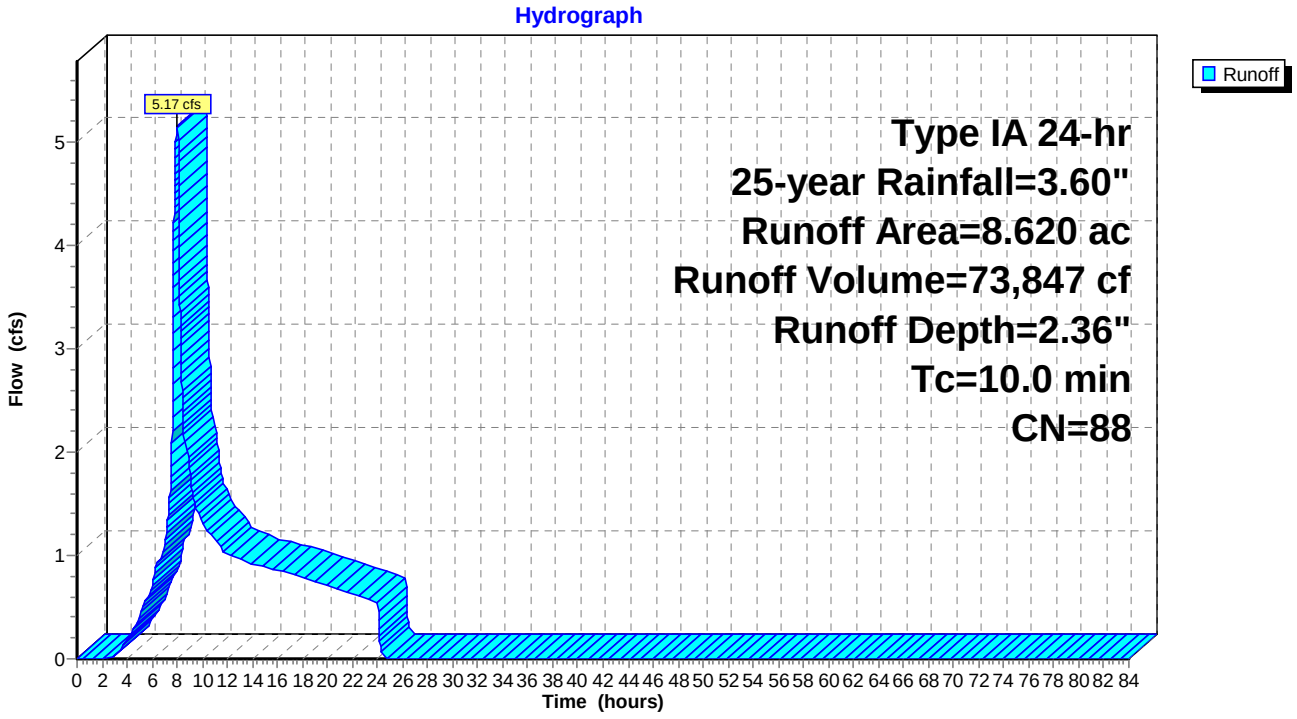
Runoff = 5.17 cfs @ 7.99 hrs, Volume= 73,847 cf, Depth= 2.36"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	5.170	98	Roofs,pavement, concrete, HSG C
	3.450	74	>75% Grass cover, Good, HSG C
	8.620	88	Weighted Average
	3.450	74	40.02% Pervious Area
	5.170	98	59.98% Impervious Area

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

Subcatchment 2: Developed Conditions Basin 2



Summary for Pond MH3: Control MH #3

Inflow Area = 375,487 sf, 59.98% Impervious, Inflow Depth = 2.36" for 25-year event
Inflow = 5.17 cfs @ 7.99 hrs, Volume= 73,847 cf
Outflow = 1.42 cfs @ 9.54 hrs, Volume= 69,238 cf, Atten= 73%, Lag= 92.9 min
Primary = 1.42 cfs @ 9.54 hrs, Volume= 69,238 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 602.22' @ 9.54 hrs Surf.Area= 8,000 sf Storage= 21,753 cf
Flood Elev= 606.00' Surf.Area= 8,000 sf Storage= 51,980 cf

Plug-Flow detention time= 434.4 min calculated for 69,229 cf (94% of inflow)
Center-of-Mass det. time= 393.2 min (1,150.8 - 757.6)

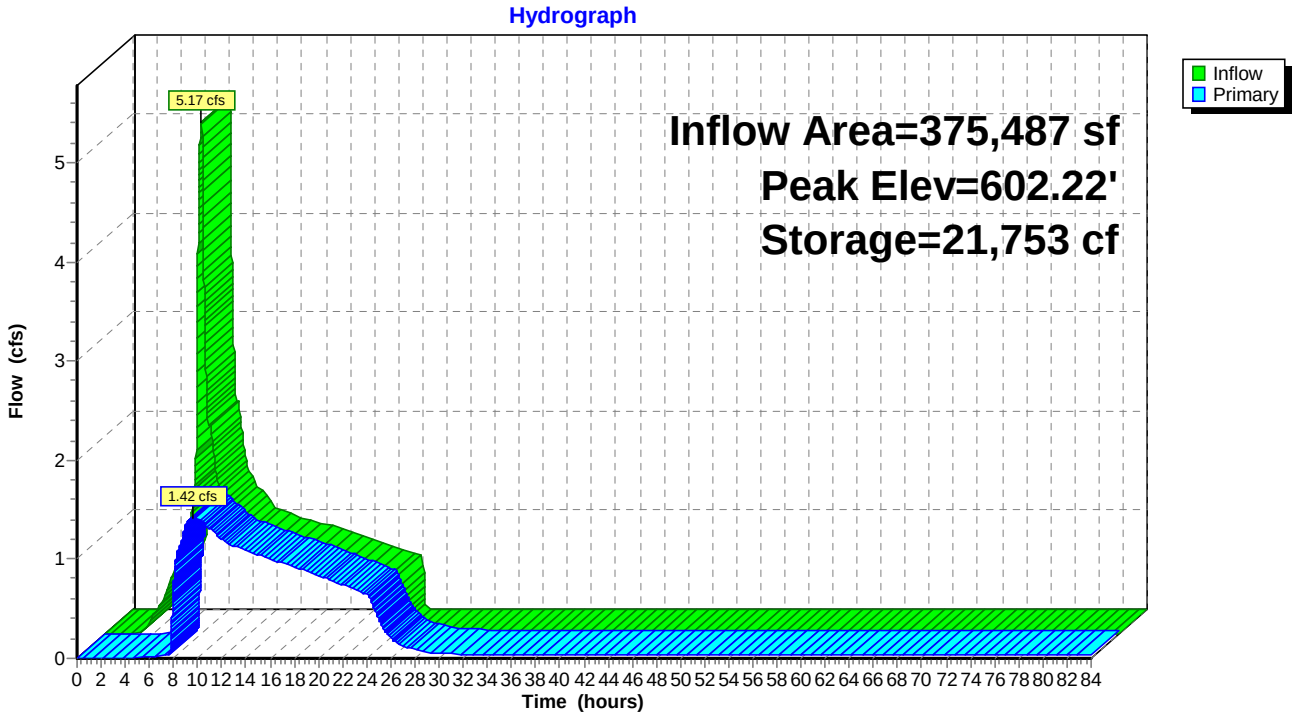
Volume	Invert	Avail.Storage	Storage Description
#1	597.24'	59,980 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
597.24	8,000	0.0	0	0
597.25	8,000	40.0	32	32
598.24	8,000	40.0	3,168	3,200
598.25	8,000	5.0	4	3,204
599.99	8,000	5.0	696	3,900
600.00	8,000	100.0	80	3,980
602.00	8,000	100.0	16,000	19,980
604.00	8,000	100.0	16,000	35,980
606.00	8,000	100.0	16,000	51,980
607.00	8,000	100.0	8,000	59,980

