EXPIRES: 06/30/2027

Stormwater Calculations for

Santiam Christian Elementary School
For
Santiam Christian School

Devco Job No. 23449

March 2026

Table of Contents

Stormwater Narrative – Santiam Christian Elementary School

Project Overview and Description

The project site is located within the Santiam Christian School campus in Adair Village, Oregon, bounded by Vandenberg Avenue, Marcus Harris Avenue, and Arnold Avenue. The stormwater basin analyzed in this report encompasses approximately 52 acres. Proposed improvements include a new school building, associated private parking and access drives, extensions of Marcus Harris Avenue and Vandenberg Avenue, and a new combination dry pond situated along the eastern boundary of the property. The pond has been sized to accommodate future development of synthetic turf sports fields.

The site has been split into 3 basins: Basin A, Basin B, and Basin C. These basins are shown on exhibit STRM1 and are described below:

- **Basin A (4.78 Acres)**
  This basin includes the area that is proposed to be the new school building and street extensions. Existing improvements such as buildings, roads, and walkways make up this basin but will be demolished for the proposed improvements to be constructed. Stormwater from this basin is currently collected and piped to the west to be conveyed offsite. Proposed conditions will capture stormwater to be conveyed to the east into the proposed dry pond.

- **Basin B (22.93 Acres)**
  This basin includes existing onsite and offsite areas that surface flow to the east and into the existing Ryals Avenue roadside ditch.

- **Basin C (24.07 Acres)**
  This is the site development area that is used to determine historic flow for allowable discharge rates from the proposed site. Existing contours demonstrate that this area naturally conveys stormwater to the east and into the existing Ryals Avenue roadside ditch. A portion of the proposed school site is included in this basin as well as future synthetic turf fields.

Table 1: Stormwater Basins Information

Basin Name	Total Area (acres)	Impervious Area (acres)	Hydrologic Soil Group	Time of Concentration (min)
Basin A (Existing)	4.78	2.55	D	5
Basin B (Existing)	22.93	0	C/D	29.7
Basin C (Historic)	24.07	0	D	55.4
Basin C (Proposed)	24.07	9.73	D	5

Design Methodology

The manual used for the project is the City of Adair Village Public Infrastructure Design Standards manual (2009) and the Benton County Stormwater Design Manual (2024). The Santa Barbara Urban Hydrograph (SBUH) method, ran using HydroCAD software, is used to generate runoff hydrographs for this project. Impervious areas were determined by taking direct quantities from the proposed development. The site consists of hydrologic group D soils, so infiltration is not considered for this design.

Water Quality and Quantity

Storm Water Quantity (Detention)
In accordance with the City of Adair Village Design Standards, a detention facility is required when a development creates an increase in impervious area. This development includes 6.10 acres of proposed impervious area, and accounts for 6.18 acres of future impervious area. A dry pond is proposed to treat both stormwater quality and quantity for this project.

Basin B and Basin C both historically drain towards the Ryals Avenue roadside ditch, as previously identified earlier in this report. The proposed dry pond is designed to release incoming flows at the historic rate of Basin C to ensure that the development does not create an increase in peak flow towards Ryals. Since Basin A stormwater is currently discharged to the west, away from Ryals Avenue, the pond is oversized to account for this additional stormwater that would not historically drain to the east.

A flow control manhole containing a standpipe and orifice regulates the release rate, with the orifice sized to limit the developed Basin A plus Basin C 25-year storm release to the historic Basin C 25-year storm rate per City of Adair Standards. An overflow pipe provides capacity to bypass the 100-year storm. The resulting discharge rates meet the release requirements specified in the City of Adair Standards, as shown in Table 2.

Table 2: Historic and Developed Discharge Rates

Storm Event	Basin A Flow Rate (cfs)	Basin B Flow Rate (cfs)	Basin C Historic Flow Rate (cfs)	Historic Flow to Ryals; Basic B+C (cfs)	Basin C Proposed Flow Rate (cfs)	Detention Pond Outflow (cfs)	Combined Outflow to Ryals Ditch; Basin B+C (cfs)
½ 2-Year	0.84	0.31	0.41	0.72	3.35	0.40	0.71
25-Year	4.24	8.76	8.15	16.91	19.86	8.10	16.86
100-Year	5.38	12.51	11.34	23.85	25.60	11.94	24.45

Stormwater Quality
Benton County standards establish the stormwater quality requirements for this project. These standards require treatment of runoff from one-half of the 2-year, 24-hour storm event when more than 10,890 square feet of new impervious surface are created. To meet this requirement, a dry pond is proposed to provide water quality treatment.

The dry pond is designed to temporarily store stormwater during rainfall events and release the water quality design volume through an underdrain system after filtration through the underlying soil media. Runoff volumes exceeding the water quality storm are conveyed through an outlet control structure.

Dry ponds are recognized as approved stormwater best management practices (BMPs) in the stormwater design manuals of both the City of Albany and the City of Corvallis. Given Adair Village's proximity to these jurisdictions, the use of a dry pond is considered an appropriate and consistent treatment approach. The proposed facility is sized to fully route the water quality storm volume through the underdrain system, as demonstrated in the HydroCAD model results.

HATCH LEGEND

BASIN A	EXISTING BASIN NOT DRAINING TOWARDS RYALS AVENUE (4.78 ACRES)
BASIN B	EXISTING BASIN DRAINING TOWARDS RYALS AVENUE TO REMAIN (22.93 ACRES)
BASIN C	EXISTING BASIN DRAINING TOWARDS RYALS AVENUE TO BE DEVELOPED (24.07 ACRES)

EAGLES

ESTRM

300' AT 1.0%

956' AT 5.4%

300' AT 6.0%

600' AT 1.83%

NW RYALS AVENUE

UNION PACIFIC RAILROAD

SCALE: 1" = 200'

0 50 100 200 400

SCALE IN FEET

PRELIMINARY

DRAWING STATUS: DATE:
- ☐ PRELIMINARY
- ☐ SUBMITTED
- ☐ BID SET
- ☐ PERMIT SET
- ☐ CONST. SET

DEVCO engineering Inc
245 NE CONIFER, P.O. BOX 1211
CORVALLIS, OR. 97339
(541) 757-1661
WWW.DEVCOENGINEERING.COM

PROJECT: SANTIAM CHRISTIAN ELEMENTARY SCHOOL
PROJECT LOCATION: ADAIR VILLAGE, OR
CLIENT: SANTIAM CHRISTIAN SCHOOL

JOB NO. 23449
DRAWN BY: DEVCO
SHEET TITLE: STORM BASINS
DRAWING: STRM1

FILE: I:\CIVIL3D Projects\23449\Reports\Stormwater\Figures\23449STRM_FIG_EX_LARGE_BASIN.dwg [STRM1] 3/3/2026 11:28 AM – Trevor

BASIN A - DEVELOPED
2.55 ACRES NEW IMPERVIOUS
2.23 ACRES PERVIOUS

NOTE:
IMPERVIOUS AREAS INCLUDE
FUTURE SYNTHETIC TURF
SPORTS FIELDS AS SHOWN

BASIN C - DEVELOPED
3.55 ACRES NEW IMPERVIOUS
6.18 ACRES FUTURE IMPERVIOUS
14.34 ACRES PERVIOUS

PROPOSED COMBINATION
DRY POND

DISCHARGE TO
FLOW SPREADER

EAGLES

STRM

FSTR

SCALE: 1" = 150'

0 37.5 75 150 300

SCALE IN FEET

DRAWING STATUS:
- ☐ PRELIMINARY
- ☐ SUBMITTED
- ☐ BID SET
- ☐ PERMIT SET
- ☐ CONST. SET

DATE:

DEVCO engineering Inc
245 NE CONIFER, P.O. BOX 1211
CORVALLIS, OR. 97339
(541) 757-8944
WWW.DEVCOENGINEERING.COM

PROJECT: SC ELEMENTARY

PROJECT LOCATION: ADAIR VILLAGE, OR

CLIENT: SANTIAM CHRISTIAN SCHOOL

JOB NO. 23449

DRAWN BY: DEVCO

SHEET TITLE: DEVELOPED

DRAWING: STRM2

FILE: I:\CIVIL3D Projects\23449\Reports\Stormwater\Figures\23449STRM_FIG_DEV.dwg [STRM2] 3/3/2026 11:27 AM – Trevor

EXISTING AND HISTORIC CONDITIONS

Subcat Reach Pond Link

Routing Diagram for 23449_Stormwater
Prepared by DEVCO Engineering, Inc, Printed 3/3/2026
HydroCAD® 10.20-8a s/n 03944 © 2025 HydroCAD Software Solutions LLC

Rainfall Events Listing (selected events)

Event#	Event Name	Storm Type	Curve	Mode	Duration (hours)	B/B	Depth (inches)	AMC
1	25-Year	Type IA 24-hr		Default	24.00	1	4.70	2
2	100-Year	Type IA 24-hr		Default	24.00	1	5.70	2

Area Listing (selected nodes)

Area (sq-ft)	CN	Description (subcatchment-numbers)
998,831	77	>75% Grass cover, Good, HSG C/D (2S)
1,048,489	80	>75% Grass cover, Good, HSG D (3S)
2,047,320	**79**	**TOTAL AREA**

Soil Listing (selected nodes)

Area (sq-ft)	Soil Group	Subcatchment Numbers
0	HSG A	
0	HSG B	
998,831	HSG C	2S
1,048,489	HSG D	3S
0	Other	
2,047,320		**TOTAL AREA**

Ground Covers (selected nodes)

HSG-A (sq-ft)	HSG-B (sq-ft)	HSG-C (sq-ft)	HSG-D (sq-ft)	Other (sq-ft)	Total (sq-ft)	Ground Cover	Su Nu
0	0	998,831	1,048,489	0	2,047,320	>75% Grass cover, Good	
0	**0**	**998,831**	**1,048,489**	**0**	**2,047,320**	**TOTAL AREA**	

Time span=0.00-30.00 hrs, dt=0.05 hrs, 601 points
Runoff by SBUH method, Split Pervious/Imperv.
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Basin B - Existing Runoff Area=22.930 ac 0.00% Impervious Runoff Depth=2.37"
Flow Length=900' Tc=29.7 min CN=77/0 Runoff=8.76 cfs 197,609 cf

Subcatchment3S: Basin C - Historic Runoff Area=24.070 ac 0.00% Impervious Runoff Depth>2.63"
Flow Length=1,256' Tc=55.4 min CN=80/0 Runoff=8.15 cfs 230,032 cf

Total Runoff Area = 2,047,320 sf Runoff Volume = 427,640 cf Average Runoff Depth = 2.51"
100.00% Pervious = 2,047,320 sf 0.00% Impervious = 0 sf

