EMERIO
Design

CIVIL ENGINEERS & PLANNERS

Stormwater Management Report
Bland Circle Subdivision
15-Lot Subdivision at
23000 & 23010 Bland Circle
West Linn, Oregon

Emerio Project Number:	0540-001
City of West Linn Permit Numbers:	TBD
Date:	07/08/2019

REGISTERED PROFESSIONAL ENGINEER
54665
OREGON
JULY 11, 2006
ERIC DANIEL EVANS
EXPIRES: 12-31-19

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Table of Contents:

Project Overview and Description:

Size and location of project site (vicinity map): The current site is located in the south part of West Linn on the east side of Bland Circle, approximately 200 feet north of the intersection of Fircrest Drive & Bland Circle. Three lots will be divided into fifteen lots. The proposed site is 3.45 acres and will encompass roughly 60,689 SF of impervious onsite improvements and 3,917 SF offsite impervious improvement. Reference the vicinity map provided in Appendix A(1).

Property Zoning: The property is zoned R7 (Residential 7,000 SF lots).

Type of Development/Proposed Improvements: The proposed development will consist of a public street, and new homes and driveways will be constructed on each lot.

Existing vs. post-construction conditions: the current (existing) site condition consists of under-developed forested/vegetated lots with two houses, attached garages, and their associated driveways and outbuildings.

Watershed Description: The site drainage area presently sheet flows west toward Bland Circle and into the existing stormwater system in Bland Circle flowing south. In the post-developed condition, the site impervious flows will be treated onsite and discharged into the stormwater pipe that will connect to the existing stormwater pipe system in Bland Circle. Drainage basin areas are shown in Appendix D(2).

Soil Classification:

The NRCS soil survey of Clackamas County, Oregon classifies the onsite soils as Jory silty clay loam, Nekia silty clay loam, and Saum loam. The associated hydrologic group of these soils is C, see Appendix B(1). A curve number of 74 is used for pre-developed pervious surfaces and 98 and 86 are used for impervious and pervious surfaces.

Methodology:

To satisfy stormwater requirements, a water quality/quantity pond will be used. The pond will be placed at the southwest portion of the site adjacent to Bland Circle.

The pond will provide water quality for the proposed right-of-way (ROW) and all lots. Detention for the site will be provided by the storage volume of the proposed onsite pond. The proposed grading will retain the general existing drainage pattern for pervious areas of the site. An upstream basin currently flowing south along the Bland Circle frontage of the proposed project will be routed through the stormwater facility; however, detention will only be provided for the increased runoff due to development of this project and some right-of-way improvements.

Water Quality

The proposed pond is based on the standard City of Portland detention pond and the Water Environment Services (WES) standard detail SWM FC-6.0 (see Appendix D(3)). Stormwater runoff will enter the pond, slowly filter down through an 18" layer of amended soil before reaching a perforated pipe within a 12" section of drain rock to be routed to the orifice control structure for the pond. The peak water surface elevation during the water quality storm for the pond is below the first ditch inlet; therefore, the volume of runoff during the water quality design storm will be fully treated (Appendix C(2)).

Quantity Control/Detention

As required by the City of West Linn, detention will be provided for the 2, 5, 10, and 25-year design storms within the pond. The pond volumes are shown in Table 1:

Table 1: Extended Wet Pond Storage		
Elevation (ft)	Area (SF)	Cumulative Detention Storage (CF)
530.5	2,208	0
531.0	2,609	1,204
532	3,015	4,016
533	3,075	7,061
534	4,035	10,616
535	4,095	14,681

Table 2: Total Site Detention Peak Flow Rates		
Return Period	Pre-Developed Site Discharge (CFS)	Post-Developed Site Discharge (CFS)
2-Year	0.87	0.87
5-Year	1.53	1.51
10-Year	2.11	2.11
25-Year	2.90	2.74

The outflow rate of the pond is controlled for the 2-year through 25-year design storm events via two orifices in a flow control structure: a 4.5" diameter orifice set at an elevation of 527.82' and an 8.7" orifice set at an elevation of 531.69'. Both orifices are set in an orifice plate between two ditch inlets per City of West Linn standard drawing number WL-610 and 611. The first ditch inlet is set at the peak water surface elevation of the water quality design storm. The second ditch inlet is set at 532.80' in the event of flows greater than the 25-year design storm. The pond will have a minimum of 1 foot of freeboard above the 25-year peak water surface elevation.

In the event of flows during the 100-year storm event and/or failure of the flow control and secondary ditch inlet structures, an emergency overflow manhole with a metal cage trash rack set at the 25-year peak water surface elevation will allow for conveyance of the pond. See conveyance section this page.

As shown in the Table 2, the water quality/quantity pond will limit the post-developed peak flow rates to the pre-developed peak flow rates for corresponding storm events.

Stormwater Conveyance

Onsite conveyance will be by means of 12" storm water pipe in Satter Street routing all the way to the discharge point in the existing utility easement south of this site. For conservatism, the total post-developed flow rate with no detention within proposed stormwater pipe was used to analyze the lowest potential pipe design slope at 0.5% during the conveyance design storm. See Appendix C(4) for HydroCAD flow rates developed during the 100-year 24-hr conveyance design storm event.

Analysis:

The following design assumptions were utilized in this design.

Design Storm:	Water quality storm = **0.83" in 24 hours**
	2-year 24-hour storm **= 2.5" in 24 hours**
	5-year 24-hour storm **= 3.0" in 24 hours**
	10-year 24-hour storm **= 3.4" in 24 hours**
	25-year 24-hour storm **= 3.9" in 24 hours**
	100-year 24-hour storm **= 4.5" in 24 hours**

Computation methods and software utilized in the design were from HydroCAD V-10.

Curve numbers utilized in the design were 98 for impervious areas, 86 for pervious areas, and 74 for predeveloped pervious areas.

Engineering Conclusions:

The design of the proposed stormwater management facilities satisfies the pollution reduction, conveyance and detention standards required by the 2010 City of West Linn Public Works Design Standards.