Device	Routing	Invert	Outlet Devices
#1	Primary	597.25'	**18.0" Round 18" Pipe** L= 50.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 597.25' / 597.00' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	597.25'	**0.8" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	600.75'	**6.5" Vert. Orifice #2** C= 0.600 Limited to weir flow at low heads
#4	Device 1	602.00'	**8.0" Vert. Orifice #3** C= 0.600 Limited to weir flow at low heads
#5	Device 1	606.00'	**18.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=1.42 cfs @ 9.54 hrs HW=602.22' (Free Discharge)
└**1=18" Pipe** (Passes 1.42 cfs of 19.28 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 0.04 cfs @ 10.70 fps)
  ├**3=Orifice #2** (Orifice Controls 1.22 cfs @ 5.28 fps)
  ├**4=Orifice #3** (Orifice Controls 0.16 cfs @ 1.60 fps)
  └**5=Overflow** (Controls 0.00 cfs)

Pond MH3: Control MH #3



Summary for Subcatchment 2E: Existing Conditions Site

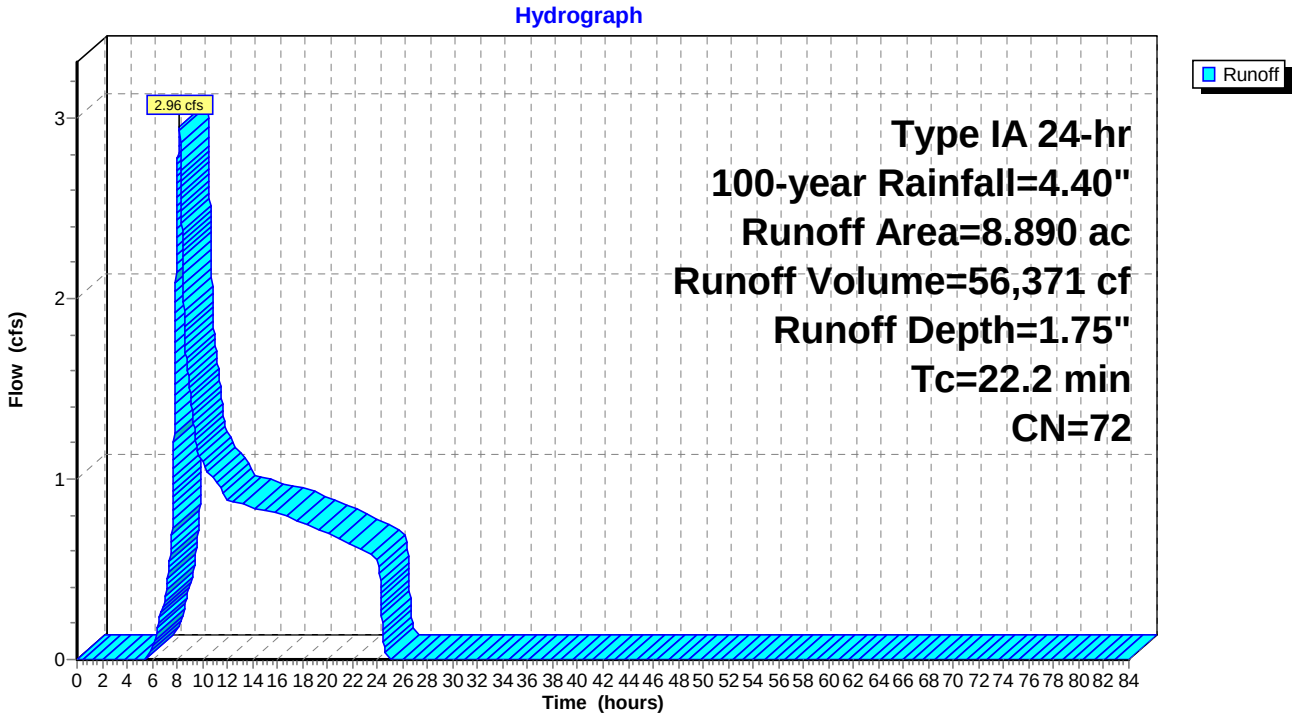
Runoff = 2.96 cfs @ 8.16 hrs, Volume= 56,371 cf, Depth= 1.75"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	8.890	72	City of Salem Pre-development, HSG C
	8.890	72	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
22.2					**Direct Entry, TR-55 Worksheet**

Subcatchment 2E: Existing Conditions Site



Summary for Subcatchment 2B: Developed Conditions Basin 2B

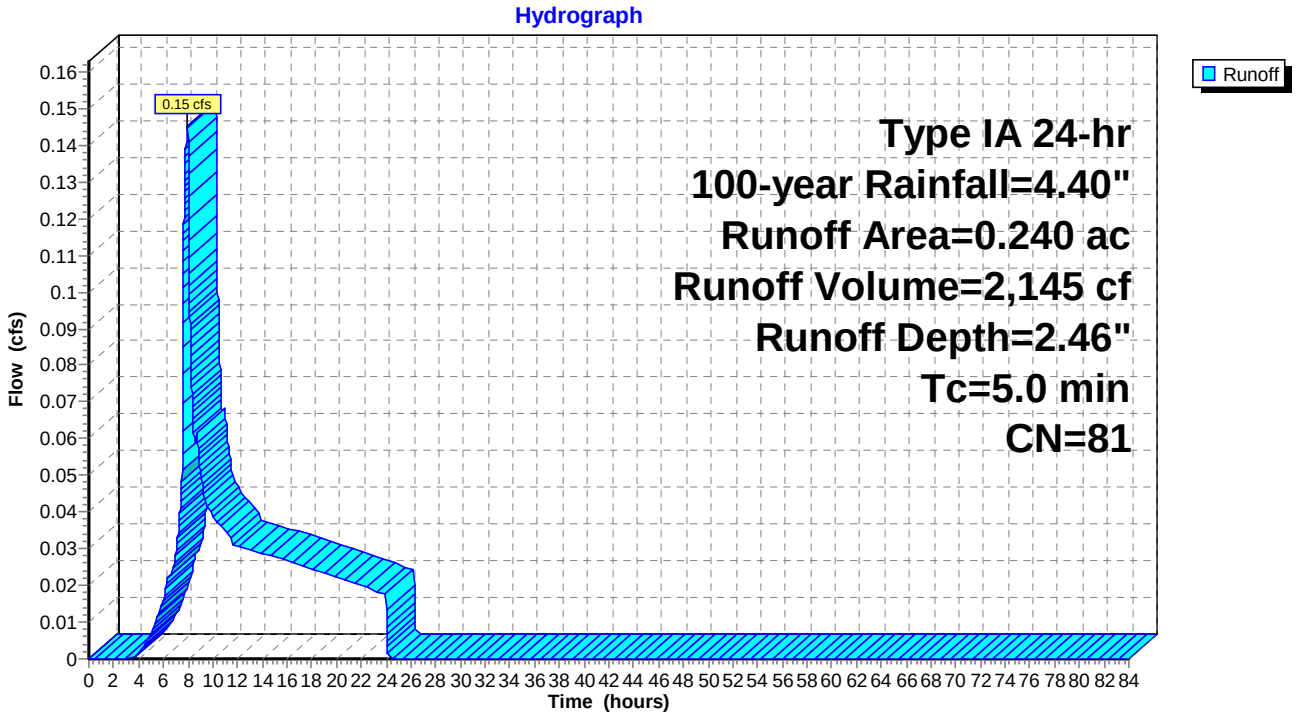
Runoff = 0.15 cfs @ 7.94 hrs, Volume= 2,145 cf, Depth= 2.46"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	0.070	98	Roofs,pavement, concrete, HSG C
	0.170	74	>75% Grass cover, Good, HSG C
	0.240	81	Weighted Average
	0.170	74	70.83% Pervious Area
	0.070	98	29.17% Impervious Area