Summary for Subcatchment 2S: Basin B - Existing

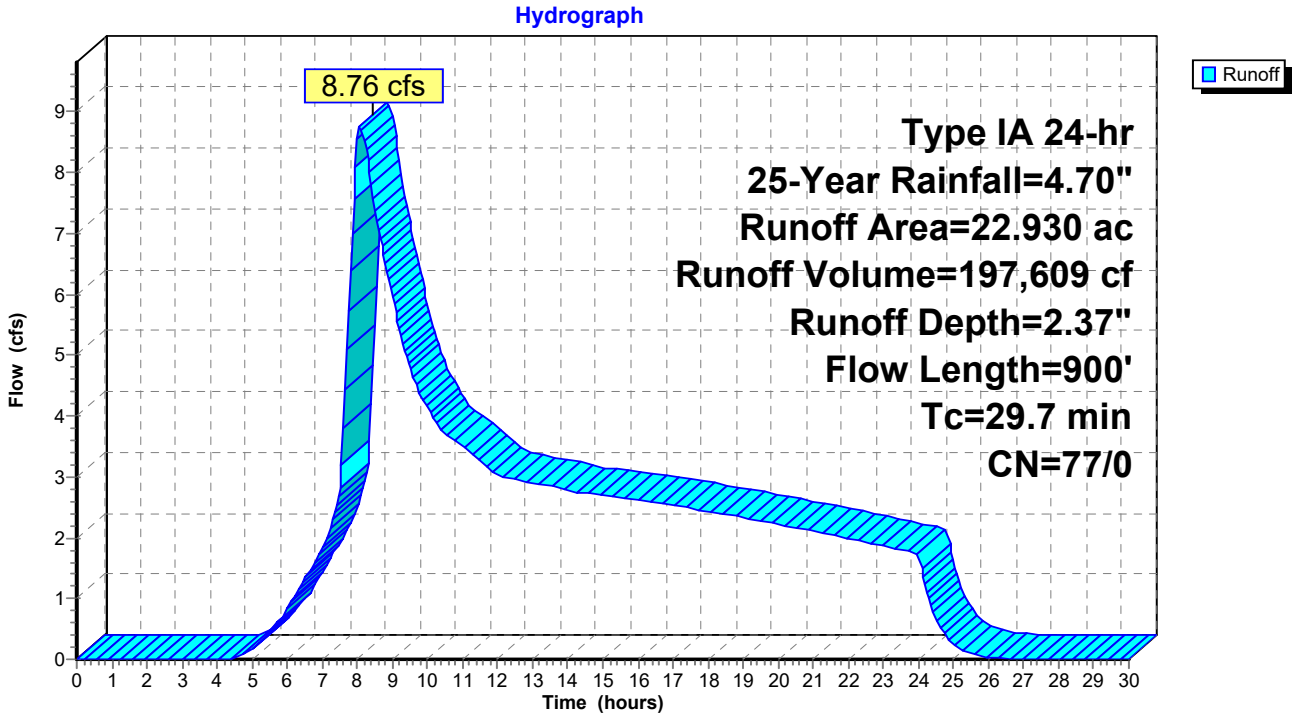
Runoff = 8.76 cfs @ 8.07 hrs, Volume= 197,609 cf, Depth= 2.37"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 25-Year Rainfall=4.70"

	Area (ac)	CN	Description
*	22.930	77	>75% Grass cover, Good, HSG C/D
	22.930	77	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
24.8	300	0.0600	0.20		**Sheet Flow,** Grass: Dense n= 0.240 P2= 2.55"
4.9	600	0.0183	2.03		**Shallow Concentrated Flow,** Grassed Waterway Kv= 15.0 fps
29.7	900	Total			

Subcatchment 2S: Basin B - Existing



Summary for Subcatchment 3S: Basin C - Historic

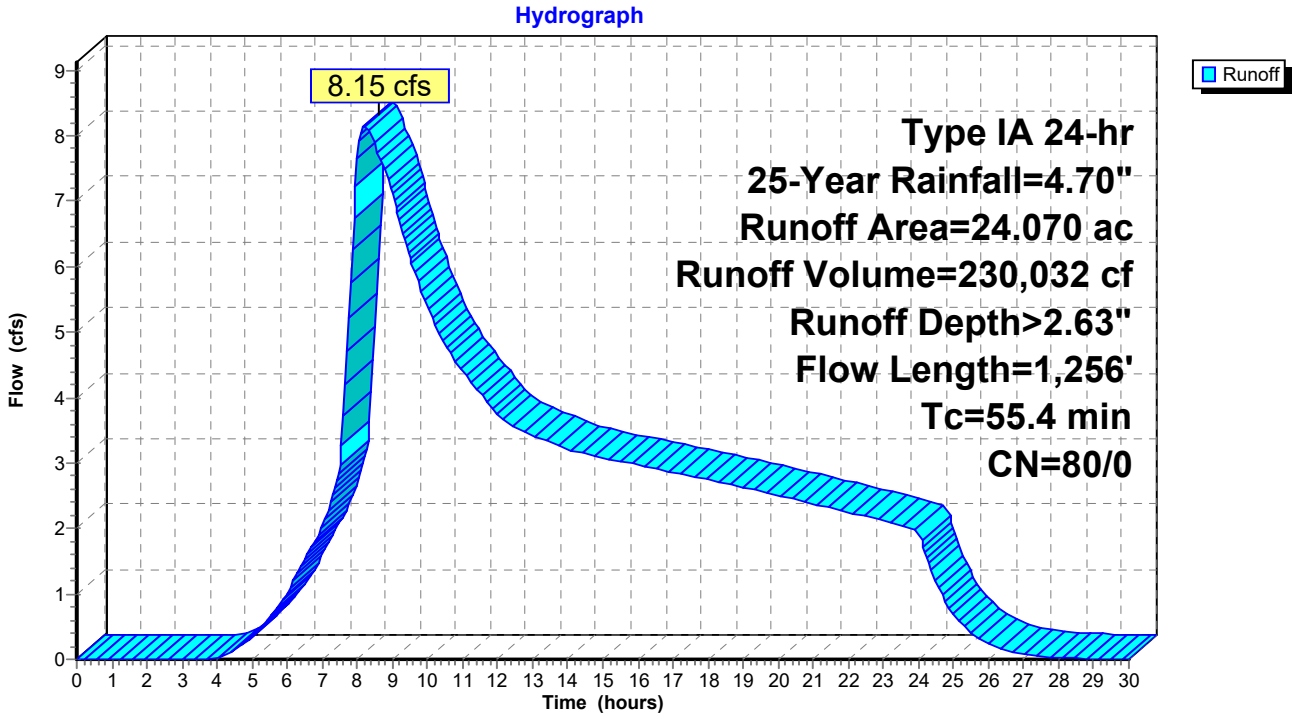
Runoff = 8.15 cfs @ 8.23 hrs, Volume= 230,032 cf, Depth> 2.63"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 25-Year Rainfall=4.70"

Area (ac)	CN	Description
24.070	80	>75% Grass cover, Good, HSG D
24.070	80	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
50.8	300	0.0100	0.10		**Sheet Flow,** Grass: Dense n= 0.240 P2= 2.55"
4.6	956	0.0540	3.49		**Shallow Concentrated Flow,** Grassed Waterway Kv= 15.0 fps
55.4	1,256	Total			

Subcatchment 3S: Basin C - Historic



Time span=0.00-30.00 hrs, dt=0.05 hrs, 601 points
Runoff by SBUH method, Split Pervious/Imperv.
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Basin B - Existing Runoff Area=22.930 ac 0.00% Impervious Runoff Depth=3.22"
Flow Length=900' Tc=29.7 min CN=77/0 Runoff=12.51 cfs 267,896 cf

Subcatchment3S: Basin C - Historic Runoff Area=24.070 ac 0.00% Impervious Runoff Depth>3.51"
Flow Length=1,256' Tc=55.4 min CN=80/0 Runoff=11.34 cfs 306,818 cf

Total Runoff Area = 2,047,320 sf Runoff Volume = 574,714 cf Average Runoff Depth = 3.37"
100.00% Pervious = 2,047,320 sf 0.00% Impervious = 0 sf

Summary for Subcatchment 2S: Basin B - Existing

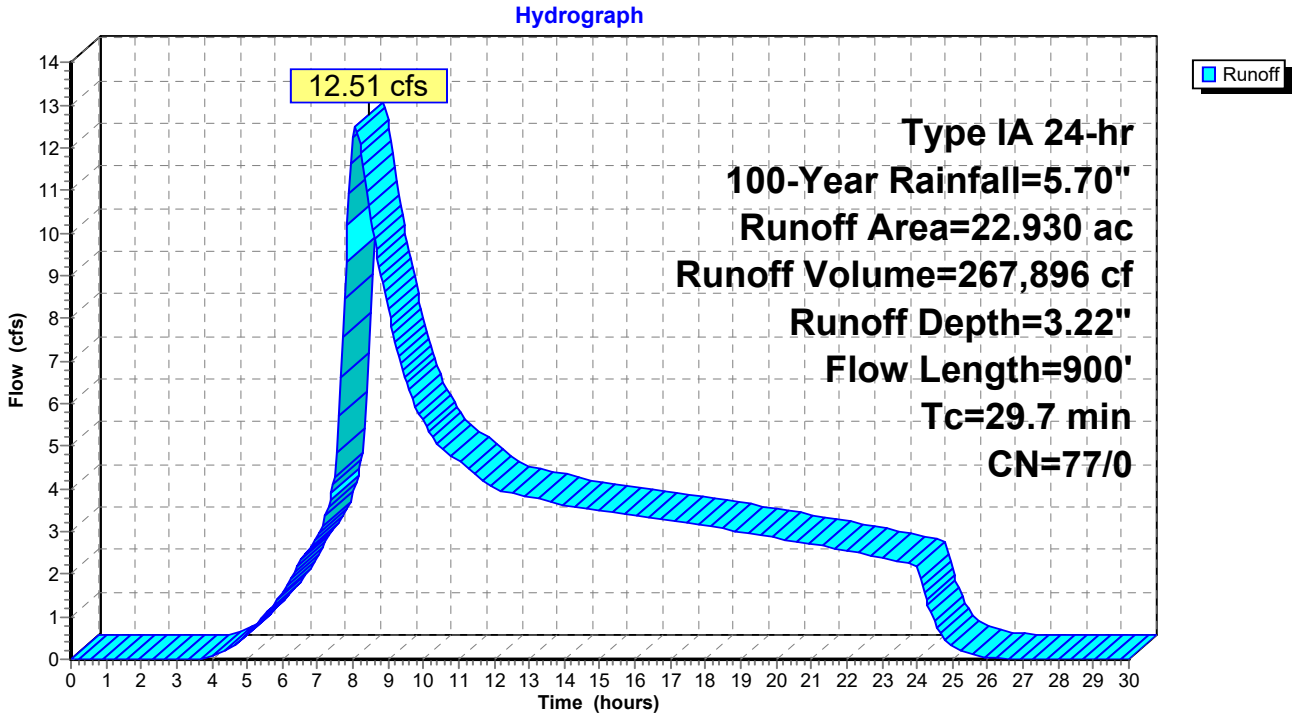
Runoff = 12.51 cfs @ 8.06 hrs, Volume= 267,896 cf, Depth= 3.22"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100-Year Rainfall=5.70"