Appendix A:

Appendix A(1)
Vicinity Map

Appendix B:

Appendix B(1)
Soil Classification

Tables — Hydrologic Soil Group — Summary By Map Unit

Summary by Map Unit — Clackamas County Area, Oregon (OR610)

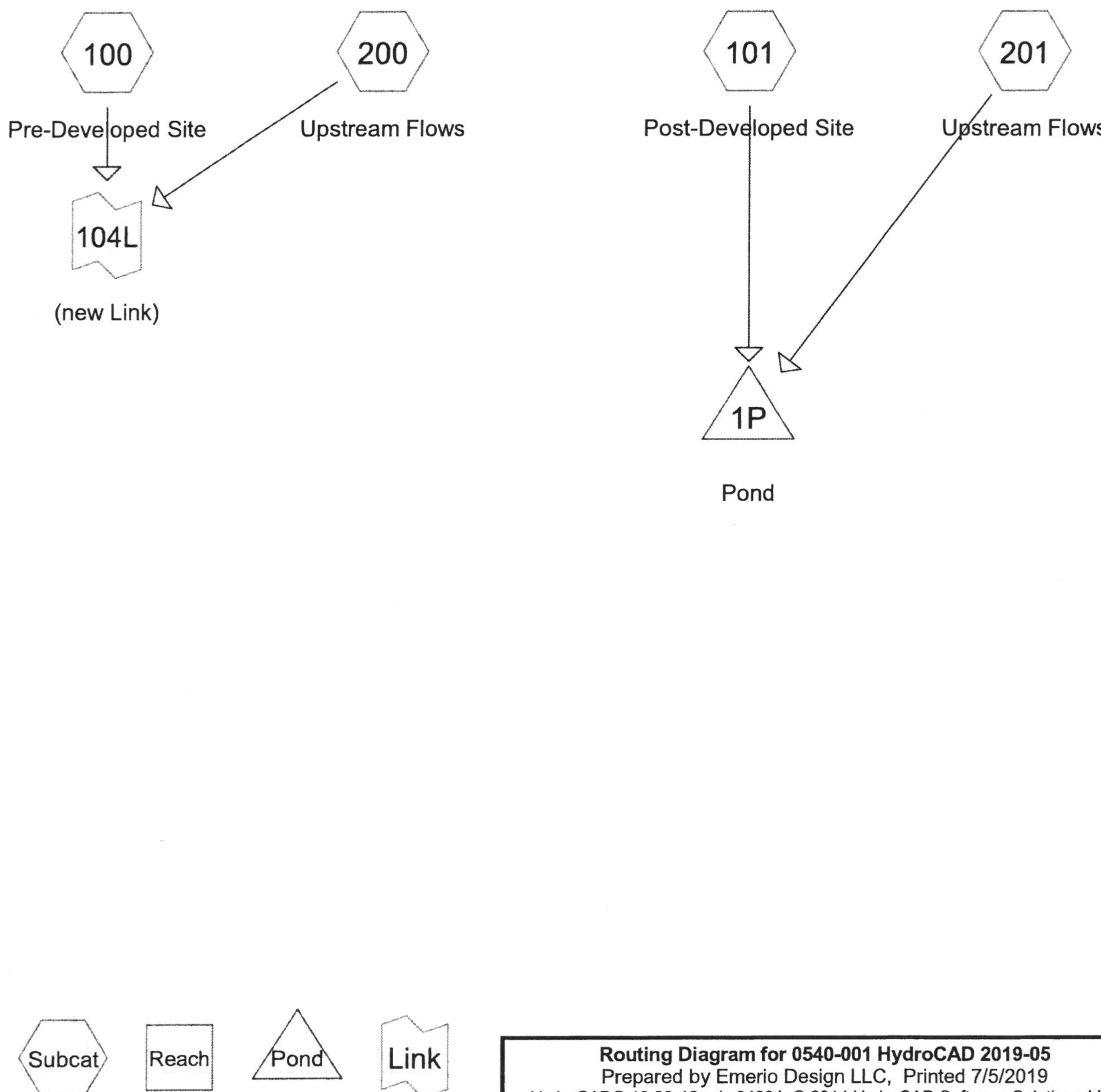
Summary by Map Unit — Clackamas County Area, Oregon (OR610)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
45C	Jory silty clay loam, 8 to 15 percent slopes	C	2.4	39.2%
64C	Nekia silty clay loam, 8 to 15 percent slopes	C	1.1	17.3%
78B	Saum silt loam, 3 to 8 percent slopes	C	0.6	10.5%
78C	Saum silt loam, 8 to 15 percent slopes	C	2.0	33.1%
Totals for Area of Interest			**6.2**	**100.0%**

Appendix C:

Basin Area Tabulated Data
23000 Bland Circle

Basin #	Name	Total Area SF	Total Area Acres	Qty of Lots	Lot Impervious SF	ROW/Tract Imp SF	Total Impervious SF	Total Pervious (Calc'd) SF
100	Pre-Developed Onsite	151,892	3.49	1	2,500	0	2,500	149,392
101	Post-Developed Onsite	151,892	3.49	15	37,500	20,996	58,496	93,396
200	Upstream Flows	258,223	5.93	3	7,500	14,005	21,505	236,718



Routing Diagram for 0540-001 HydroCAD 2019-05
Prepared by Emerio Design LLC, Printed 7/5/2019
HydroCAD® 10.00-13 s/n 04804 © 2014 HydroCAD Software Solutions LLC