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

Subcatchment 2B: Developed Conditions Basin 2B



Summary for Subcatchment 2: Developed Conditions Basin 2

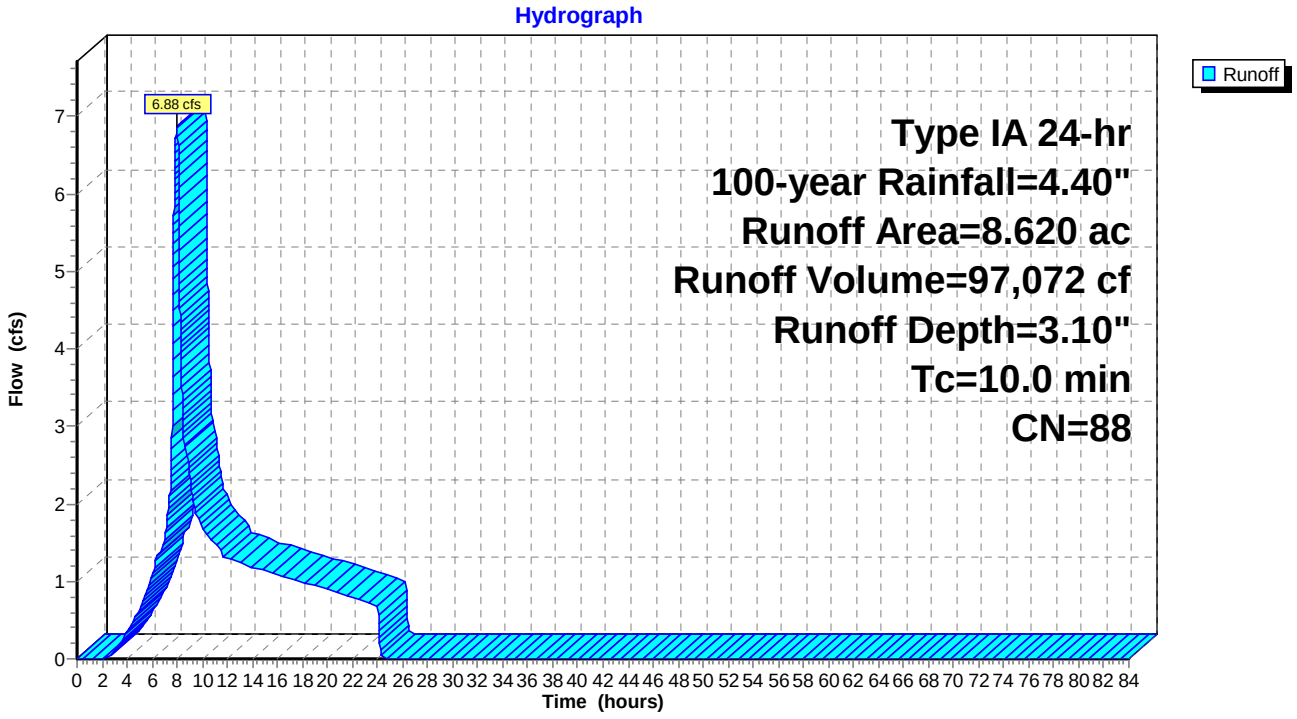
Runoff = 6.88 cfs @ 7.99 hrs, Volume= 97,072 cf, Depth= 3.10"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	5.170	98	Roofs,pavement, concrete, HSG C
	3.450	74	>75% Grass cover, Good, HSG C
	8.620	88	Weighted Average
	3.450	74	40.02% Pervious Area
	5.170	98	59.98% Impervious Area

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

Subcatchment 2: Developed Conditions Basin 2



Summary for Pond MH3: Control MH #3

Inflow Area = 375,487 sf, 59.98% Impervious, Inflow Depth = 3.10" for 100-year event
Inflow = 6.88 cfs @ 7.99 hrs, Volume= 97,072 cf
Outflow = 2.56 cfs @ 8.88 hrs, Volume= 92,397 cf, Atten= 63%, Lag= 53.3 min
Primary = 2.56 cfs @ 8.88 hrs, Volume= 92,397 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 602.74' @ 8.88 hrs Surf.Area= 8,000 sf Storage= 25,891 cf
Flood Elev= 606.00' Surf.Area= 8,000 sf Storage= 51,980 cf

Plug-Flow detention time= 359.3 min calculated for 92,386 cf (95% of inflow)
Center-of-Mass det. time= 326.8 min (1,069.7 - 742.9)

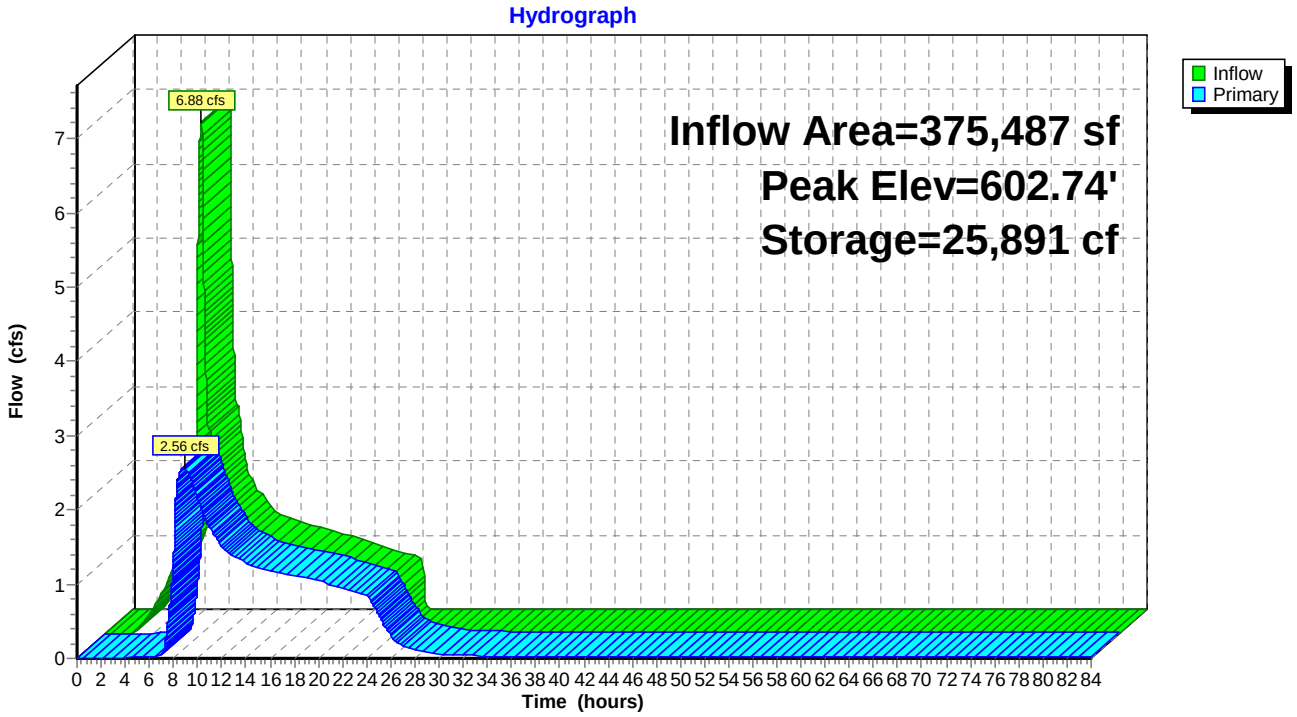
Volume	Invert	Avail.Storage	Storage Description
#1	597.24'	59,980 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
597.24	8,000	0.0	0	0
597.25	8,000	40.0	32	32
598.24	8,000	40.0	3,168	3,200
598.25	8,000	5.0	4	3,204
599.99	8,000	5.0	696	3,900
600.00	8,000	100.0	80	3,980
602.00	8,000	100.0	16,000	19,980
604.00	8,000	100.0	16,000	35,980
606.00	8,000	100.0	16,000	51,980
607.00	8,000	100.0	8,000	59,980

Device	Routing	Invert	Outlet Devices
#1	Primary	597.25'	**18.0" Round 18" Pipe** L= 50.0' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 597.25' / 597.00' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	597.25'	**0.8" Vert. Orifice #1** C= 0.600 Limited to weir flow at low heads
#3	Device 1	600.75'	**6.5" Vert. Orifice #2** C= 0.600 Limited to weir flow at low heads
#4	Device 1	602.00'	**8.0" Vert. Orifice #3** C= 0.600 Limited to weir flow at low heads
#5	Device 1	606.00'	**18.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=2.56 cfs @ 8.88 hrs HW=602.74' (Free Discharge)
└**1=18" Pipe** (Passes 2.56 cfs of 20.57 cfs potential flow)
  ├**2=Orifice #1** (Orifice Controls 0.04 cfs @ 11.25 fps)
  ├**3=Orifice #2** (Orifice Controls 1.45 cfs @ 6.31 fps)
  ├**4=Orifice #3** (Orifice Controls 1.07 cfs @ 3.07 fps)
  └**5=Overflow** (Controls 0.00 cfs)

Pond MH3: Control MH #3



Drawing is NOT to scale

Basin #1
46,700 cu. ft
Capacity

Basin #3
52,000 cu. ft.
Capacity

SEE SHEET 302 FOR
OFF-SITE UTILITY PLAN

MATCH LINE 1

MATCH LINE 2

RED OAK AV.