	Area (ac)	CN	Description
*	22.930	77	>75% Grass cover, Good, HSG C/D
	22.930	77	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
24.8	300	0.0600	0.20		**Sheet Flow,** Grass: Dense n= 0.240 P2= 2.55"
4.9	600	0.0183	2.03		**Shallow Concentrated Flow,** Grassed Waterway Kv= 15.0 fps
29.7	900	Total			

Subcatchment 2S: Basin B - Existing



Summary for Subcatchment 3S: Basin C - Historic

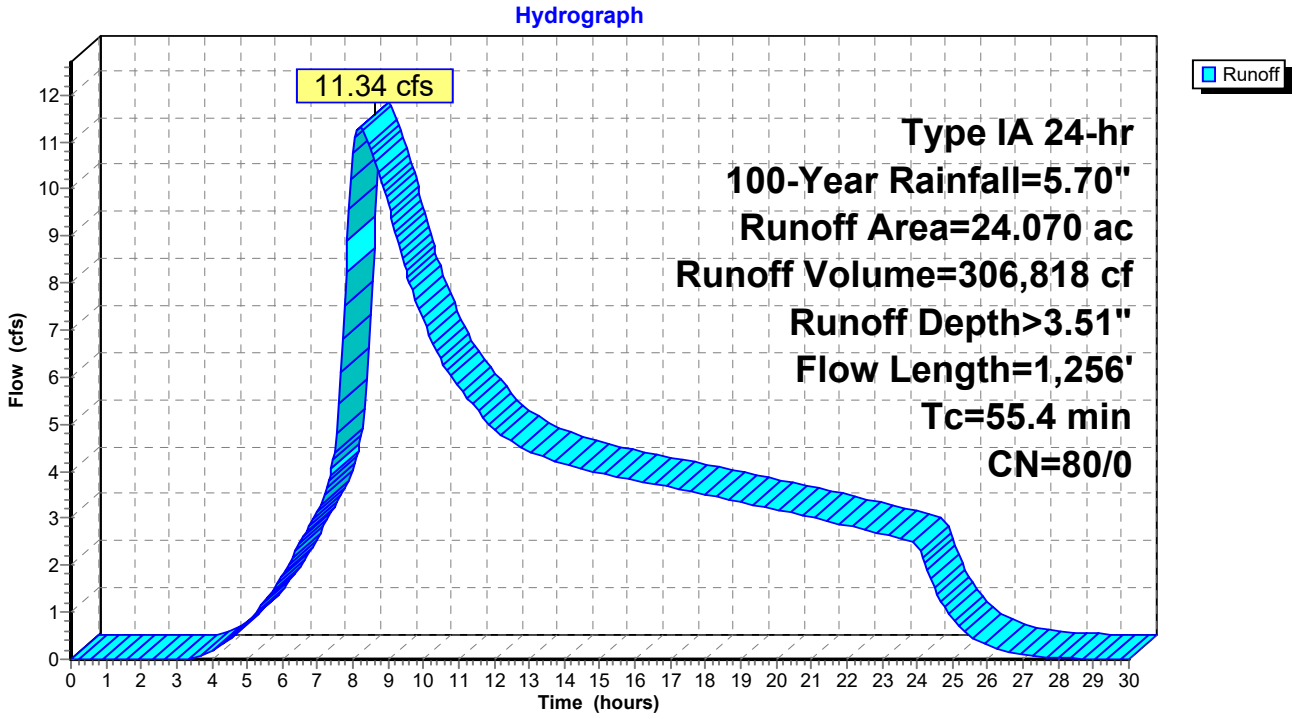
Runoff = 11.34 cfs @ 8.21 hrs, Volume= 306,818 cf, Depth> 3.51"

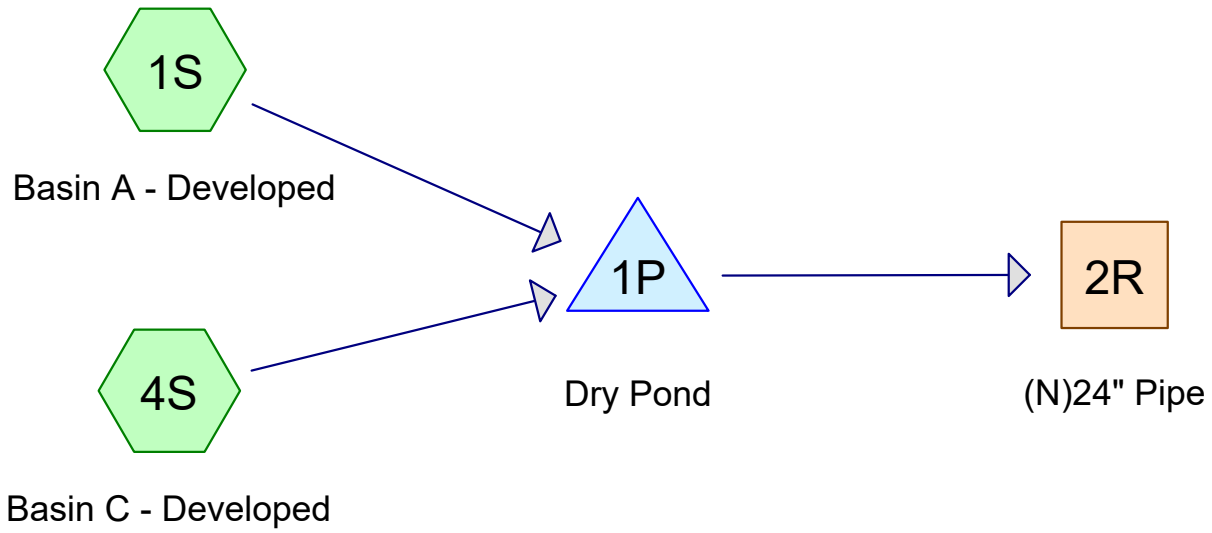
Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100-Year Rainfall=5.70"

Area (ac)	CN	Description
24.070	80	>75% Grass cover, Good, HSG D
24.070	80	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
50.8	300	0.0100	0.10		**Sheet Flow,** Grass: Dense n= 0.240 P2= 2.55"
4.6	956	0.0540	3.49		**Shallow Concentrated Flow,** Grassed Waterway Kv= 15.0 fps
55.4	1,256	Total			

Subcatchment 3S: Basin C - Historic





PROPOSED CONDITIONS

Subcat Reach Pond Link

Routing Diagram for 23449_Stormwater
Prepared by DEVCO Engineering, Inc, Printed 3/3/2026
HydroCAD® 10.20-8a s/n 03944 © 2025 HydroCAD Software Solutions LLC

Rainfall Events Listing (selected events)

Event#	Event Name	Storm Type	Curve	Mode	Duration (hours)	B/B	Depth (inches)	AMC
1	25-Year	Type IA 24-hr		Default	24.00	1	4.70	2
2	100-Year	Type IA 24-hr		Default	24.00	1	5.70	2

Area Listing (selected nodes)

Area (sq-ft)	CN	Description (subcatchment-numbers)
721,789	80	>75% Grass cover, Good, HSG D (1S, 4S)
269,201	98	Future Imperviuous Area (4S)
111,078	98	Impervious Area (1S)
154,638	98	New Impervious Area (4S)
1,256,706	**88**	**TOTAL AREA**

Soil Listing (selected nodes)

Area (sq-ft)	Soil Group	Subcatchment Numbers
0	HSG A	
0	HSG B	
0	HSG C	
721,789	HSG D	1S, 4S
534,917	Other	1S, 4S
1,256,706		**TOTAL AREA**

Ground Covers (selected nodes)

HSG-A (sq-ft)	HSG-B (sq-ft)	HSG-C (sq-ft)	HSG-D (sq-ft)	Other (sq-ft)	Total (sq-ft)	Ground Cover	Su Nu
0	0	0	721,789	0	721,789	>75% Grass cover, Good	
0	0	0	0	269,201	269,201	Future Imperviuous Area	
0	0	0	0	111,078	111,078	Impervious Area	
0	0	0	0	154,638	154,638	New Impervious Area	
0	**0**	**0**	**721,789**	**534,917**	**1,256,706**	**TOTAL AREA**	

Pipe Listing (selected nodes)

Line#	Node Number	In-Invert (feet)	Out-Invert (feet)	Length (feet)	Slope (ft/ft)	n	Width (inches)	Diam/Height (inches)	Inside-Fill (inches)	Node Name
1	2R	274.50	273.50	112.0	0.0089	0.013	0.0	24.0	0.0	
2	1P	275.25	274.00	50.0	0.0250	0.013	0.0	18.0	0.0	

Time span=0.00-30.00 hrs, dt=0.05 hrs, 601 points
Runoff by SBUH method, Split Pervious/Imperv.
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment1S: Basin A - Developed Runoff Area=4.780 ac 53.35% Impervious Runoff Depth=3.61"
Tc=5.0 min CN=80/98 Runoff=4.24 cfs 62,630 cf

Subcatchment4S: Basin C - Developed Runoff Area=24.070 ac 40.42% Impervious Runoff Depth=3.37"
Tc=5.0 min CN=80/98 Runoff=19.86 cfs 294,706 cf

Reach 2R: (N)24" Pipe Avg. Flow Depth=0.85' Max Vel=6.33 fps Inflow=8.10 cfs 304,324 cf
24.0" Round Pipe n=0.013 L=112.0' S=0.0089 '/' Capacity=21.38 cfs Outflow=8.10 cfs 304,300 cf

Pond 1P: Dry Pond Peak Elev=282.10' Storage=106,025 cf Inflow=24.10 cfs 357,337 cf
Outflow=8.10 cfs 304,324 cf

Total Runoff Area = 1,256,706 sf Runoff Volume = 357,337 cf Average Runoff Depth = 3.41"
57.44% Pervious = 721,789 sf 42.56% Impervious = 534,917 sf