Summary for Subcatchment 100: Pre-Developed Site

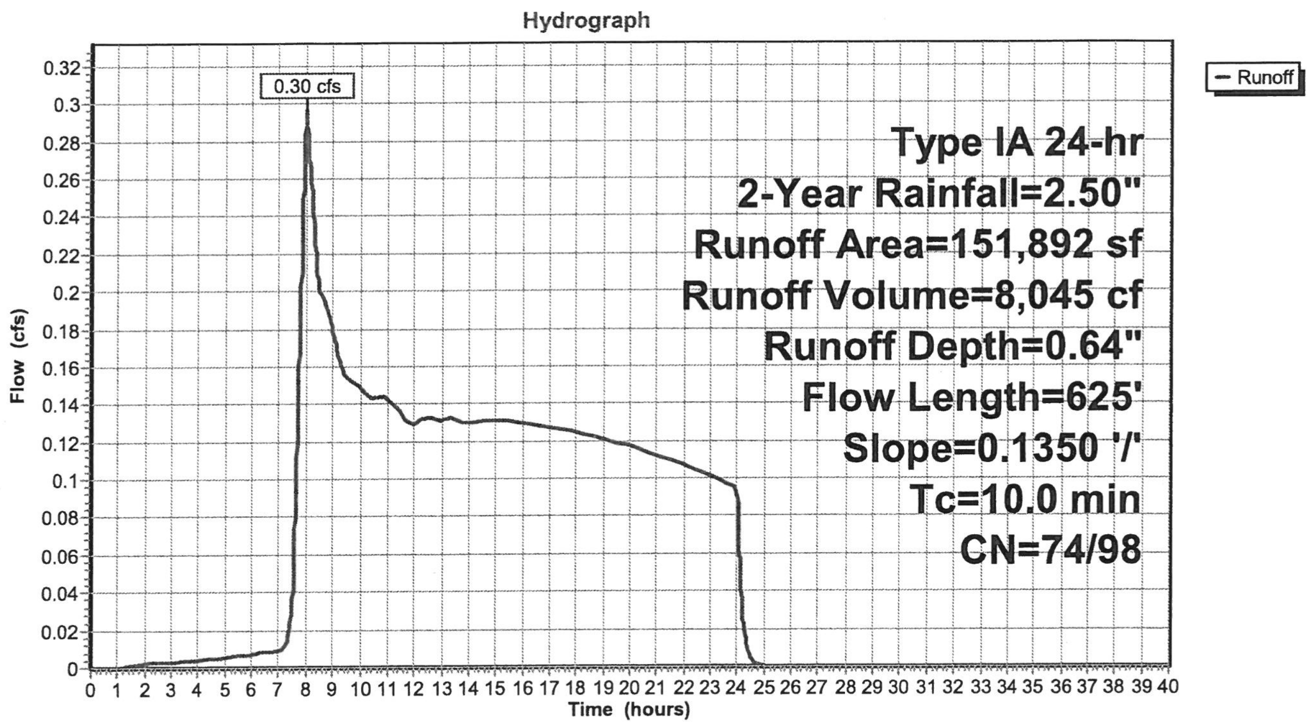
Runoff = 0.30 cfs @ 8.01 hrs, Volume= 8,045 cf, Depth= 0.64"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-Year Rainfall=2.50"

	Area (sf)	CN	Description
*	2,500	98	roofs
	149,392	74	>75% Grass cover, Good, HSG C
	151,892	74	Weighted Average
	149,392	74	98.35% Pervious Area
	2,500	98	1.65% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.2	100	0.1350	0.32		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.50"
4.8	525	0.1350	1.84		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
10.0	625	Total			

Subcatchment 100: Pre-Developed Site



Summary for Subcatchment 101: Post-Developed Site

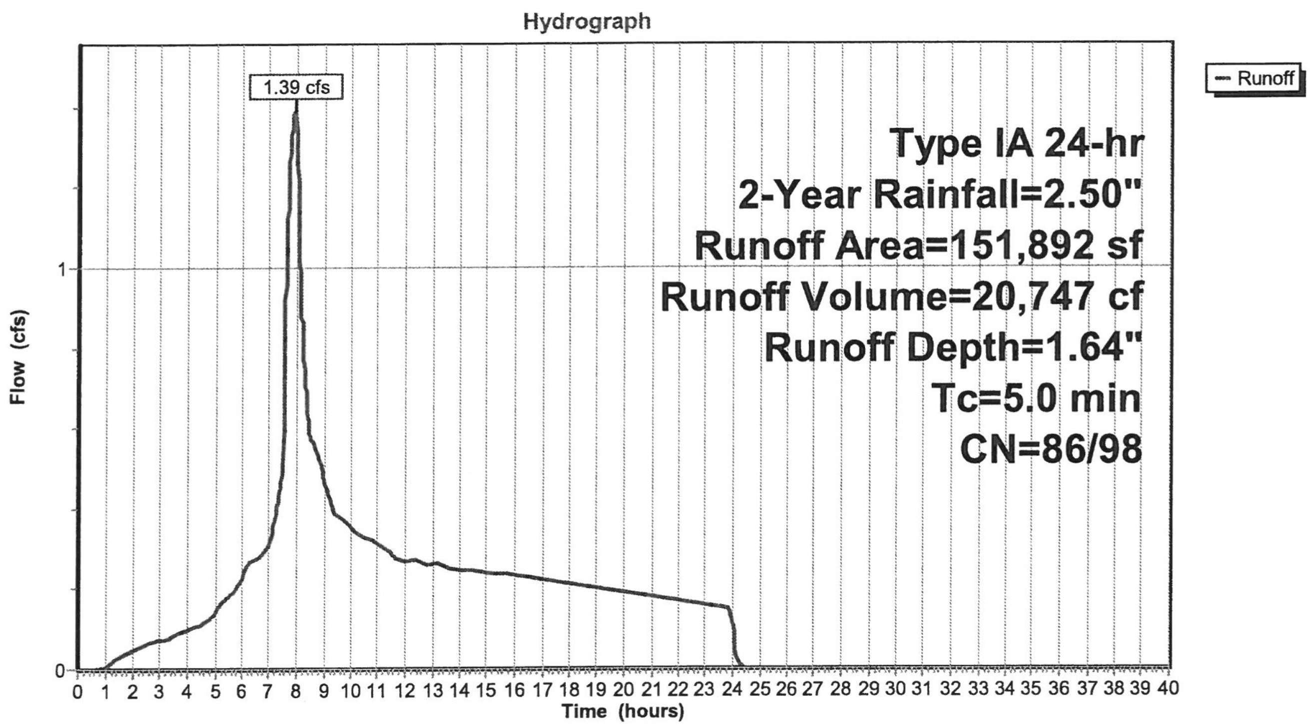
Runoff = 1.39 cfs @ 7.92 hrs, Volume= 20,747 cf, Depth= 1.64"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-Year Rainfall=2.50"

	Area (sf)	CN	Description
*	37,500	98	lots
*	7,602	98	sidewalks
*	13,394	98	ROW
	93,396	86	<50% Grass cover, Poor, HSG C
	151,892	91	Weighted Average
	93,396	86	61.49% Pervious Area
	58,496	98	38.51% Impervious Area