LONE OAK ROAD

ALPHA AVENUE

BETA COURT

CHI STREET

DELTA COURT

DELTA AVENUE

FOUR STREET

FIVE STREET

DEVON AVE

TRACT A

TRACT B

PUMP STATION

SEE SHEET 302 FOR
OFF-SITE UTILITY PLAN

N
W E
S
0' 50'
1 INCH
SCALE: 1" = 50'

MULTI/TECH
ENGINEERING SERVICES, INC.
1155 13th ST. S.E. SALEM, OR. 97302
PH. (503) 363-9227 FAX (503) 364-1260
www.mtengineering.net office@mtengineering.net

PRELIMINARY
UTILITY PLAN - ONSITE

DEVON ESTATES

6500x36 P301UT

Design: M.D.G.
Drawn: P.H.S.
Checked: B.M.G.
Date: NOV. 2017
Scale: AS SHOWN
As-Built: ---

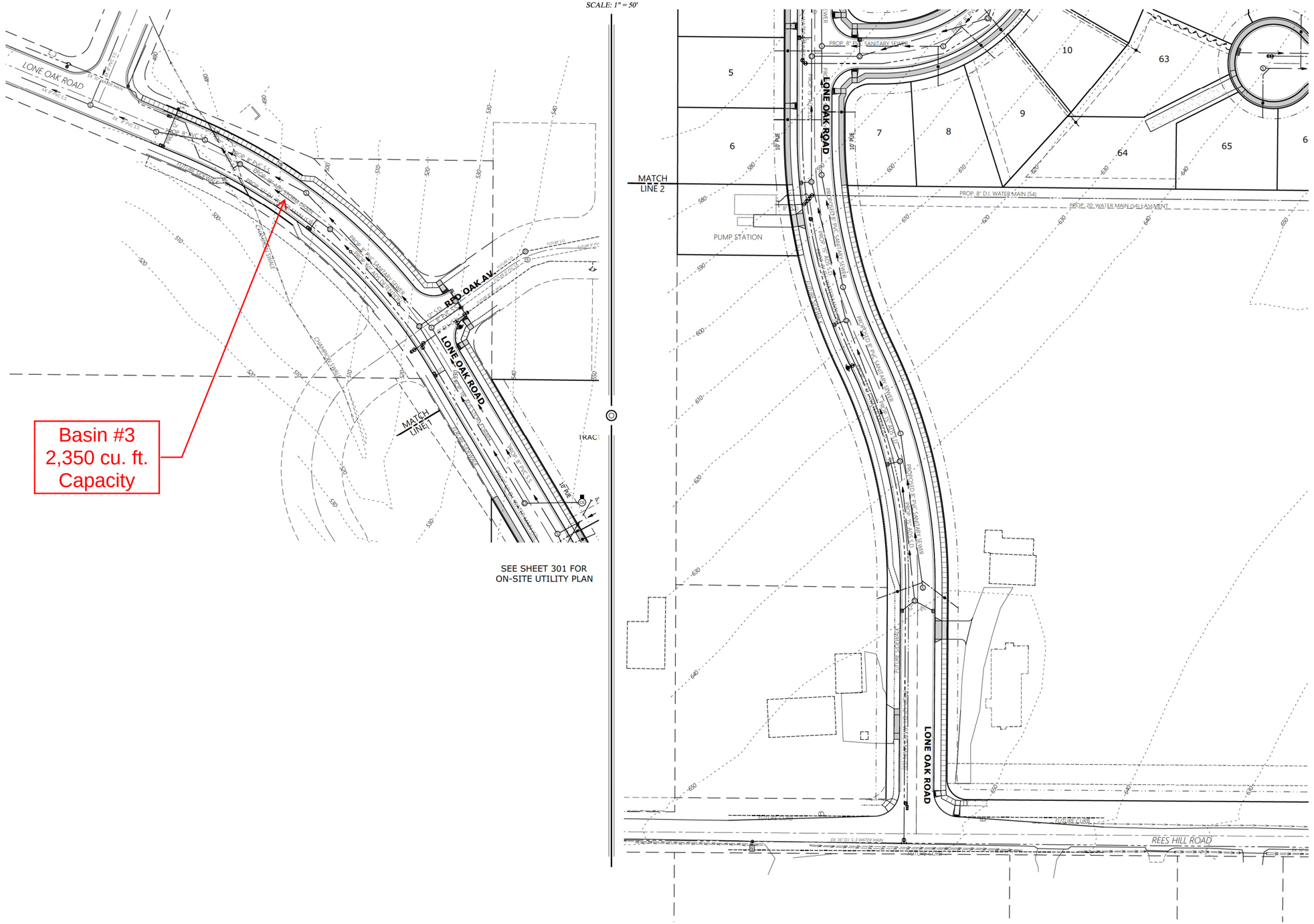
REGISTERED PROFESSIONAL ENGINEER
OREGON
JULY 14, 1978
MARK D. GRENZ

EXPIRES: 06-30-2021

JOB # 6502

P301

J:\6500-6599\6502-DevonAvenueSubdivision\Dwg\01 \6502sDE.dwg, P301UT, 1/19/2021 12:36:50 PM, PSouders



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www.mtengineering.net office@mtengineering.net

PRELIMINARY
UTILITY PLAN - OFFSITE

DEVON ESTATES

6502b2c P302UT

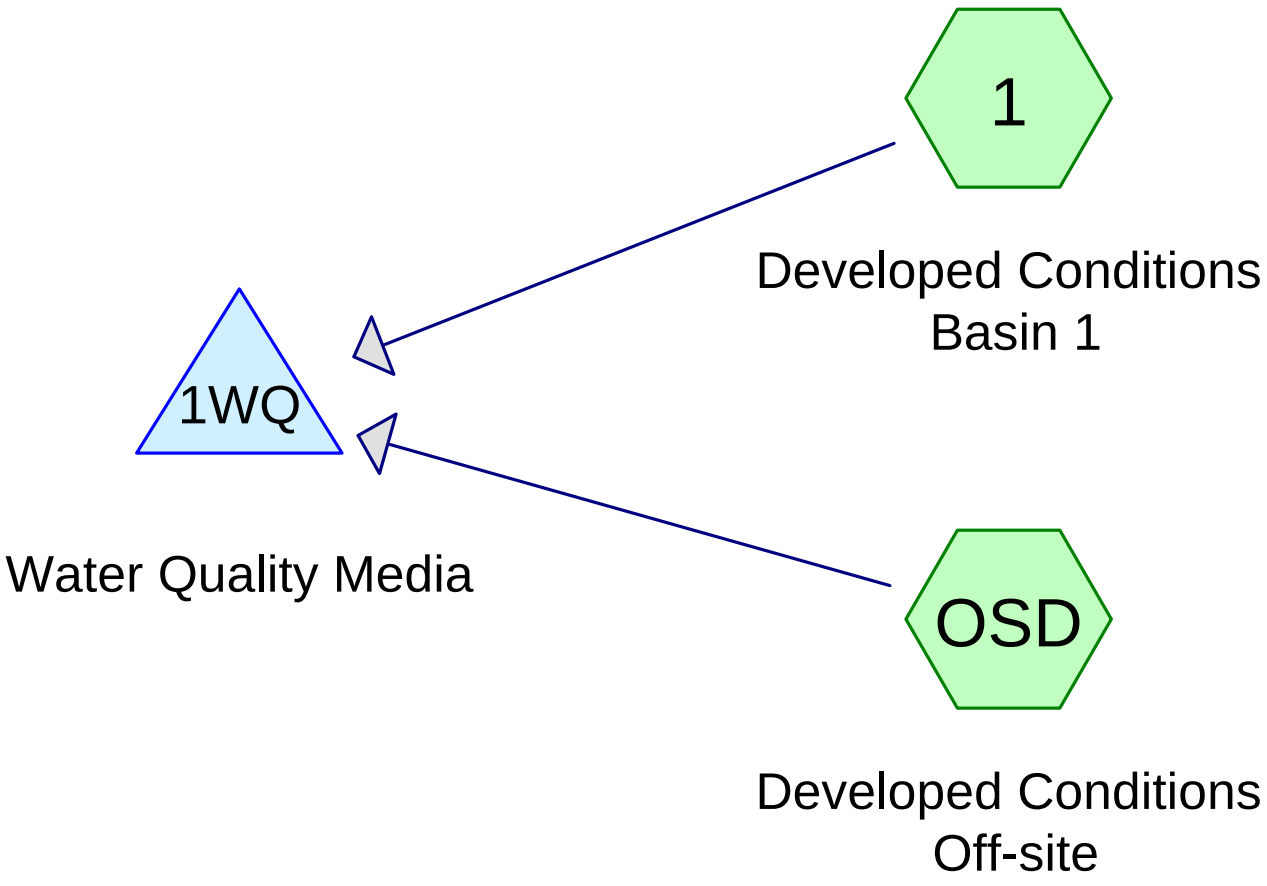
Design: M.D.G.
Drawn: P.H.S.
Checked: B.M.G.
Date: NOV. 2017
Scale: AS SHOWN
As-Built: ---