Summary for Subcatchment 1S: Basin A - Developed

[49] Hint: Tc<2dt may require smaller dt

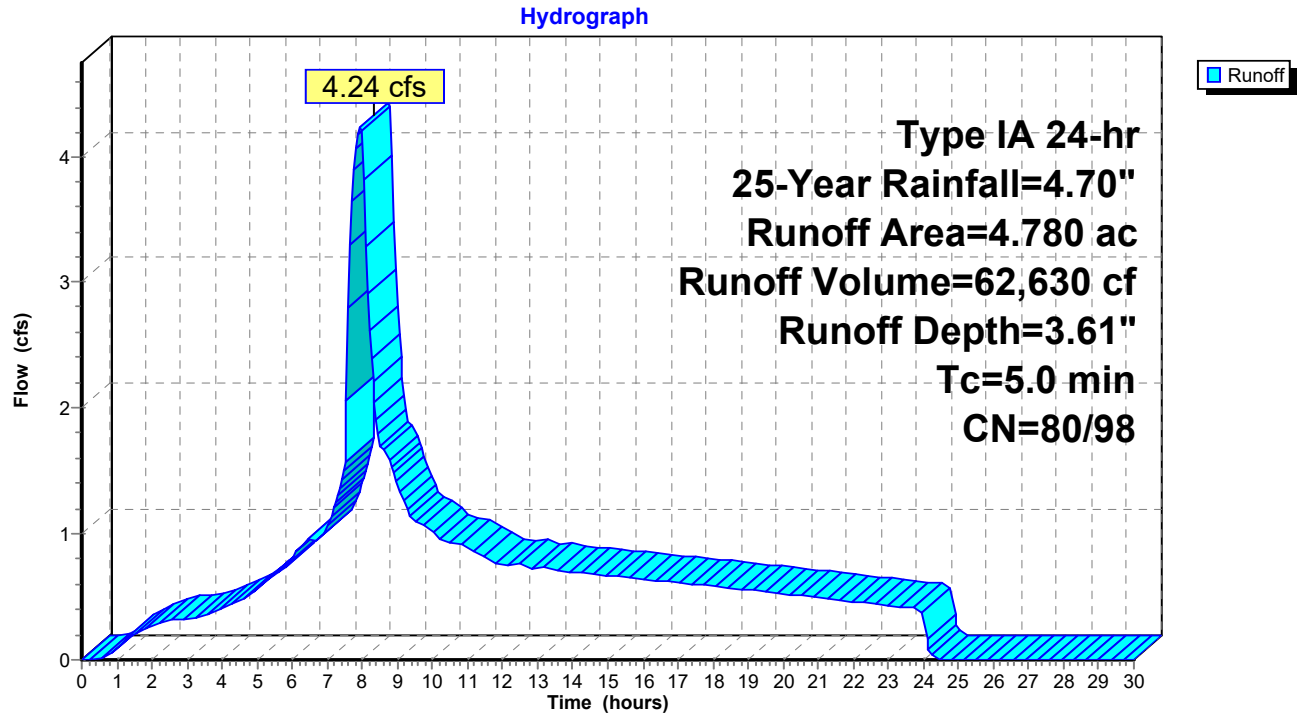
Runoff = 4.24 cfs @ 7.92 hrs, Volume= 62,630 cf, Depth= 3.61"
Routed to Pond 1P : Dry Pond

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 25-Year Rainfall=4.70"

	Area (ac)	CN	Description
*	2.550	98	Impervious Area
	2.230	80	>75% Grass cover, Good, HSG D
	4.780	90	Weighted Average
	2.230	80	46.65% Pervious Area
	2.550	98	53.35% Impervious Area

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

Subcatchment 1S: Basin A - Developed



Summary for Subcatchment 4S: Basin C - Developed

[49] Hint: Tc<2dt may require smaller dt

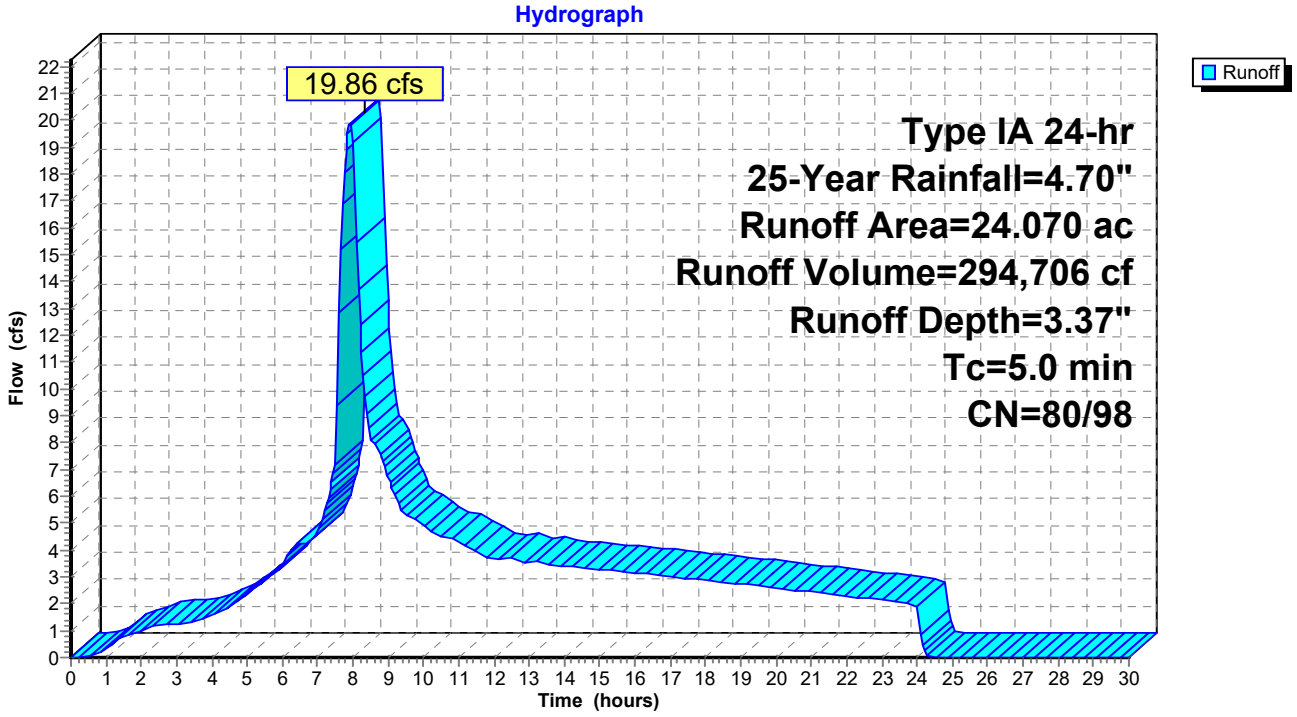
Runoff = 19.86 cfs @ 7.93 hrs, Volume= 294,706 cf, Depth= 3.37"
Routed to Pond 1P : Dry Pond

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 25-Year Rainfall=4.70"

	Area (ac)	CN	Description
*	3.550	98	New Impervious Area
*	6.180	98	Future Impervious Area
	14.340	80	>75% Grass cover, Good, HSG D
	24.070	87	Weighted Average
	14.340	80	59.58% Pervious Area
	9.730	98	40.42% Impervious Area

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

Subcatchment 4S: Basin C - Developed



Summary for Reach 2R: (N)24" Pipe

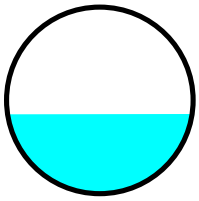
[52] Hint: Inlet/Outlet conditions not evaluated
[79] Warning: Submerged Pond 1P Primary device # 1 INLET by 0.10'

Inflow Area = 1,256,706 sf, 42.56% Impervious, Inflow Depth > 2.91" for 25-Year event
Inflow = 8.10 cfs @ 9.00 hrs, Volume= 304,324 cf
Outflow = 8.10 cfs @ 9.01 hrs, Volume= 304,300 cf, Atten= 0%, Lag= 0.5 min

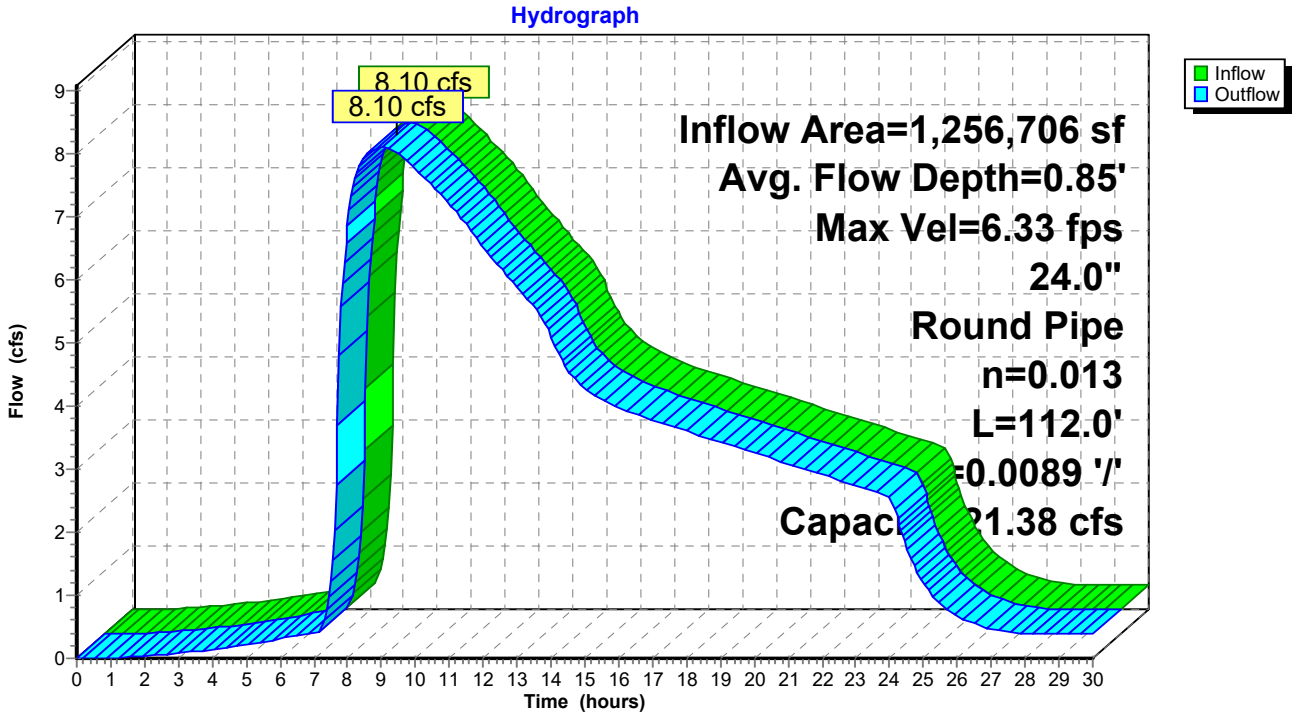
Routing by Stor-Ind+Trans method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Max. Velocity= 6.33 fps, Min. Travel Time= 0.3 min
Avg. Velocity = 4.08 fps, Avg. Travel Time= 0.5 min

Peak Storage= 143 cf @ 9.01 hrs
Average Depth at Peak Storage= 0.85' , Surface Width= 1.98'
Bank-Full Depth= 2.00' Flow Area= 3.1 sf, Capacity= 21.38 cfs

24.0" Round Pipe
n= 0.013
Length= 112.0' Slope= 0.0089 '/'
Inlet Invert= 274.50', Outlet Invert= 273.50'



Reach 2R: (N)24" Pipe



Summary for Pond 1P: Dry Pond

[44] Hint: Outlet device #4 is below defined storage

Inflow Area = 1,256,706 sf, 42.56% Impervious, Inflow Depth = 3.41" for 25-Year event
Inflow = 24.10 cfs @ 7.93 hrs, Volume= 357,337 cf
Outflow = 8.10 cfs @ 9.00 hrs, Volume= 304,324 cf, Atten= 66%, Lag= 64.2 min
Primary = 8.10 cfs @ 9.00 hrs, Volume= 304,324 cf
Routed to Reach 2R : (N)24" Pipe

Routing by Stor-Ind method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Peak Elev= 282.10' @ 9.00 hrs Surf.Area= 31,718 sf Storage= 106,025 cf