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

Subcatchment 101: Post-Developed Site



Summary for Subcatchment 200: Upstream Flows

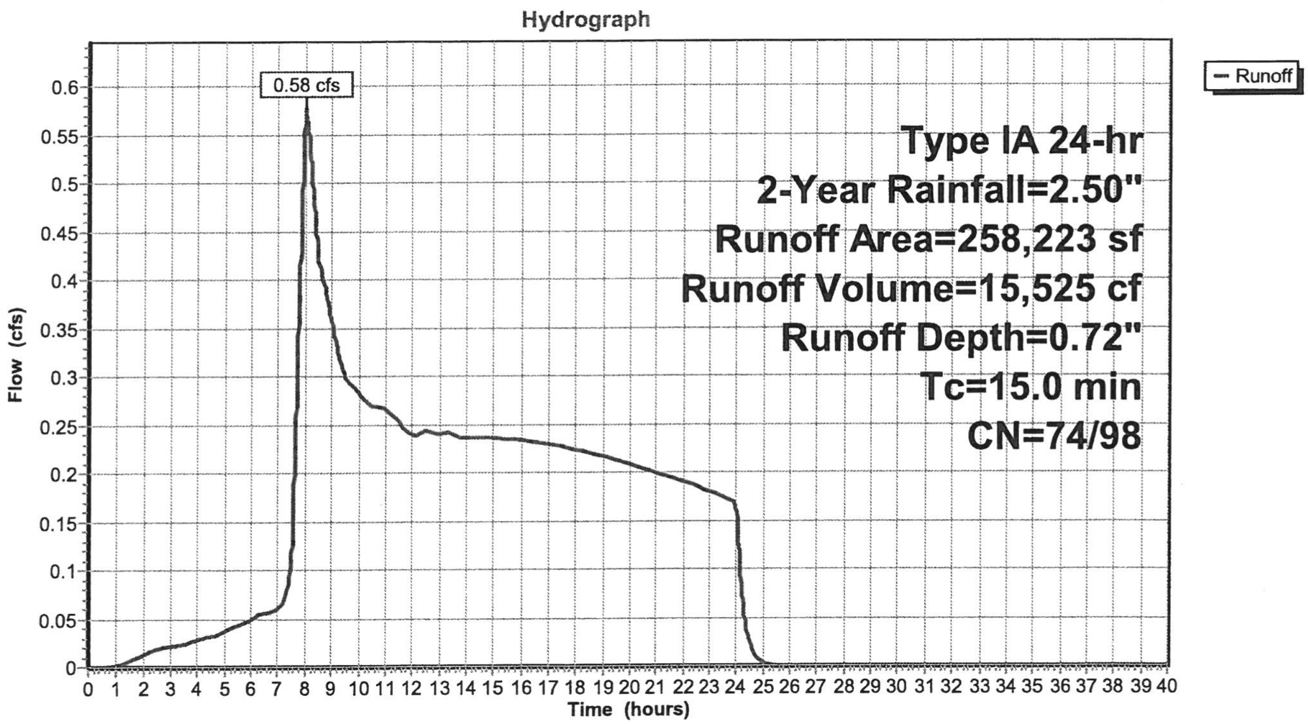
Runoff = 0.58 cfs @ 8.01 hrs, Volume= 15,525 cf, Depth= 0.72"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-Year Rainfall=2.50"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	240,635	74	>75% Grass cover, Good, HSG C
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	240,635	74	93.19% Pervious Area
	17,588	98	6.81% Impervious Area

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

Subcatchment 200: Upstream Flows



Summary for Subcatchment 201: Upstream Flows

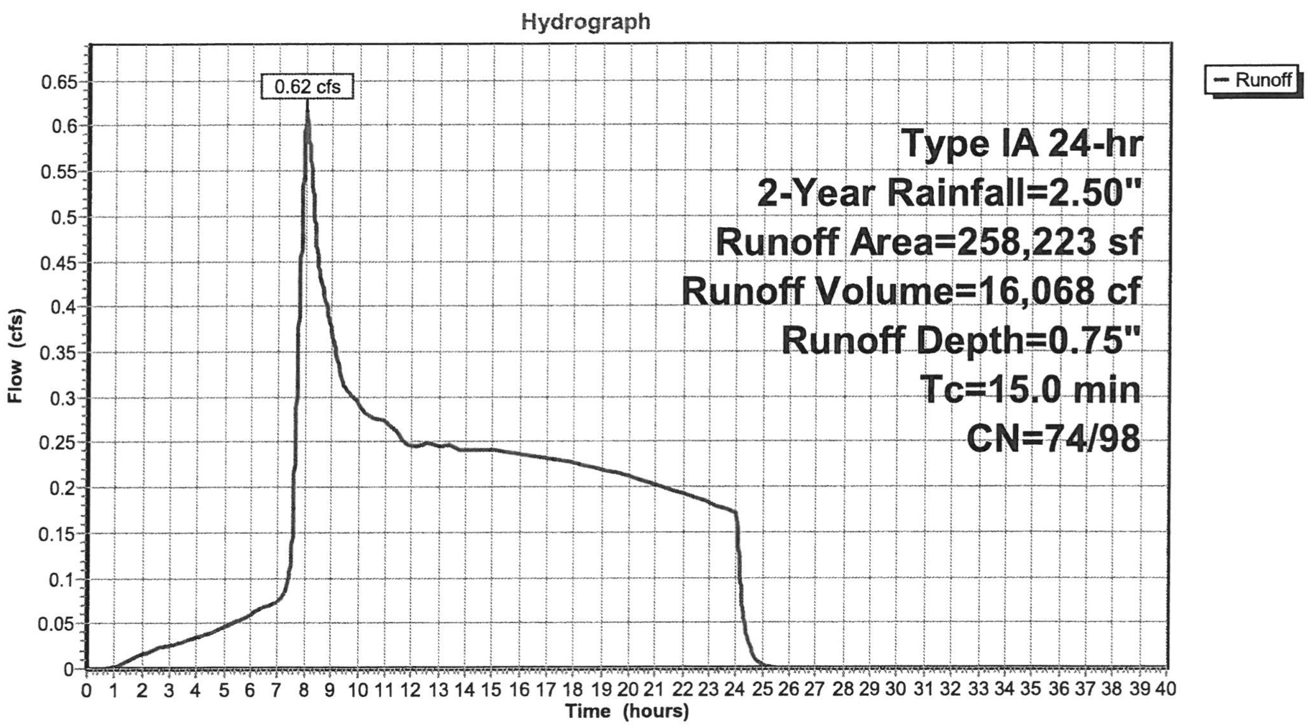
Runoff = 0.62 cfs @ 8.01 hrs, Volume= 16,068 cf, Depth= 0.75"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 2-Year Rainfall=2.50"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	236,718	74	>75% Grass cover, Good, HSG C
*	3,917	98	frontage new ROW
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	236,718	74	91.67% Pervious Area
	21,505	98	8.33% Impervious Area