REGISTERED PROFESSIONAL ENGINEER
OREGON
JULY 14, 1978
MARK D. GRENZ
EXPIRES: 06-30-2021

JOB # 6502

P302

Appendix E



Subcat Reach Pond Link

Routing Diagram for 20210112 Devon Estates Phase 1 WQ
Prepared by Multitech Engineering Services, Inc., Printed 2/17/2021

Summary for Subcatchment 1: Developed Conditions Basin 1

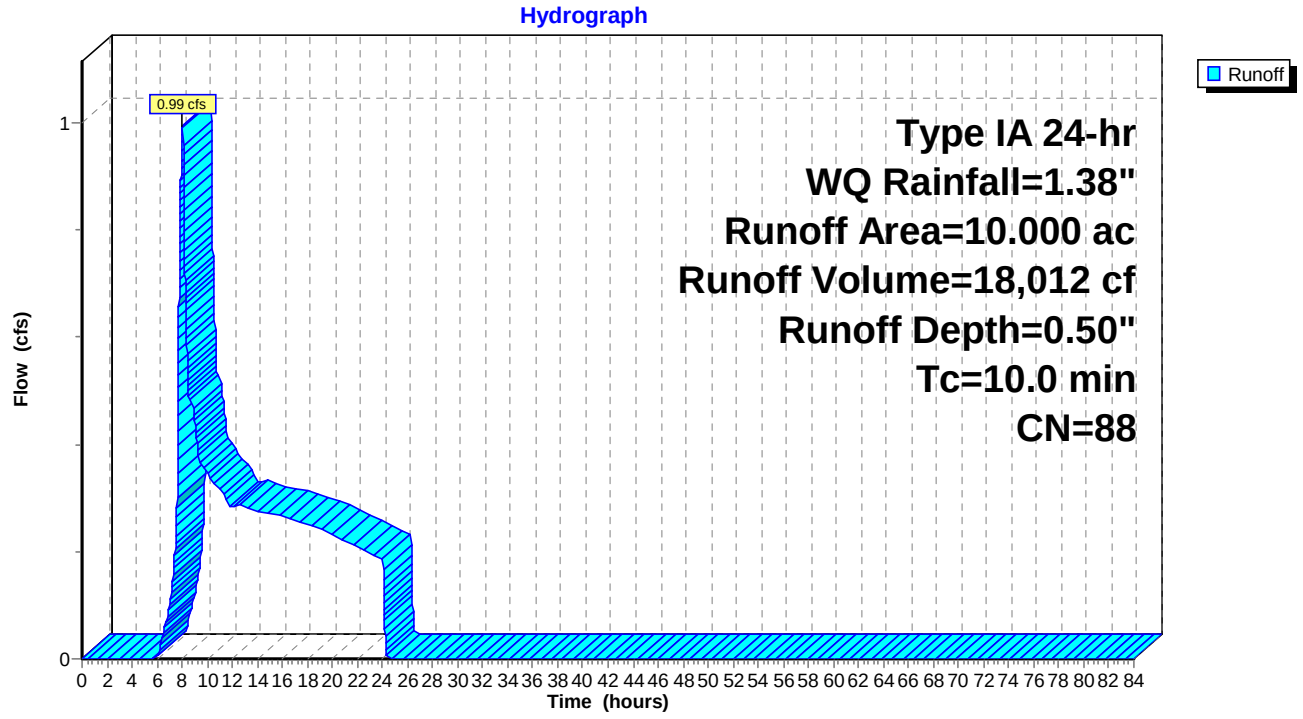
Runoff = 0.99 cfs @ 8.04 hrs, Volume= 18,012 cf, Depth= 0.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr WQ Rainfall=1.38"

	Area (ac)	CN	Description
*	6.000	98	Roofs,pavement, concrete, HSG C
	4.000	74	>75% Grass cover, Good, HSG C
	10.000	88	Weighted Average
	4.000	74	40.00% Pervious Area
	6.000	98	60.00% Impervious Area

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

Subcatchment 1: Developed Conditions Basin 1



Summary for Subcatchment OSD: Developed Conditions Off-site

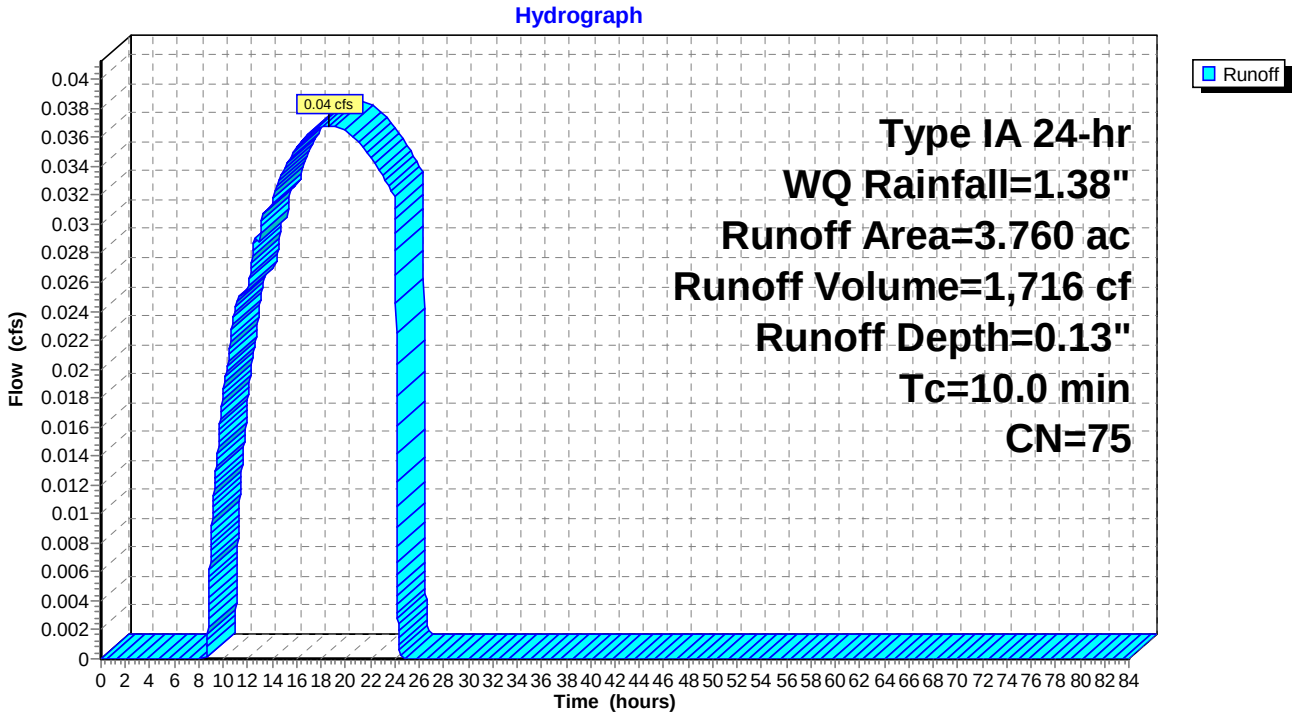
Runoff = 0.04 cfs @ 18.61 hrs, Volume= 1,716 cf, Depth= 0.13"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr WQ Rainfall=1.38"