Plug-Flow detention time= 262.3 min calculated for 303,817 cf (85% of inflow)
Center-of-Mass det. time= 163.9 min (876.9 - 713.1)

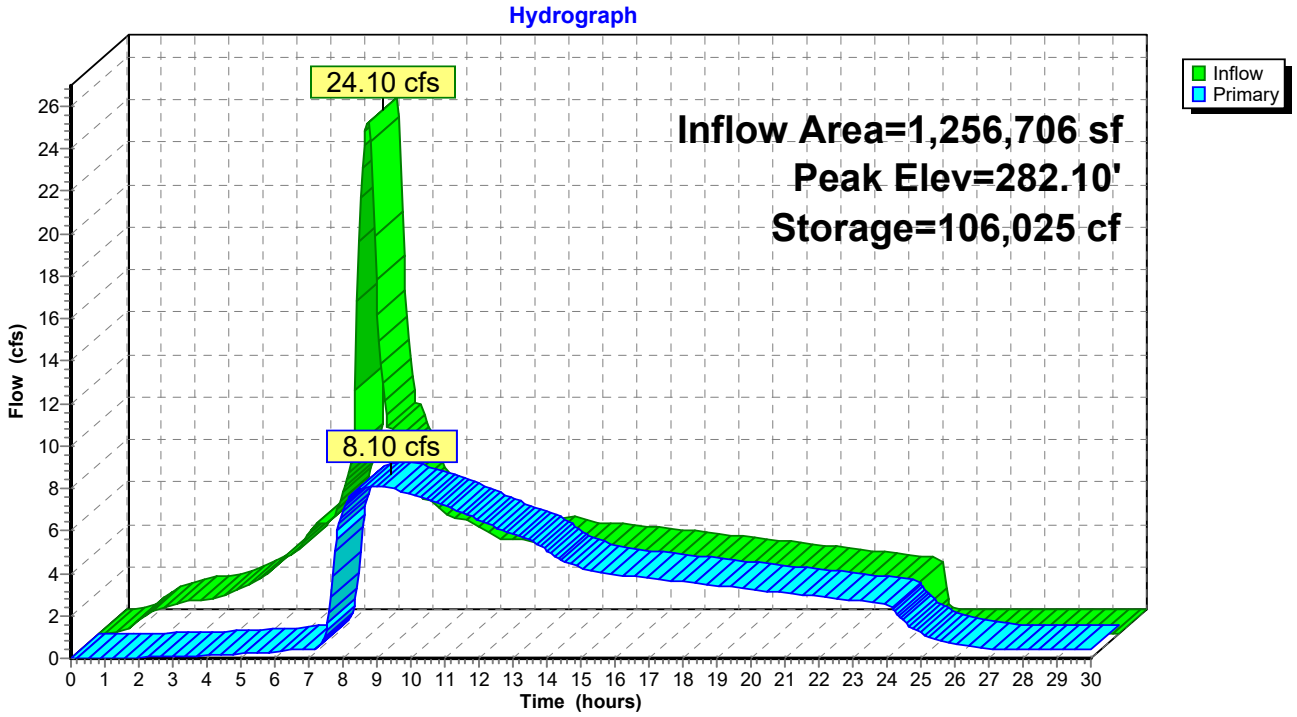
Volume	Invert	Avail.Storage	Storage Description
#1	278.00'	135,750 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
278.00	20,010	0	0
283.00	34,290	135,750	135,750

Device	Routing	Invert	Outlet Devices
#1	Primary	275.25'	**18.0" Round Culvert** L= 50.0' CMP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 275.25' / 274.00' S= 0.0250 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	278.00'	**3.000 in/hr Exfiltration over Surface area from 278.00' - 280.00'** Excluded Surface area = 20,010 sf
#3	Device 1	280.40'	**24.0" W x 24.0" H 45° Grate - Inlet** C= 0.600 Limited to weir flow at low heads
#4	Device 3	274.25'	**15.0" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#5	Device 3	282.50'	**24.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=8.10 cfs @ 9.00 hrs HW=282.10' (Free Discharge)
└─**1=Culvert** (Passes 8.10 cfs of 16.59 cfs potential flow)
  ├─**2=Exfiltration** (Exfiltration Controls 0.40 cfs)
  └─**3=Grate - Inlet** (Passes 7.70 cfs of 18.73 cfs potential flow)
    ├─**4=Orifice** (Orifice Controls 7.70 cfs @ 6.28 fps)
    └─**5=Overflow** (Controls 0.00 cfs)

Pond 1P: Dry Pond



Time span=0.00-30.00 hrs, dt=0.05 hrs, 601 points
Runoff by SBUH method, Split Pervious/Imperv.
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment1S: Basin A - Developed Runoff Area=4.780 ac 53.35% Impervious Runoff Depth=4.55"
Tc=5.0 min CN=80/98 Runoff=5.38 cfs 78,988 cf

Subcatchment4S: Basin C - Developed Runoff Area=24.070 ac 40.42% Impervious Runoff Depth=4.30"
Tc=5.0 min CN=80/98 Runoff=25.60 cfs 375,723 cf

Reach 2R: (N)24" Pipe Avg. Flow Depth=1.07' Max Vel=6.99 fps Inflow=11.94 cfs 401,426 cf
24.0" Round Pipe n=0.013 L=112.0' S=0.0089 '/' Capacity=21.38 cfs Outflow=11.94 cfs 401,402 cf

Pond 1P: Dry Pond Peak Elev=282.74' Storage=127,093 cf Inflow=30.98 cfs 454,710 cf
Outflow=11.94 cfs 401,426 cf

Total Runoff Area = 1,256,706 sf Runoff Volume = 454,710 cf Average Runoff Depth = 4.34"
57.44% Pervious = 721,789 sf 42.56% Impervious = 534,917 sf

Summary for Subcatchment 1S: Basin A - Developed

[49] Hint: Tc<2dt may require smaller dt

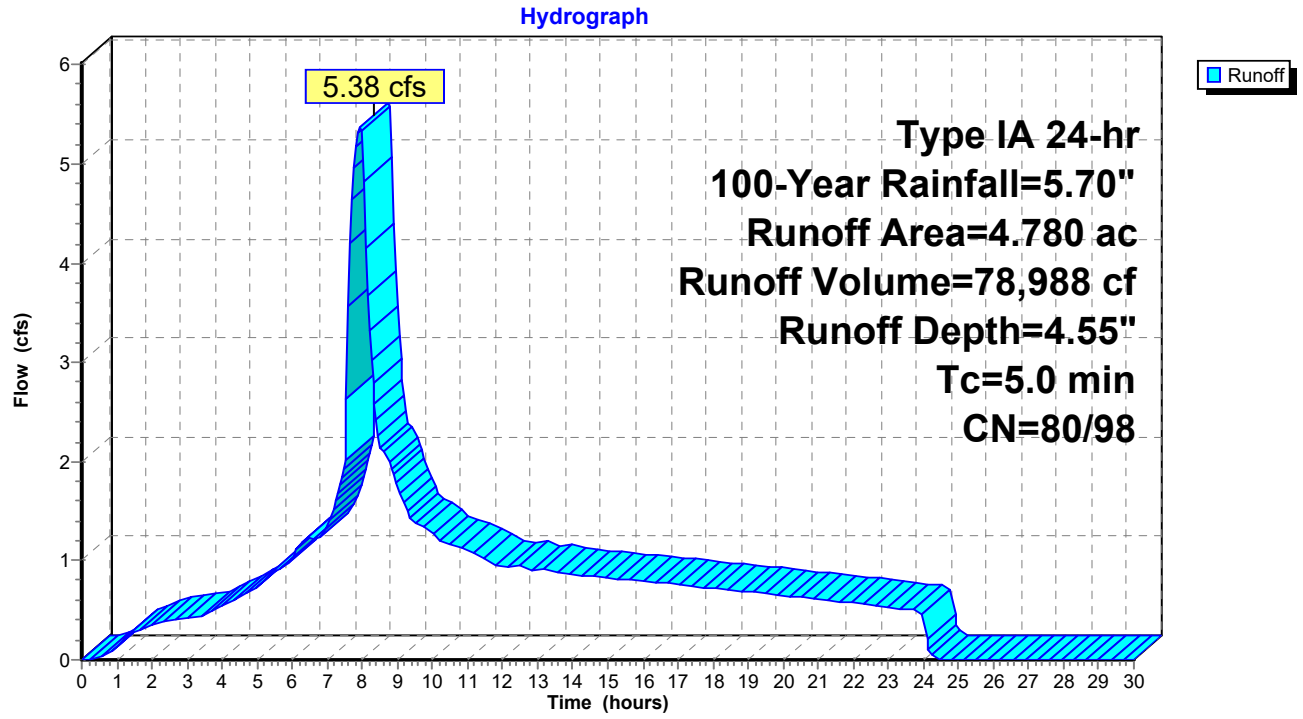
Runoff = 5.38 cfs @ 7.92 hrs, Volume= 78,988 cf, Depth= 4.55"
Routed to Pond 1P : Dry Pond

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100-Year Rainfall=5.70"

	Area (ac)	CN	Description
*	2.550	98	Impervious Area
	2.230	80	>75% Grass cover, Good, HSG D
	4.780	90	Weighted Average
	2.230	80	46.65% Pervious Area
	2.550	98	53.35% Impervious Area

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

Subcatchment 1S: Basin A - Developed



Summary for Subcatchment 4S: Basin C - Developed

[49] Hint: Tc<2dt may require smaller dt

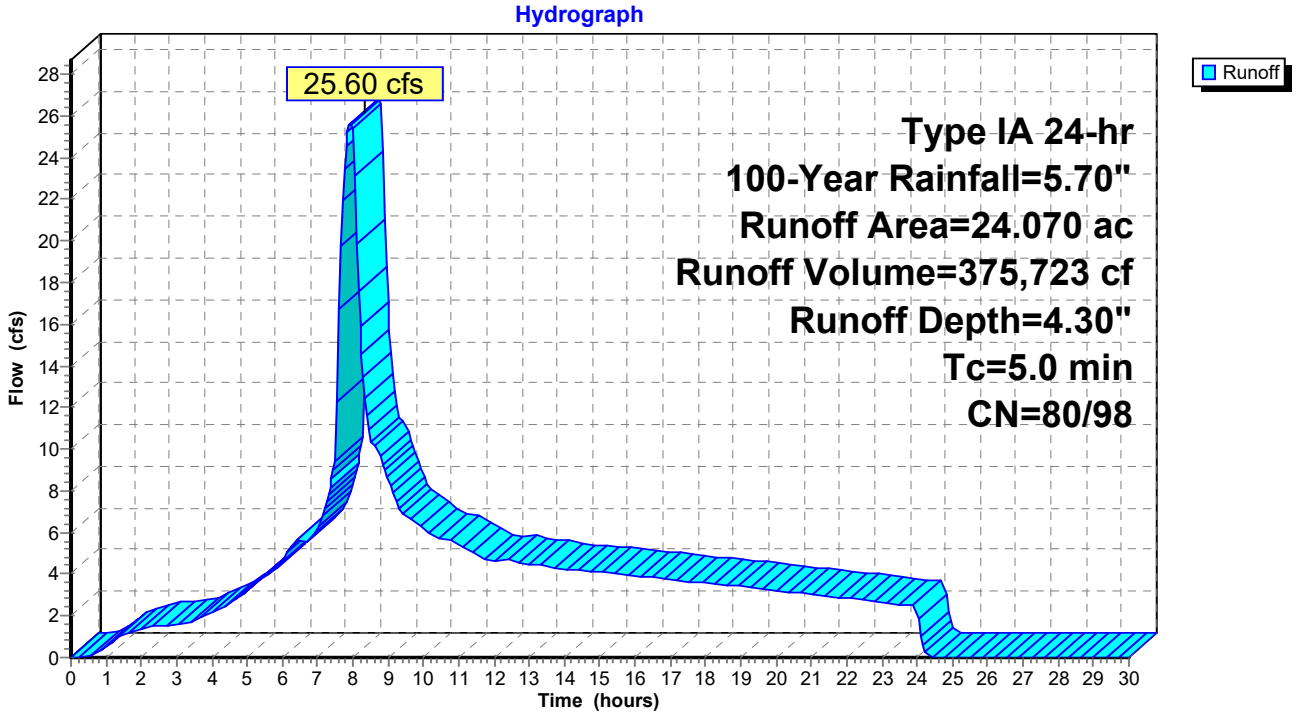
Runoff = 25.60 cfs @ 7.93 hrs, Volume= 375,723 cf, Depth= 4.30"
Routed to Pond 1P : Dry Pond