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

Subcatchment 201: Upstream Flows



Summary for Pond 1P: Pond

Inflow Area = 410,115 sf, 19.51% Impervious, Inflow Depth = 1.08" for 2-Year event
Inflow = 1.98 cfs @ 8.00 hrs, Volume= 36,815 cf
Outflow = 0.87 cfs @ 8.95 hrs, Volume= 36,815 cf, Atten= 56%, Lag= 57.5 min
Primary = 0.87 cfs @ 8.95 hrs, Volume= 36,815 cf
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0 cf
Tertiary = 0.00 cfs @ 0.00 hrs, Volume= 0 cf

Routing by Stor-Ind method, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Peak Elev= 531.77' @ 8.95 hrs Surf.Area= 2,921 sf Storage= 3,332 cf

Plug-Flow detention time= 35.1 min calculated for 36,806 cf (100% of inflow)
Center-of-Mass det. time= 35.1 min (820.5 - 785.4)

Volume	Invert	Avail.Storage	Storage Description
#1	530.50'	14,681 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
530.50	2,208	0	0
531.00	2,609	1,204	1,204
532.00	3,015	2,812	4,016
533.00	3,075	3,045	7,061
534.00	4,035	3,555	10,616
535.00	4,095	4,065	14,681

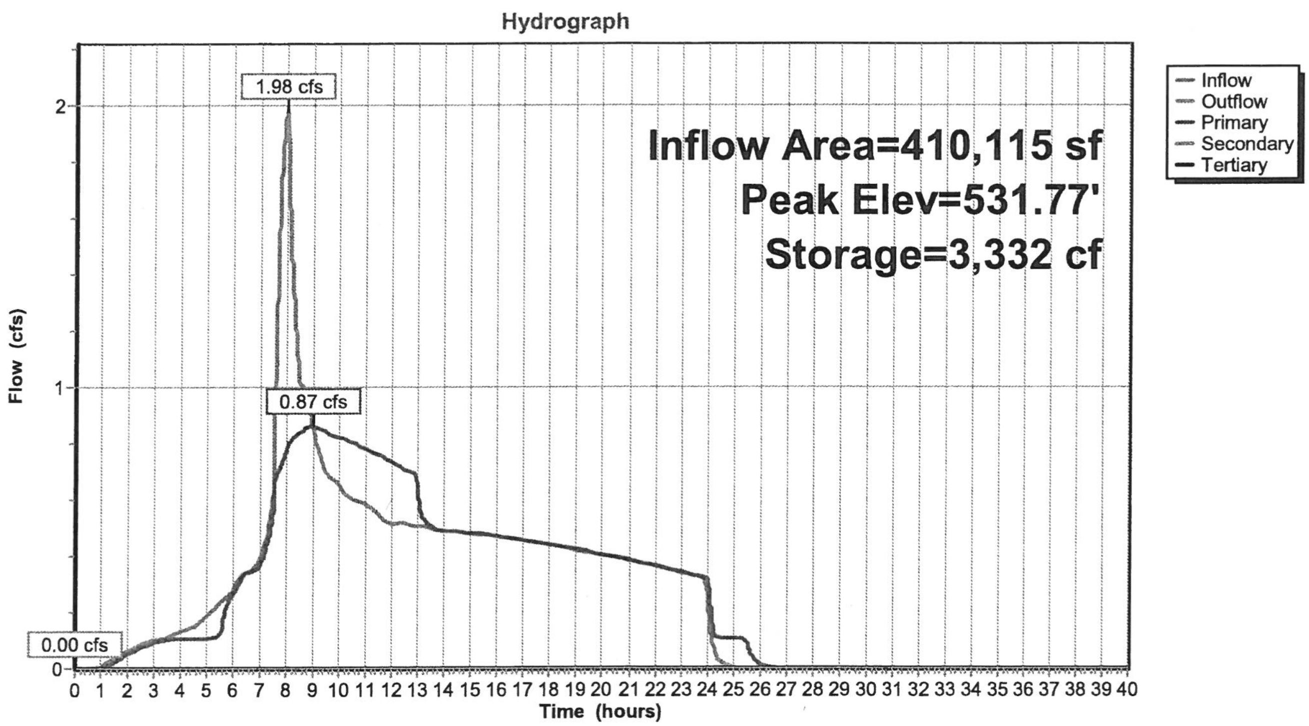
Device	Routing	Invert	Outlet Devices
#1	Primary	529.00'	**15.0" Round Culvert** L= 20.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 529.00' / 528.90' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.23 sf
#2	Device 1	527.82'	**4.4" Vert. Low Orifice** C= 0.600
#3	Device 2	530.50'	**2.000 in/hr Exfiltration over Surface area**
#4	Device 2	530.72'	**24.0" x 24.0" Horiz. Ditch Inlet #1** C= 0.600 Limited to weir flow at low heads
#5	Device 1	531.70'	**8.6" Vert. High Orifice** C= 0.600
#6	Secondary	532.87'	**24.0" x 24.0" Horiz. Ditch Inlet #2** C= 0.600 Limited to weir flow at low heads
#7	Tertiary	532.87'	**48.0" Horiz. Overflow Manhole** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.86 cfs @ 8.95 hrs HW=531.77' (Free Discharge)
└─**1=Culvert** (Passes 0.86 cfs of 8.65 cfs potential flow)
  └─**2=Low Orifice** (Orifice Controls 0.85 cfs @ 8.01 fps)
    └─**3=Exfiltration** (Passes < 0.14 cfs potential flow)
    └─**4=Ditch Inlet #1** (Passes < 19.73 cfs potential flow)
  └─**5=High Orifice** (Orifice Controls 0.02 cfs @ 0.90 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=530.50' (Free Discharge)
└─**6=Ditch Inlet #2** (Controls 0.00 cfs)

Tertiary OutFlow Max=0.00 cfs @ 0.00 hrs HW=530.50' (Free Discharge)
└─**7=Overflow Manhole** (Controls 0.00 cfs)