	Area (ac)	CN	Description
*	3.260	72	City of Salem Pre-developed, HSG C
	0.500	98	Paved roads w/curbs & sewers, HSG C
	3.760	75	Weighted Average
	3.260	72	86.70% Pervious Area
	0.500	98	13.30% Impervious Area

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

Subcatchment OSD: Developed Conditions Off-site



Summary for Pond 1WQ: Water Quality Media

Inflow Area = 599,386 sf, 47.24% Impervious, Inflow Depth = 0.39" for WQ event
Inflow = 0.99 cfs @ 8.04 hrs, Volume= 19,728 cf
Outflow = 0.39 cfs @ 7.68 hrs, Volume= 19,728 cf, Atten= 60%, Lag= 0.0 min
Primary = 0.39 cfs @ 7.68 hrs, Volume= 19,728 cf
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 542.15' @ 9.31 hrs Surf.Area= 8,500 sf Storage= 1,329 cf
Flood Elev= 562.00' Surf.Area= 8,500 sf Storage= 51,085 cf

Plug-Flow detention time= 19.9 min calculated for 19,726 cf (100% of inflow)
Center-of-Mass det. time= 19.9 min (888.6 - 868.7)

Volume	Invert	Avail.Storage	Storage Description
#1	541.99'	51,085 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

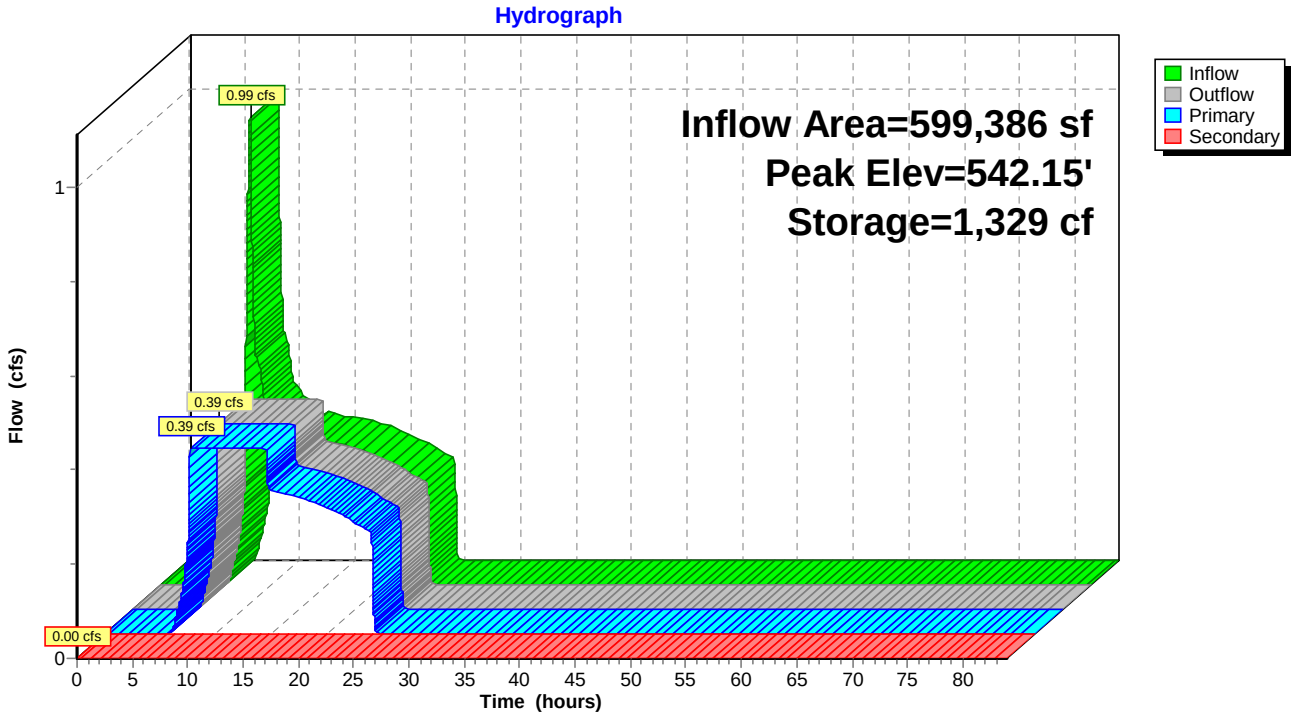
Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
541.99	8,500	0.0	0	0
542.00	8,500	100.0	85	85
544.00	8,500	100.0	17,000	17,085
546.00	8,500	100.0	17,000	34,085
547.00	8,500	100.0	8,500	42,585
548.00	8,500	100.0	8,500	51,085

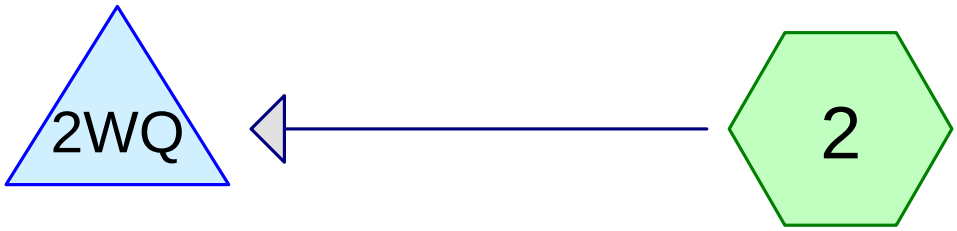
Device	Routing	Invert	Outlet Devices
#1	Primary	541.99'	**2.000 in/hr Exfiltration over Surface area**
#2	Secondary	543.00'	**24.0" Horiz. Beehive Overflow** C= 0.600 Limited to weir flow at low heads
#3	Secondary	547.00'	**15.0" Horiz. Emergency Overflow CB** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.39 cfs @ 7.68 hrs HW=542.00' (Free Discharge)
└**1=Exfiltration** (Exfiltration Controls 0.39 cfs)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=541.99' (Free Discharge)
├**2=Beehive Overflow** (Controls 0.00 cfs)
└**3=Emergency Overflow CB** (Controls 0.00 cfs)

Pond 1WQ: Water Quality Media





Water Quality Media

Developed Conditions
Basin 2

Subcat Reach Pond Link

Routing Diagram for 20210112 Devon Estates Phase 2 WQ
Prepared by Multitech Engineering Services, Inc., Printed 2/17/2021