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100-Year Rainfall=5.70"

	Area (ac)	CN	Description
*	3.550	98	New Impervious Area
*	6.180	98	Future Impervious Area
	14.340	80	>75% Grass cover, Good, HSG D
	24.070	87	Weighted Average
	14.340	80	59.58% Pervious Area
	9.730	98	40.42% Impervious Area

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

Subcatchment 4S: Basin C - Developed



Summary for Reach 2R: (N)24" Pipe

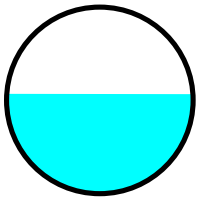
[52] Hint: Inlet/Outlet conditions not evaluated
[79] Warning: Submerged Pond 1P Primary device # 1 INLET by 0.32'

Inflow Area = 1,256,706 sf, 42.56% Impervious, Inflow Depth > 3.83" for 100-Year event
Inflow = 11.94 cfs @ 8.73 hrs, Volume= 401,426 cf
Outflow = 11.94 cfs @ 8.74 hrs, Volume= 401,402 cf, Atten= 0%, Lag= 0.5 min

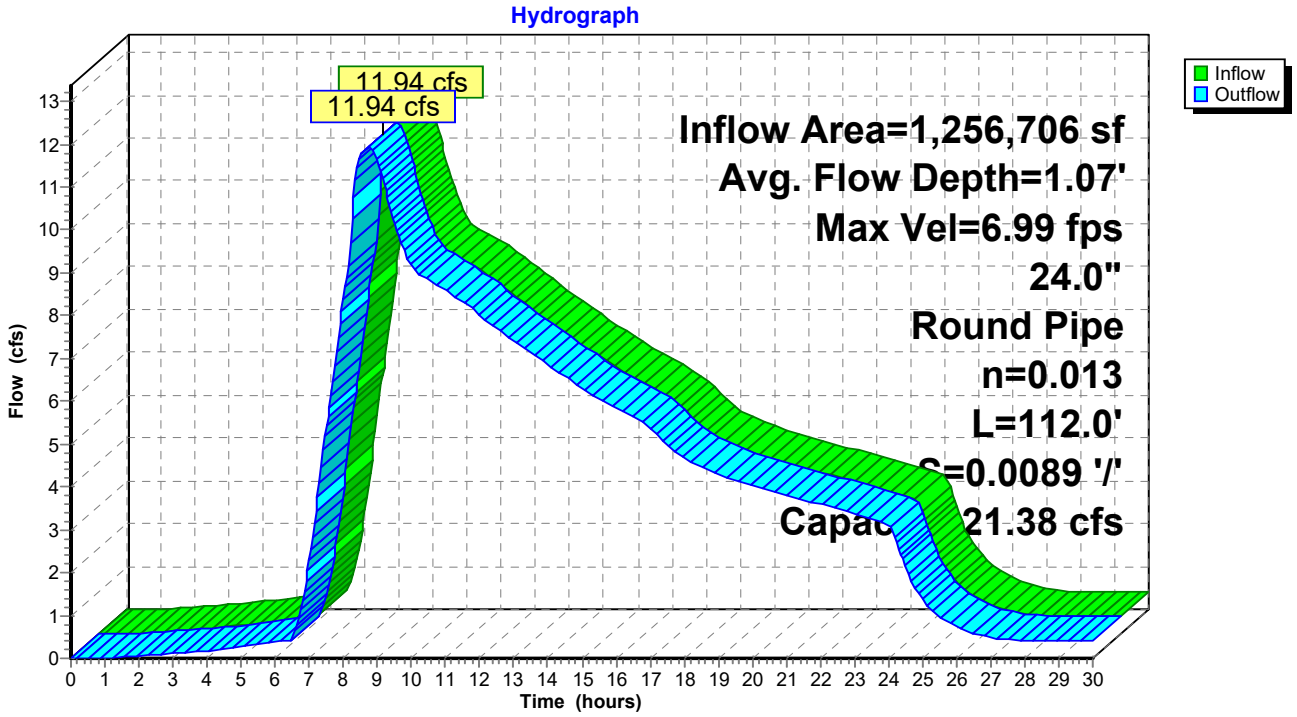
Routing by Stor-Ind+Trans method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Max. Velocity= 6.99 fps, Min. Travel Time= 0.3 min
Avg. Velocity = 4.40 fps, Avg. Travel Time= 0.4 min

Peak Storage= 191 cf @ 8.74 hrs
Average Depth at Peak Storage= 1.07' , Surface Width= 2.00'
Bank-Full Depth= 2.00' Flow Area= 3.1 sf, Capacity= 21.38 cfs

24.0" Round Pipe
n= 0.013
Length= 112.0' Slope= 0.0089 '/'
Inlet Invert= 274.50', Outlet Invert= 273.50'



Reach 2R: (N)24" Pipe



Summary for Pond 1P: Dry Pond

[44] Hint: Outlet device #4 is below defined storage

Inflow Area = 1,256,706 sf, 42.56% Impervious, Inflow Depth = 4.34" for 100-Year event
Inflow = 30.98 cfs @ 7.93 hrs, Volume= 454,710 cf
Outflow = 11.94 cfs @ 8.73 hrs, Volume= 401,426 cf, Atten= 61%, Lag= 48.3 min
Primary = 11.94 cfs @ 8.73 hrs, Volume= 401,426 cf
Routed to Reach 2R : (N)24" Pipe

Routing by Stor-Ind method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Peak Elev= 282.74' @ 8.73 hrs Surf.Area= 33,561 sf Storage= 127,093 cf

Plug-Flow detention time= 234.6 min calculated for 401,426 cf (88% of inflow)
Center-of-Mass det. time= 154.4 min (860.4 - 706.0)

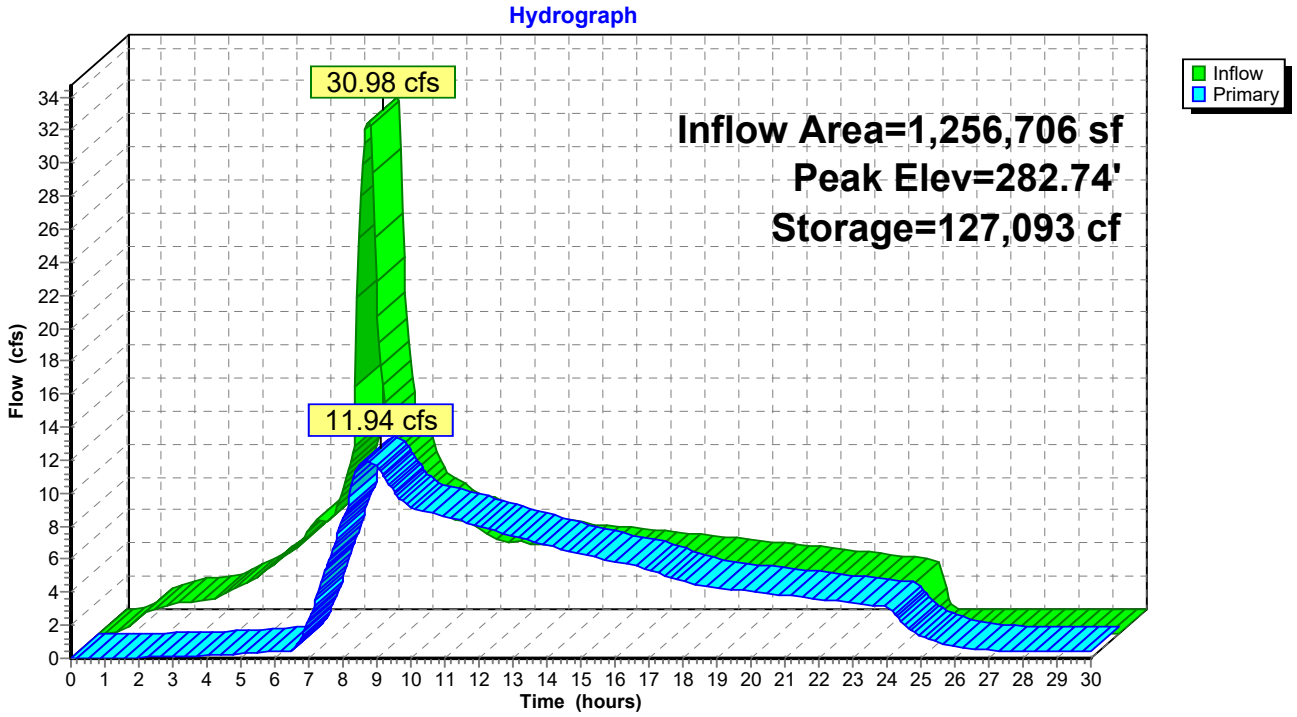
Volume	Invert	Avail.Storage	Storage Description
#1	278.00'	135,750 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
278.00	20,010	0	0
283.00	34,290	135,750	135,750

Device	Routing	Invert	Outlet Devices
#1	Primary	275.25'	**18.0" Round Culvert** L= 50.0' CMP, projecting, no headwall, Ke= 0.900 Inlet / Outlet Invert= 275.25' / 274.00' S= 0.0250 '/' Cc= 0.900 n= 0.013, Flow Area= 1.77 sf
#2	Device 1	278.00'	**3.000 in/hr Exfiltration over Surface area from 278.00' - 280.00'** Excluded Surface area = 20,010 sf
#3	Device 1	280.40'	**24.0" W x 24.0" H 45° Grate - Inlet** C= 0.600 Limited to weir flow at low heads
#4	Device 3	274.25'	**15.0" Horiz. Orifice** C= 0.600 Limited to weir flow at low heads
#5	Device 3	282.50'	**24.0" Horiz. Overflow** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=11.93 cfs @ 8.73 hrs HW=282.74' (Free Discharge)
└**1=Culvert** (Passes 11.93 cfs of 17.45 cfs potential flow)
├**2=Exfiltration** (Exfiltration Controls 0.40 cfs)
└**3=Grate - Inlet** (Passes 11.53 cfs of 24.45 cfs potential flow)
├**4=Orifice** (Orifice Controls 9.05 cfs @ 7.37 fps)
└**5=Overflow** (Weir Controls 2.49 cfs @ 1.62 fps)

Pond 1P: Dry Pond



Appendix

e. Soil condition, type, and composition

f. Land use and zoning of the subject and surrounding property or properties

g. Projected density and impervious area created by the subject and surrounding property or properties at build-out

h. Flows from commercial, industrial or institutional users

i. Condition and size of existing storm drains, ditches, and conveyances

j. Location of approved disposal point(s)

k. Long-term maintenance responsibility, including accessibility for cleaning, inspection, personnel, and equipment.