Pond 1P: Pond

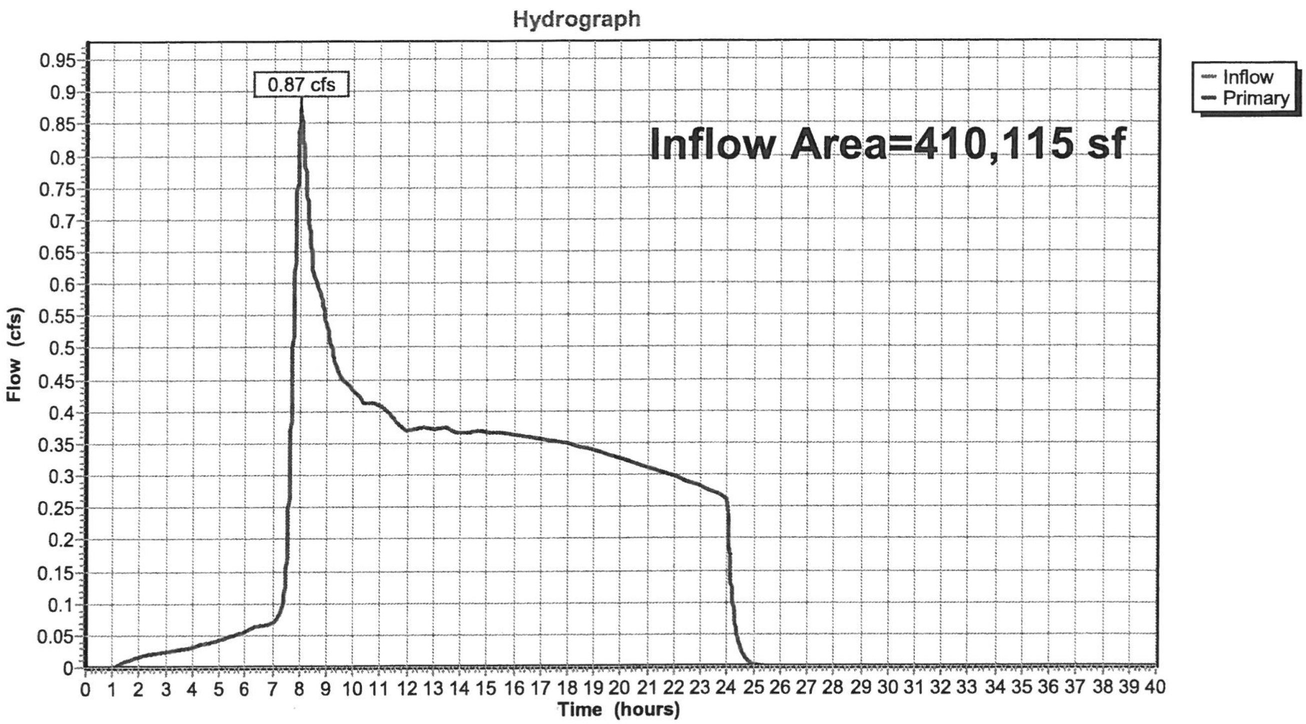


Summary for Link 104L: (new Link)

Inflow Area =	410,115 sf,	4.90% Impervious,	Inflow Depth = 0.69"	for 2-Year event
Inflow =	0.87 cfs @	8.01 hrs, Volume=	23,571 cf	
Primary =	0.87 cfs @	8.01 hrs, Volume=	23,571 cf, Atten= 0%, Lag= 0.0 min	

Primary outflow = Inflow, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs

Link 104L: (new Link)



Summary for Subcatchment 100: Pre-Developed Site

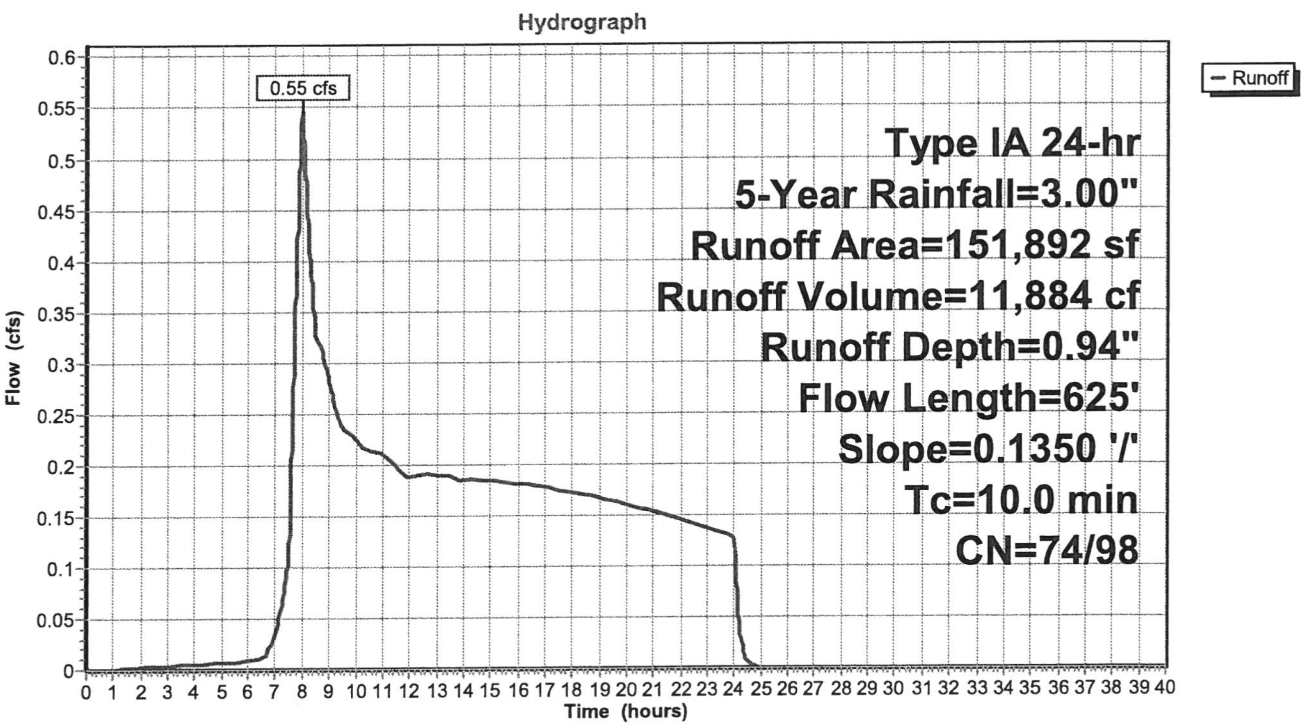
Runoff = 0.55 cfs @ 8.00 hrs, Volume= 11,884 cf, Depth= 0.94"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-Year Rainfall=3.00"