Summary for Subcatchment 2: Developed Conditions Basin 2

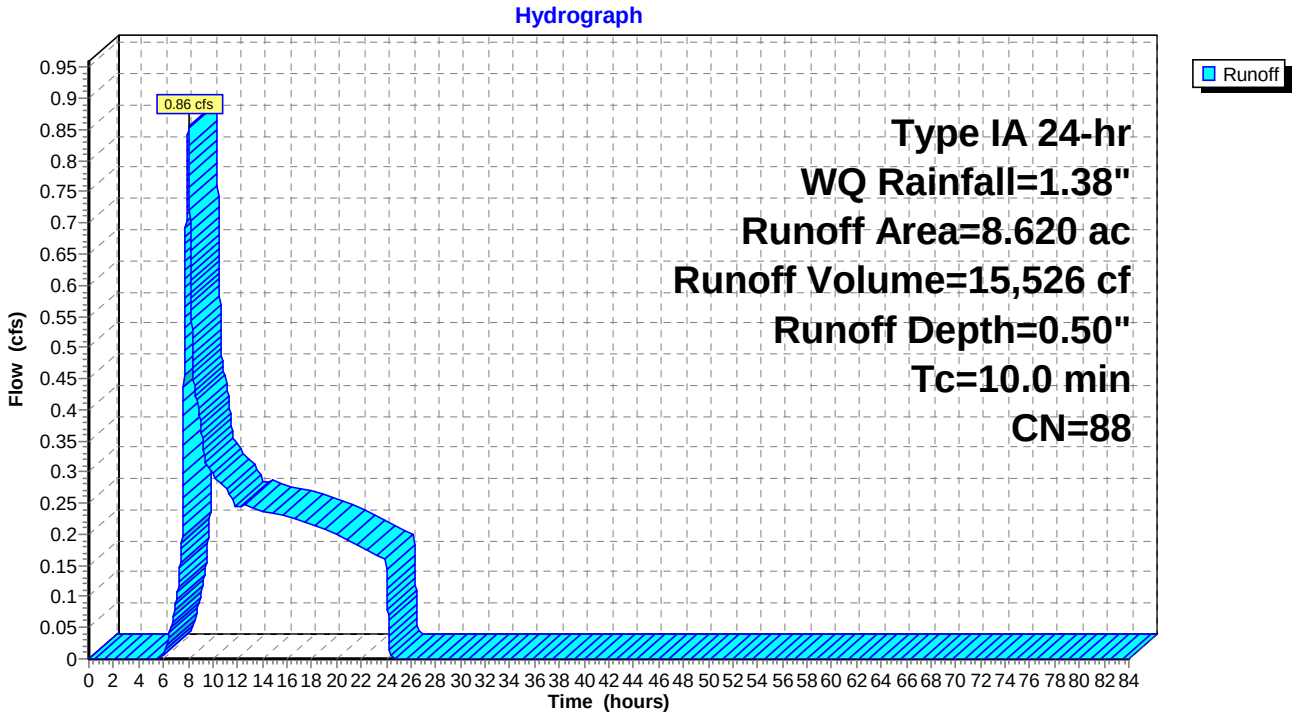
Runoff = 0.86 cfs @ 8.04 hrs, Volume= 15,526 cf, Depth= 0.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Type IA 24-hr WQ Rainfall=1.38"

	Area (ac)	CN	Description
*	5.170	98	Roofs,pavement, concrete, HSG C
	3.450	74	>75% Grass cover, Good, HSG C
	8.620	88	Weighted Average
	3.450	74	40.02% Pervious Area
	5.170	98	59.98% Impervious Area

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

Subcatchment 2: Developed Conditions Basin 2



Summary for Pond 2WQ: Water Quality Media

Inflow Area = 375,487 sf, 59.98% Impervious, Inflow Depth = 0.50" for WQ event
Inflow = 0.86 cfs @ 8.04 hrs, Volume= 15,526 cf
Outflow = 0.37 cfs @ 7.70 hrs, Volume= 15,526 cf, Atten= 57%, Lag= 0.0 min
Primary = 0.37 cfs @ 7.70 hrs, Volume= 15,526 cf
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0 cf

Routing by Stor-Ind method, Time Span= 0.00-84.00 hrs, dt= 0.01 hrs
Peak Elev= 600.11' @ 9.05 hrs Surf.Area= 8,000 sf Storage= 969 cf
Flood Elev= 606.00' Surf.Area= 8,000 sf Storage= 48,080 cf

Plug-Flow detention time= 12.9 min calculated for 15,524 cf (100% of inflow)
Center-of-Mass det. time= 12.9 min (865.9 - 853.0)

Volume	Invert	Avail.Storage	Storage Description
#1	599.99'	56,080 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

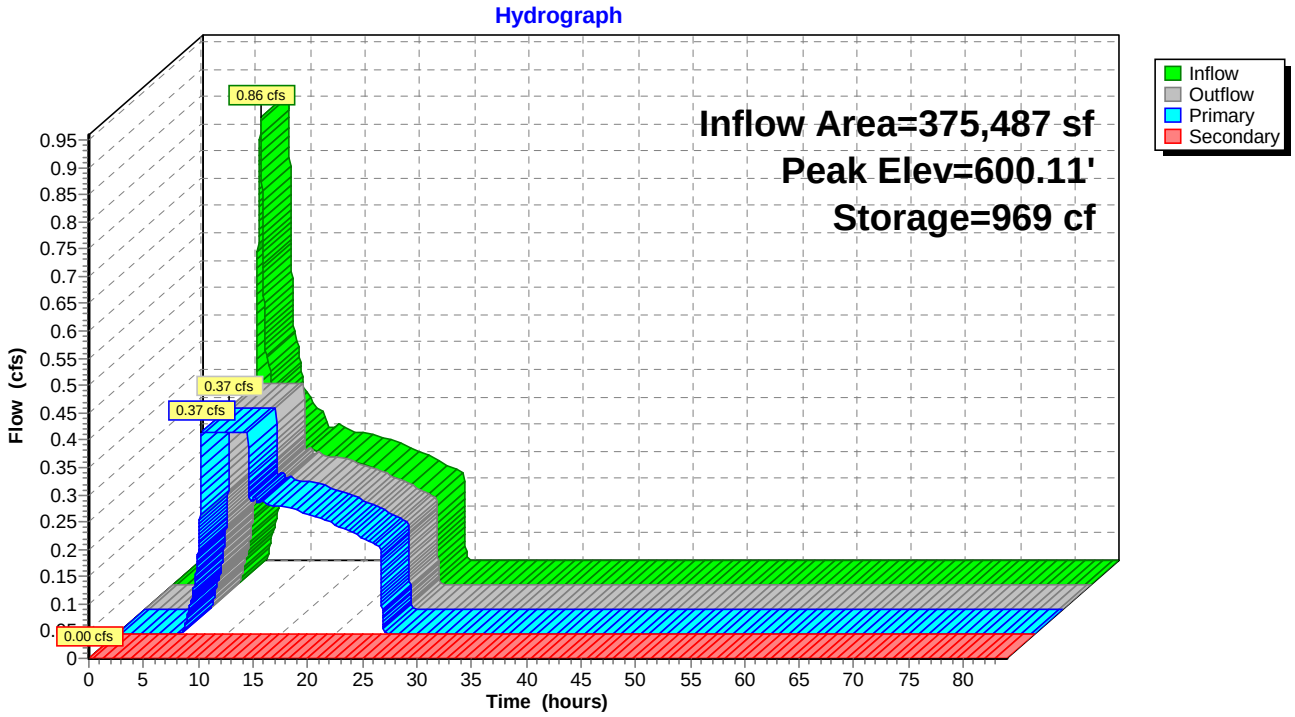
Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
599.99	8,000	0.0	0	0
600.00	8,000	100.0	80	80
602.00	8,000	100.0	16,000	16,080
604.00	8,000	100.0	16,000	32,080
606.00	8,000	100.0	16,000	48,080
607.00	8,000	100.0	8,000	56,080

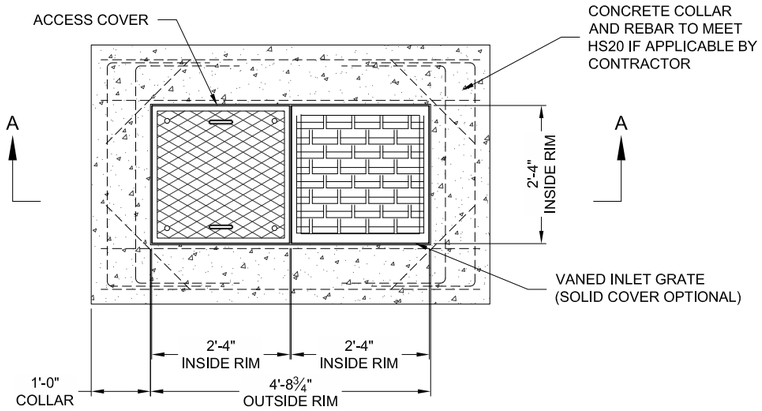
Device	Routing	Invert	Outlet Devices
#1	Primary	599.99'	**2.000 in/hr Exfiltration over Surface area**
#2	Secondary	600.25'	**24.0" Horiz. Beehive Overflow** C= 0.600 Limited to weir flow at low heads
#3	Secondary	606.00'	**15.0" Horiz. Emergency Overflow CB** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.37 cfs @ 7.70 hrs HW=600.00' (Free Discharge)
└**1=Exfiltration** (Exfiltration Controls 0.37 cfs)