1.2 DESIGN CALCULATIONS AND CAPACITY

A. Design Calculations

1. Design calculations shall be submitted for all drainage facilities and shall be stamped by a professional engineer licensed in the State of Oregon. Peak flows shall be calculated using either the Rational Method or the Santa Barbara Urban Hydrograph (SBUH) method, subject to requirements herein and direction from the County Engineer and/or other agencies with jurisdiction.

    a. A summary of these drainage calculations, including basin maps, shall be included on the site plan drawings.

2. Rational Method: One method used for calculating peak flows from small drainages less than 200 acres is the Rational Method.

    a. Peak design discharges shall be computed using the rational method formula, Q=CiA, where Q = flow in cfs, C = runoff coefficient, i = rainfall intensity, and A = area in acres.

    b. The County Engineer reserves the right to verify all calculations using the Rational Method, and to require larger pipe sizes if the Rational Method calculations result in higher flows than the SBUH methodology.

3. SBUH: Another method of involves the use of the Santa Barbara Urban Hydrograph (SBUH) method to develop runoff hydrographs using 24-hour storm data for the local area, based on current NOAA Atlas 24-hour isopluvials for Oregon.

    a. For Philomath, the 24-hour precipitation values from the NOAA Atlas 2, Volume X, for use with the SBUH method are as follows (ODOT design rainfall intensities may differ):

        i. 5-YR 24-Hr = 3.7"

        ii. 10-YR 24-Hr = 4.3"

        iii. 25-YR 24-Hr = 5.0"

iv. 50-YR 24-Hr = 5.7"

b. For Corvallis and within the Corvallis urban growth boundary use the methods described in the City of Corvallis Stormwater Design Criteria.

c. For Albany and within the Albany urban growth boundary use the methods described in the City of Albany Engineering Standards, Division E, Stormwater Management Engineering Standards.

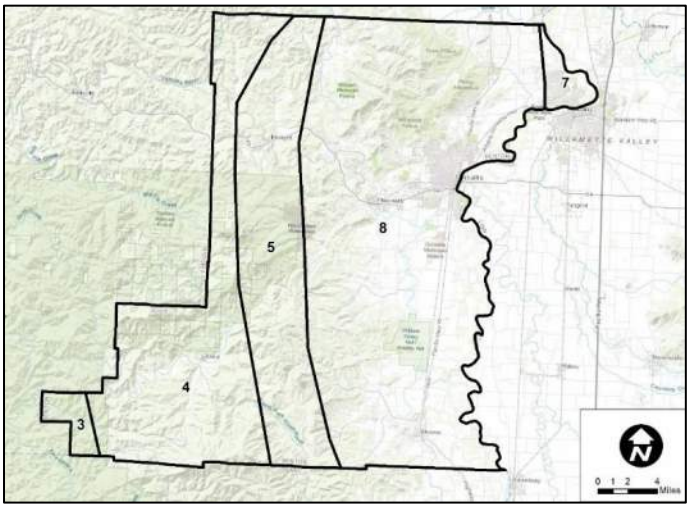
d. For the balance of the unincorporated areas of Benton County use the 24-hour precipitation values calculated using ODOT IDF and local precipitation data, for use with the SBUH method as follows:

e. The County Engineer reserves the right to verify all calculations using the Rational Method, and to require larger pipe sizes if the Rational Method calculations result in higher flows than the SBUH methodology.

Table 1 24-hour Precipitation Depth-Frequency Relationships for Benton County Zones Defined from ODOT's Rainfall Intensity-Depth-Recurrence Interval (IDRI) Maps.

Zone	Area (mi^2)	Average Annual Depth (inches)	Precipitation Depth (inches)							
			2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr	1000-yr
3	10.3	75	3.7	4.4	4.9	5.7	6.3	6.9	8.3	8.9
4	153.2	80	4.0	4.7	5.3	6.1	6.8	7.4	8.9	9.5
5	127.5	74	3.8	4.6	5.1	6.0	6.6	7.2	8.7	9.4
7	10.8	43	2.4	2.9	3.3	3.9	4.3	4.8	5.9	6.4
8	376	53	2.9	3.5	4.0	4.7	5.2	5.7	7.0	7.5

Figure 1 ODOT IDRI zones in Benton County



criteria above and receive DEQ approval prior to final County approval or construction.

2.0 STORMWATER QUALITY CONTROL (TREATMENT)

2.1 STORMWATER QUALITY FACILITIES

A. Where Required

1. Development and other activities which create or replace 10,890 square feet or more of impervious surfaces, or increase the amount or concentration of runoff are required to provide permanent water quality facilities to reduce contaminants entering the storm water system and the Benton County stormwater conveyances.

2. An on-site water quality facility shall be constructed unless, in the judgment of the County Engineer, any of the following conditions exist:

    a. There is a more efficient and effective regional site within the sub-basin that was designed to incorporate the development or is in the near vicinity with the capacity to treat the site.

    b. A sufficient buffer exists between the constructed impervious surfaces and open conveyances (ditches, streams, creeks, rivers).

3. If an on-site water quality facility cannot be constructed to treat the runoff from the development's impervious surface, or if portions of the on-site runoff cannot be treated, then the County Engineer may allow an offsite water quality facility to be designed to treat runoff from an equivalent area of adjacent untreated impervious surfaces.

4. Exempt from this requirement are water lines, sanitary sewer lines, private utilities, or other land development activities that will not directly add impervious surface area or increase the amount of storm water runoff or pollution leaving the site once construction has been completed and the site is either restored to or not altered from its approximate original condition. Projects will be considered on a case-by-case basis by the County Engineer, based on existing facilities, nature of construction activities, maintenance considerations, State and Federal permitting requirements and other factors.

B. Stormwater Quality Design

1. The stormwater quality facilities shall be designed to remove suspended solids, debris, oil and grease carried by the storm drain system, and retain them for safe and easy removal. The stormwater quality facility chosen shall possess design features to prevent resuspension of previously collected contaminants and materials.

2. The stormwater quality facilities shall treat the runoff from all newly constructed impervious surfaces. For purposes of stormwater quality requirements, impervious surfaces shall include pavement, gravel roads, buildings, public and private roadways, and all other surfaces with similar runoff characteristics.

3. Design Rainfall. The stormwater quality facilities shall be designed for the runoff from one half of the 2-year; 24-hour storm event (i.e. first flush). Flows resulting from rainfall greater than one half of the 2-year; 24-hour storm event may either bypass the water quality facility or pass through the water quality facility with a reduced level of treatment.

4. Residential Subdivisions, Impervious Area. The water quality facilities for any residential subdivision shall be adequately sized to accommodate the public streets & infrastructure within the subdivision, as well as for the future construction of housing on the individual lots based on an assumed 10,890 square feet of impervious surface per dwelling unit.

5. An Operations & Maintenance Plan or Manual (O&MP or O&MM) shall be prepared detailing how the water quality facility is to be maintained, and shall be submitted for review with the final design.

6. Long-Term Maintenance Agreement. (LTMA) For private stormwater quality facilities, an executed LTMA approved by the County Engineer prior to final approval or acceptance of the water quality facility, and recorded against all affected properties.

7. Infiltration Testing Requirements. If a design based on stormwater infiltration is proposed, soil infiltration tests shall be performed as summarized in Section 2.2 below, including type of testing, notice of testing, location and depth of testing, etc. A final infiltration report stamped by the Engineer shall be provided with the design drawings submitted for review by the County.

C. Water Quality BMPs

1. Approval of the appropriate BMP for each development will be considered on a case-by-case basis by the County Engineer, and shall be based on maintenance considerations, nature of activities within the development, and other factors.

2. The following are guidelines for BMPs generally acceptable to the County. This listing is not all inclusive, and other design options which are functionally equivalent may be proposed for consideration by the County Engineer.

3. Mechanical Treatment

   a. The device shall be capable of removing at least 80% of suspended fine and coarse sediment load and 80% of the floatable oil and grease.

   b. The selected device shall incorporate a high-flow bypass or contain an internal built-in diversion structure to divert intense runoff events and prevent scouring of the previously collected contaminants.

   c. The device shall be capable of storing a minimum of 24-inches of sediment or 15% of the operating volume, whichever is greater. The storage chamber shall incorporate a means of inspecting sediment levels, as well as oil and grease accumulation levels, without entering the structure.

3.12.7 Storm Drain Outfalls

A. All outfalls shall conform to the requirements of all federal, state, and local regulations.

B. Outfalls shall be above the mean low water level except as approved by the City Engineer. Installation of tide gates may be required when the outfall is in a tailwater condition.

C. Erosion must be prevented at the outfall. All outfalls shall be provided with a rock splash or other approved erosion control protection measure. Mechanisms which reduce velocity prior to discharge from an outfall are encouraged and may be required. Examples are drop manholes, energy dissipaters, and rapid expansion into pipes of much larger size.

D. Other forms of energy dissipation may include stilling basins, drop pools, hydraulic jump basins, baffled aprons, or bucket aprons, shall be provided for outfalls with velocities at design flow greater than 10 FPS.

E. If required, tidegates, flapgates, or other outlet gates will be installed on specified outfalls. Gates will meet the requirements of the Oregon Department of Fish and Wildlife (ODFW), National Oceanic and Atmospheric Administration (NOAA), and other agencies as applicable.

3.12.8 On-site Detention Design - General

A. Mitigation of the impacts of new development on the downstream drainage system can be accomplished through on-site detention systems or by improving the capacity of the downstream conveyance system.

B. On-site detention facilities shall be constructed when any of the following conditions exist:
   1. An identified downstream deficiency along with upstream detention, rather than downstream conveyance system enlargement, is determined to be the more effective solution.
   2. There is an identified regional detention site within the boundary of the development.
   3. The need for pre-treatment of stormwater discharge dictates that flows be detained for water quality processes.
   4. There is a need to mitigate flow impacts on receiving streams.
   5. There is a need for additional detention due to an increase in impermeable surface area.