	Area (sf)	CN	Description
*	2,500	98	roofs
	149,392	74	>75% Grass cover, Good, HSG C
	151,892	74	Weighted Average
	149,392	74	98.35% Pervious Area
	2,500	98	1.65% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.2	100	0.1350	0.32		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.50"
4.8	525	0.1350	1.84		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
10.0	625	Total			

Subcatchment 100: Pre-Developed Site



Summary for Subcatchment 101: Post-Developed Site

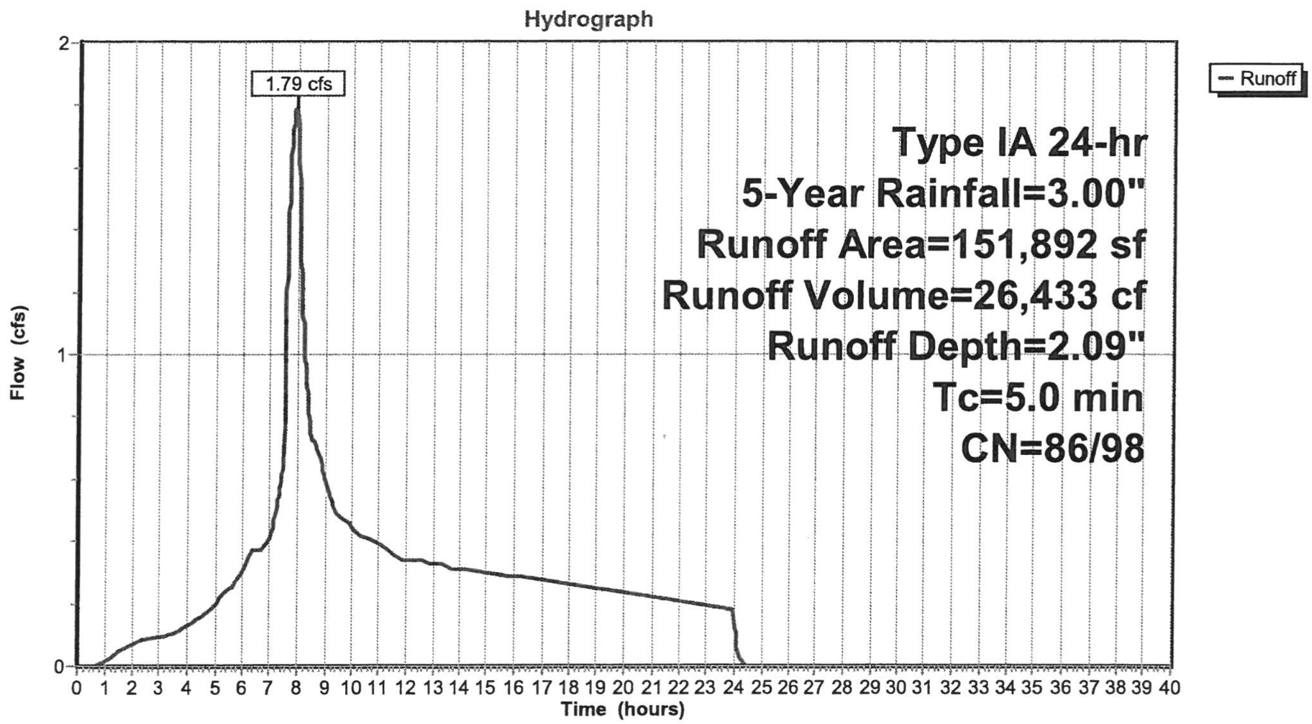
Runoff = 1.79 cfs @ 7.92 hrs, Volume= 26,433 cf, Depth= 2.09"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-Year Rainfall=3.00"

	Area (sf)	CN	Description
*	37,500	98	lots
*	7,602	98	sidewalks
*	13,394	98	ROW
	93,396	86	<50% Grass cover, Poor, HSG C
	151,892	91	Weighted Average
	93,396	86	61.49% Pervious Area
	58,496	98	38.51% Impervious Area

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

Subcatchment 101: Post-Developed Site



Summary for Subcatchment 200: Upstream Flows

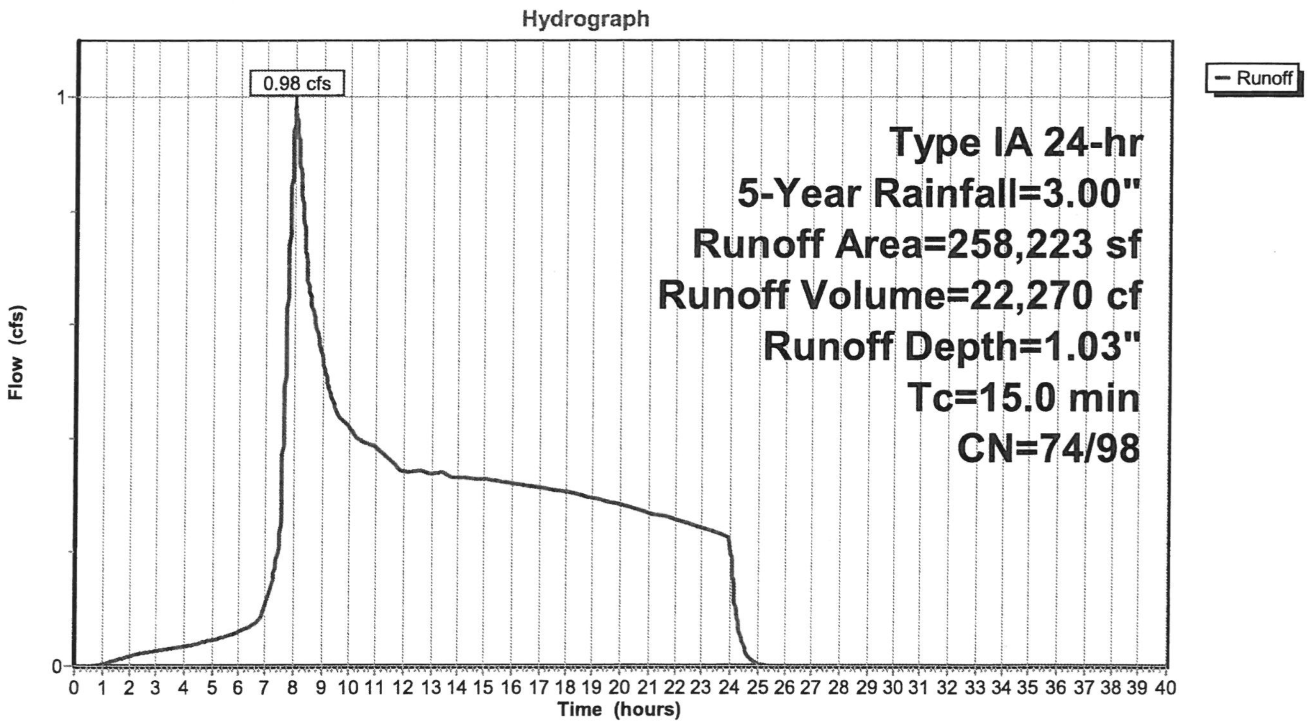
Runoff = 0.98 cfs @ 8.01 hrs, Volume= 22,270 cf, Depth= 1.03"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-Year Rainfall=3.00"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	240,635	74	>75% Grass cover, Good, HSG C
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	240,635	74	93.19% Pervious Area
	17,588	98	6.81% Impervious Area