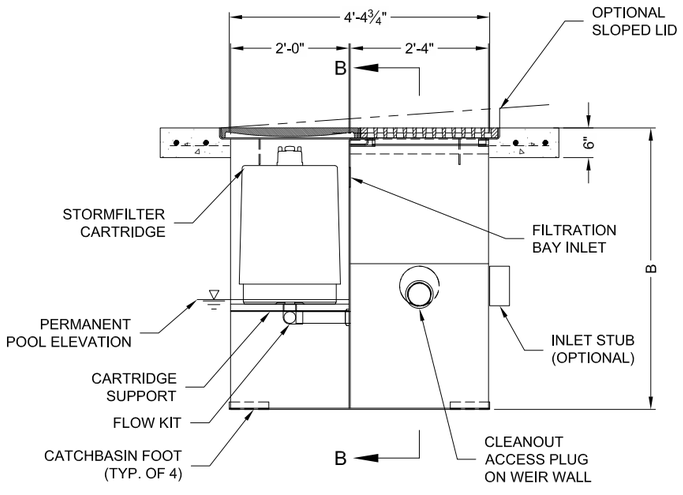
Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=599.99' (Free Discharge)
├**2=Beehive Overflow** (Controls 0.00 cfs)
└**3=Emergency Overflow CB** (Controls 0.00 cfs)

Pond 2WQ: Water Quality Media

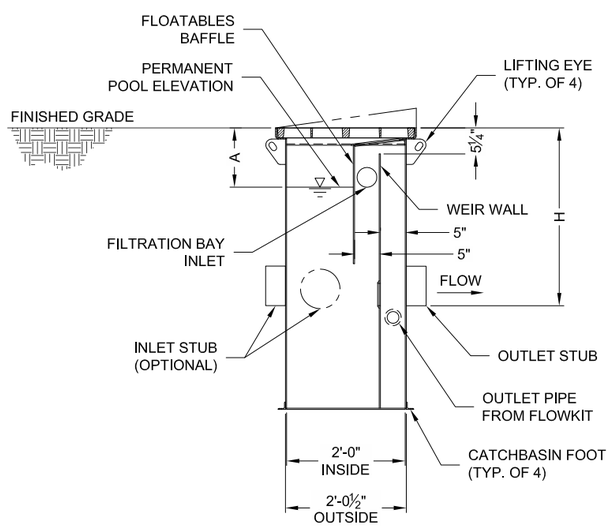




PLAN VIEW



SECTION A-A



SECTION B-B

STORMFILTER STEEL CATCHBASIN DESIGN NOTES

STORMFILTER TREATMENT CAPACITY IS A FUNCTION OF THE CARTRIDGE SELECTION AND THE NUMBER OF CARTRIDGES. 1 CARTRIDGE CATCHBASIN HAS A MAXIMUM OF ONE CARTRIDGE. SYSTEM IS SHOWN WITH A 27" CARTRIDGE, AND IS ALSO AVAILABLE WITH AN 18" CARTRIDGE. STORMFILTER CATCHBASIN CONFIGURATIONS ARE AVAILABLE WITH A DRY INLET BAY FOR VECTOR CONTROL.
PEAK HYDRAULIC CAPACITY PER TABLE BELOW. IF THE SITE CONDITIONS EXCEED PEAK HYDRAULIC CAPACITY, AN UPSTREAM BYPASS STRUCTURE IS REQUIRED.

CARTRIDGE SELECTION

CARTRIDGE HEIGHT	27"			18"			18" DEEP		
RECOMMENDED HYDRAULIC DROP (H)	3.05'			2.3'			3.3'		
SPECIFIC FLOW RATE (gpm/sf)	2 gpm/sf	1.67* gpm/sf	1 gpm/sf	2 gpm/sf	1.67* gpm/sf	1 gpm/sf	2 gpm/sf	1.67* gpm/sf	1 gpm/sf
CARTRIDGE FLOW RATE (gpm)	22.5	18.79	11.25	15	12.53	7.5	15	12.53	7.5
PEAK HYDRAULIC CAPACITY	1.0			1.0			1.8		
INLET PERMANENT POOL LEVEL (A)	1'-0"			1'-0"			2'-0"		
OVERALL STRUCTURE HEIGHT (B)	4'-9"			3'-9"			4'-9"		

* 1.67 gpm/sf SPECIFIC FLOW RATE IS APPROVED WITH PHOSPHOSORB® (PSORB) MEDIA ONLY

GENERAL NOTES
1. CONTECH TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.
2. FOR SITE SPECIFIC DRAWINGS WITH DETAILED STORMFILTER CATCHBASIN STRUCTURE DIMENSIONS AND WEIGHTS, PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS LLC REPRESENTATIVE. www.contechES.com
3. STORMFILTER CATCHBASIN WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING.
4. INLET SHOULD NOT BE LOWER THAN OUTLET. INLET (IF APPLICABLE) AND OUTLET PIPING TO BE SPECIFIED BY ENGINEER AND PROVIDED BY CONTRACTOR.
5. MANUFACTURER TO APPLY A SURFACE BEAD WELD IN THE SHAPE OF THE LETTER "O" ABOVE THE OUTLET PIPE STUB ON THE EXTERIOR SURFACE OF THE STEEL SFCB.
6. STORMFILTER CATCHBASIN EQUIPPED WITH 4 INCH (APPROXIMATE) LONG STUBS FOR INLET (IF APPLICABLE) AND OUTLET PIPING. STANDARD OUTLET STUB IS 8 INCHES IN DIAMETER. MAXIMUM OUTLET STUB IS 15 INCHES IN DIAMETER. CONNECTION TO COLLECTION PIPING CAN BE MADE USING FLEXIBLE COUPLING BY CONTRACTOR.
7. STEEL STRUCTURE TO BE MANUFACTURED OF 1/4 INCH STEEL PLATE. CASTINGS SHALL MEET AASHTO M306 LOAD RATING. TO MEET HS20 LOAD RATING ON STRUCTURE, A CONCRETE COLLAR IS REQUIRED. WHEN REQUIRED, CONCRETE COLLAR WITH #4 REINFORCING BARS TO BE PROVIDED BY CONTRACTOR.
8. FILTER CARTRIDGES SHALL BE MEDIA-FILLED, PASSIVE, SIPHON ACTUATED, RADIAL FLOW, AND SELF CLEANING. RADIAL MEDIA DEPTH SHALL BE 7-INCHES. FILTER MEDIA CONTACT TIME SHALL BE AT LEAST 38 SECONDS.
9. SPECIFIC FLOW RATE IS EQUAL TO THE FILTER TREATMENT CAPACITY (gpm) DIVIDED BY THE FILTER CONTACT SURFACE AREA (sq ft).

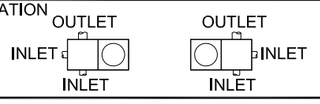
INSTALLATION NOTES
A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE CATCHBASIN (LIFTING CLUTCHES PROVIDED).
C. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT CARTRIDGES FROM CONSTRUCTION-RELATED EROSION RUNOFF.

1-CARTRIDGE CATCHBASIN STORMFILTER DATA

STRUCTURE ID	XXX
WATER QUALITY FLOW RATE (cfs)	X.XX
PEAK FLOW RATE (<1 cfs)	X.XX
RETURN PERIOD OF PEAK FLOW (yrs)	XXX
CARTRIDGE HEIGHT (27", 18", 18" DEEP)	XX
CARTRIDGE FLOW RATE (gpm)	XX
MEDIA TYPE (PERLITE, ZPG, PSORB)	XXXXX
RIM ELEVATION	XXX.XX'

PIPE DATA:	I.E.	DIAMETER
INLET STUB	XXX.XX'	XX"
OUTLET STUB	XXX.XX'	XX"

CONFIGURATION



SLOPED LID	YES\NO
SOLID COVER	YES\NO

NOTES/SPECIAL REQUIREMENTS:

The Stormwater Management StormFilter®

CONTECH® ENGINEERED SOLUTIONS LLC
www.contechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-526-3999 513-645-7000 513-645-7993 FAX

1 CARTRIDGE CATCHBASIN STORMFILTER STANDARD DETAIL