C. When required, on-site stormwater detention facilities shall be designed to capture run-off so the run-off rates from the site after development do not exceed the predevelopment conditions, based upon a 25-year, 24-hour return storm. Volume and duration of predevelopment conditions will be considered.

D. When required, due to an identified downstream deficiency, on-site stormwater detention facilities shall be designed so that peak run-off rates

will not exceed predevelopment rates for the specific range of storms that cause the downstream deficiency.

E. Construction of on-site detention shall not be allowed as an option if such a detention facility would have an adverse effect upon receiving waters in the basin or sub-basin in the event of flooding, or would increase the likelihood or severity of flooding problems downstream of the site.

F. Impervious Area Calculations
   1. For single family and duplex residential subdivisions, stormwater quantity detention facilities shall be used for all impervious areas created by the subdivisions, including all streets, residences on individual lots at a rate of 2,640 square feet of impervious surface area per dwelling unit, and other impervious areas. Such facilities shall be constructed with the subdivisions public improvements.
   2. For all development other than single family and duplex, the sizing of stormwater detention facilities shall be based on the impervious area to be created by the development, including structures and all streets and impervious areas. Impervious surfaces shall be determined based upon building permits, construction plans, aerial mapping, or other appropriate methods as deemed reliable by the City Engineer or representative.

3.12.9 Detention Pond Design

A. Detention ponds and other open impoundment facilities such as landscape areas, open playing fields, and parklands, shall comply with the requirements of ORS 537, in general, and more specifically, ORS 537.4.

B. Facility Geometrics
   1. Interior side slopes up to the maximum water surface shall be no steeper than 3H:1V. If interior slopes need to be mowed, the slope shall be 4H:1V
   2. Exterior side slopes shall not be steeper than 2H:1V unless analyzed for stability by a geotechnical engineer.
   3. Ponds walls and/or dikes may be retaining walls, provided that the design is prepared and stamped by a registered professional engineer and that a fence is provided along the top of the wall and that at least 25 percent of the pond perimeter will be a vegetated soil slope of not greater than 3H:1V.
   4. Pond bottoms shall be level, and shall be located a minimum of 0.5 feet below the inlet and outlet to provide sediment storage.
   5. Outlet control systems shall utilize gates, valves, weirs, or other control structures and systems to control the outflow from the pond so that the downstream systems are not overwhelmed. If desired, water must be capable of being held in the pond indefinitely.

C. Overflow/Emergency Spillway
   1. A pond overflow system shall provide controlled discharge of the design storm event for developed contributing area without

overtopping any part of the pond embankment for exceeding the capacity of the emergency spillway.

2. The design shall provide controlled discharge directly into the downstream conveyance system.
3. An emergency overflow spillway (secondary overflow) shall be provided to safely pass the 100-year, 24-hour design storm event over the pond embankment in the event of control structure failure and for storm/runoff events exceeding design.
4. The spillway shall be located to direct overflows safely towards the downstream conveyance system.
5. The emergency overflow shall be stabilized with riprap or other approved means and shall extend to the toe of each face of the berm embankment.

D. Access/Maintenance

1. Pond access easements and roads shall be provided when the ponds do not abut public right-of-way. Access roads shall provide access to the control structure and along one or both sides of pond as necessary for vehicular maintenance and as determined by the City Engineer or representative.
2. Access roads shall have a maximum grade of 15 percent and a maximum cross slope of 3 percent.
3. 40 foot minimum outside turning radius shall be provided on the access road.
4. Access roads shall be a minimum of 15 feet wide on curved sections and 12 feet wide on straight sections.
5. Access roads less than 400 feet in length shall have a turn around unless approved by the City Engineer or representative.
6. Access roads shall be designed and constructed to support a 20-ton vehicle under all weather conditions.
7. The roads shall be constructed of gravel, crushed rock, or asphalt.

E. Slope Stabilization (Detention ponds)

1. Pond berm embankment higher than six-feet shall be designed by a geotechnical engineer.
2. The berm embankment shall have a minimum 15-foot top width, where necessary, for maintenance access; otherwise, top width may vary as recommended by the design engineer, but in no case shall top width be less than four feet.
3. The toe of the exterior slope of the pond berm shall be no closer than five feet from the tract or easement property line.
4. The pond berm embankment shall be constructed on native consolidated (or adequately compacted and stable fill soils analyzed by a geotechnical engineer) free of loose surface soil materials, roots and other organic debris.
5. The pond berm embankments shall be constructed by excavating a 'key' equal to 50 percent of the berm embankment cross-sectional height and width or as designed by a geotechnical engineer.
6. The berm embankment shall be constructed on compacted soil (95 percent minimum dry density per AASHTO T99, placed in 6-inch lifts, with the following characteristics:

2. Dry Pond Stormwater Quality Facilities

Sizing of dry ponds is based on hydrologic routing of the Water Quality Design Storm. Sizing factors, such as those provided for Vegetated Water Quality Facilities, are not appropriate for ponds due to variations in design parameters. When used to meet both the stormwater quality and detention requirements, dry ponds must also meet applicable flow control requirements as described in Section 6.3.

a. Dry ponds are designed to fill during storm events and either infiltrate, or slowly release the Water Quality Design Storm volume through an underdrain system after filtering through soil media. When constructed to also serve as detention, volumes greater than the water quality storm volume are routed through an outlet control structure.

b. Dry ponds are divided into two cells: a forebay cell and a treatment cell. A pre-treatment manhole is required ahead of the forebay. The pretreatment manhole and the forebay cell serve to extend the life and efficiency of the treatment cell by removing readily settleable debris and floatable debris and oils.

c. The inlet to the pond must extend from the pretreatment manhole to the forebay cell with the invert elevation at the outfall located above the sediment storage elevation. Outlet protection must be provided.

d. The forebay should contain approximately 10% of the design surface area and provide a minimum 6-inch depth of dead storage for sediment accumulation. Dead storage volume may not be included in design as available storage for treatment (or detention) volumes.

e. Both the forebay and treatment cell bottoms are comprised of an 18-inch layer of growing medium over a 3-inch gravel lens layer and 12 inches of drain rock. The growing medium, but not the rock, must also be placed on the side slopes of the facility extending up to the water quality event design surface elevation. On the facility side slopes above the water quality design surface elevation, 12 inches of topsoil must be placed up to the top of the bank, and beyond to the extent necessary to support required/proposed plantings.

f. A rock energy dissipater must separate the forebay from the treatment cell. The top of the rock energy dissipater must be set at 12 inches below the Water Quality Design Storm water surface and have maximum side slopes of 2H:1V. The energy dissipater must be designed using published references such as Hydraulic Design of Energy Dissipaters for Culverts and Channels published by the FHA or the ODOT Hydraulics Manual. The design reference must be cited on the construction plan submittal. The energy dissipater must be designed to be stable for all anticipated hydraulic conditions. The energy dissipater rock gradation must be sufficient to allow for minimum head differential between the forebay and the treatment cell during the draining cycle of the pond while also allowing for deposition and retention of larger sediment to the forebay.

g. For unlined facilities not dependent on infiltration for managing the Water Quality Design Storm, and for lined facilities, a perforated pipe in the drain rock along with the surface overflow system collect stormwater and direct it to the storm drain system. The design must assume a 3-inch-per-hour infiltration rate through the growing medium. The required number and spacing of drain pipe must be designed assuming no infiltration is occurring into native soils such that the underdrain system is the primary means of

conveyance for the water quality storm. For unlined facilities not dependent on infiltration for managing the Water Quality Design Storm, the bottom of the perforated pipe should be elevated 2.5 inches above the bottom of the facility/drain rock. For lined facilities, the perforated pipe should be placed at the bottom of the drain rock.

h. If a lined facility is proposed with less than 3 ft of separation from the seasonal high groundwater table, the liner must be designed to resist buoyant forces.

i. Underdrain systems may not be used for facilities that depend on infiltration for managing the Water Quality Design Storm or be used to meet detention requirements beyond the Water Quality Design Storm.

j. Underdrain systems can have many different configurations; pipe sizing and lateral spacing must be based on facility size and individual site conditions subject to the following minimum requirements:
   - Minimum perforated pipe size is 4-inch-diameter.
   - Underdrain laterals can be no more than 10-ft-on-center-spacing; at minimum provide one underdrain for every 1,000 ft^2 of surface area.
   - Include at least two cleanouts for each underdrain lateral, one at the upstream end and one at the downstream end.
   - Piping must conform to the requirements of the Oregon Plumbing Specialty Code.

k. The soil surface on the bottom of the facility must be generally level to promote infiltration of stormwater throughout the entire length of the facility. The elevation of the soil surface must be identified on the construction plans.

l. The minimum freeboard in ponds is 1-ft above the emergency overflow structure or spillway elevation.

m. Dry ponds may have a maximum water depth of 4 ft. The minimum width at the bottom of the pond is 4 ft. The width of the pond must vary by 4 ft at a minimum of two separate points to produce a more natural pond shape.

n. Side slopes for dry ponds should not exceed a slope of 3H:1V. Moderately undulating slopes are acceptable and can provide a more natural setting for the facility. Gentle side slopes improve the aesthetic attributes of the facility and enhance safety.
   - For public facilities, interior sides slopes must be no steeper than 3H:1V, even if fenced, to minimize safety risks. Access to the side slopes must be provided for maintenance. For private facilities, the same is recommended rather than required.
   - Exterior side slopes for public or private facilities may not be steeper than 2H:1V unless confirmed stable by a geotechnical engineer.
   - The maximum water depth of any pond is 4 ft.

See Section 6.3 (Stormwater Detention), Appendix D and related Construction Specifications for additional design requirements for dry ponds.

3. Manufactured Facilities

Manufactured facilities must be approved by the City Engineer. The treatment device must be designed and installed per the manufacturer's recommendations. Since these facilities may be constructed on private property, land use approvals and building permits may be required.

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

You have zoomed in beyond the scale at which the soil map for this area is intended to be used. Mapping of soils is done at a particular scale. The soil surveys that comprise your AOI were mapped at 1:24,000. The design of map units and the level of detail shown in the resulting soil map are dependent on that map 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.

Tables — Hydrologic Soil Group — Summary By Map Unit

Summary by Map Unit — Benton County, Oregon (OR003)

Summary by Map Unit — Benton County, Oregon (OR003)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
60	Dixonville-Gellatly-Witham complex, 2 to 12 percent slopes	D	56.8	51.4%
91	Jory silty clay loam, basalt bedrock, 2 to 12 percent slopes	C	0.0	0.0%
113	McAlpin silty clay loam, 0 to 3 percent slopes	C/D	18.5	16.8%
114	McAlpin silty clay loam, 3 to 6 percent slopes	C/D	19.8	17.9%
155	Waldo silty clay loam, 0 to 3 percent slopes	C/D	0.3	0.3%
159	Water		0.3	0.3%
170	Willamette silt loam, 3 to 12 percent slopes	B	0.6	0.5%
178	Woodburn silt loam, 3 to 12 percent slopes	C	9.3	8.4%
2205A	Conser silty clay loam, 0 to 3 percent slopes	D	4.8	4.4%
Totals for Area of Interest			**110.3**	**100.0%**