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

Subcatchment 200: Upstream Flows



Summary for Subcatchment 201: Upstream Flows

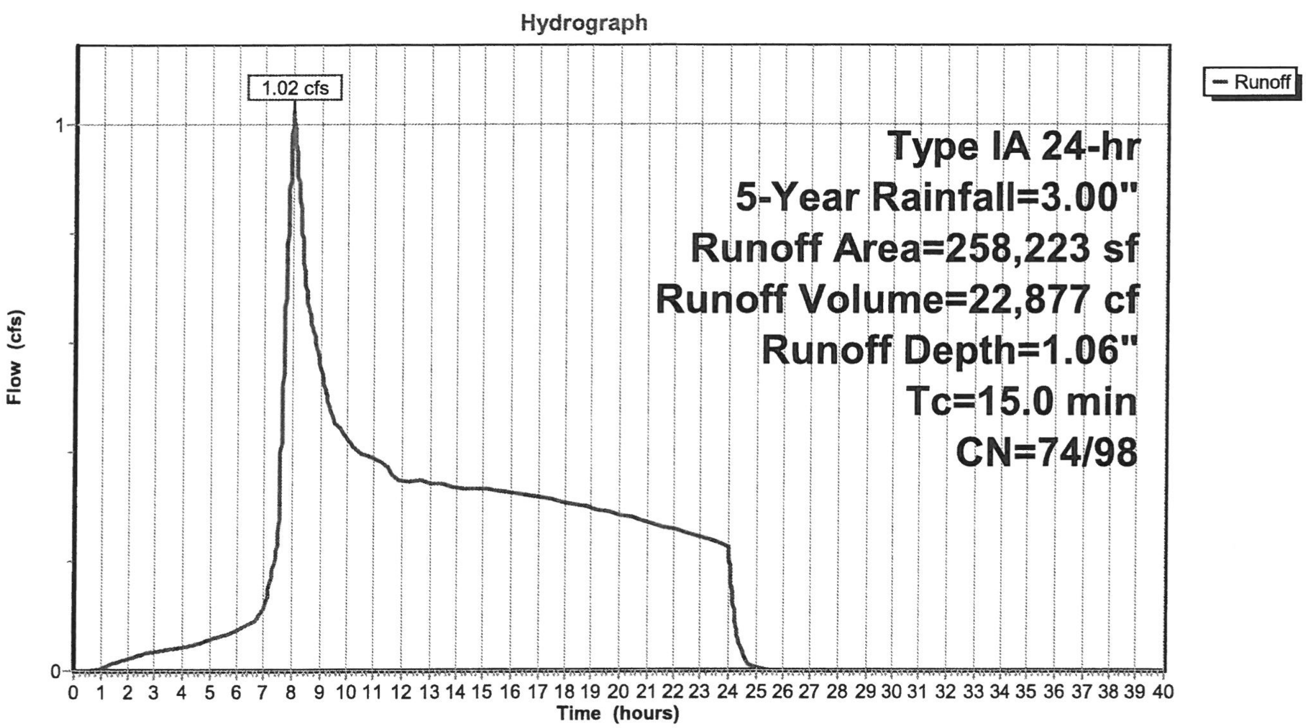
Runoff = 1.02 cfs @ 8.01 hrs, Volume= 22,877 cf, Depth= 1.06"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 5-Year Rainfall=3.00"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	236,718	74	>75% Grass cover, Good, HSG C
*	3,917	98	frontage new ROW
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	236,718	74	91.67% Pervious Area
	21,505	98	8.33% Impervious Area

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

Subcatchment 201: Upstream Flows



Summary for Pond 1P: Pond

Inflow Area =	410,115 sf,	19.51% Impervious,	Inflow Depth = 1.44"	for 5-Year event
Inflow =	2.78 cfs @	8.00 hrs, Volume=	49,311 cf	
Outflow =	1.51 cfs @	8.43 hrs, Volume=	49,311 cf,	Atten= 46%, Lag= 25.5 min
Primary =	1.51 cfs @	8.43 hrs, Volume=	49,311 cf	
Secondary =	0.00 cfs @	0.00 hrs, Volume=	0 cf	
Tertiary =	0.00 cfs @	0.00 hrs, Volume=	0 cf	

Routing by Stor-Ind method, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Peak Elev= 532.15' @ 8.43 hrs Surf.Area= 3,024 sf Storage= 4,470 cf

Plug-Flow detention time= 39.5 min calculated for 49,311 cf (100% of inflow)
Center-of-Mass det. time= 39.5 min (816.5 - 777.0)

Volume	Invert	Avail.Storage	Storage Description
#1	530.50'	14,681 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
530.50	2,208	0	0
531.00	2,609	1,204	1,204
532.00	3,015	2,812	4,016
533.00	3,075	3,045	7,061
534.00	4,035	3,555	10,616
535.00	4,095	4,065	14,681

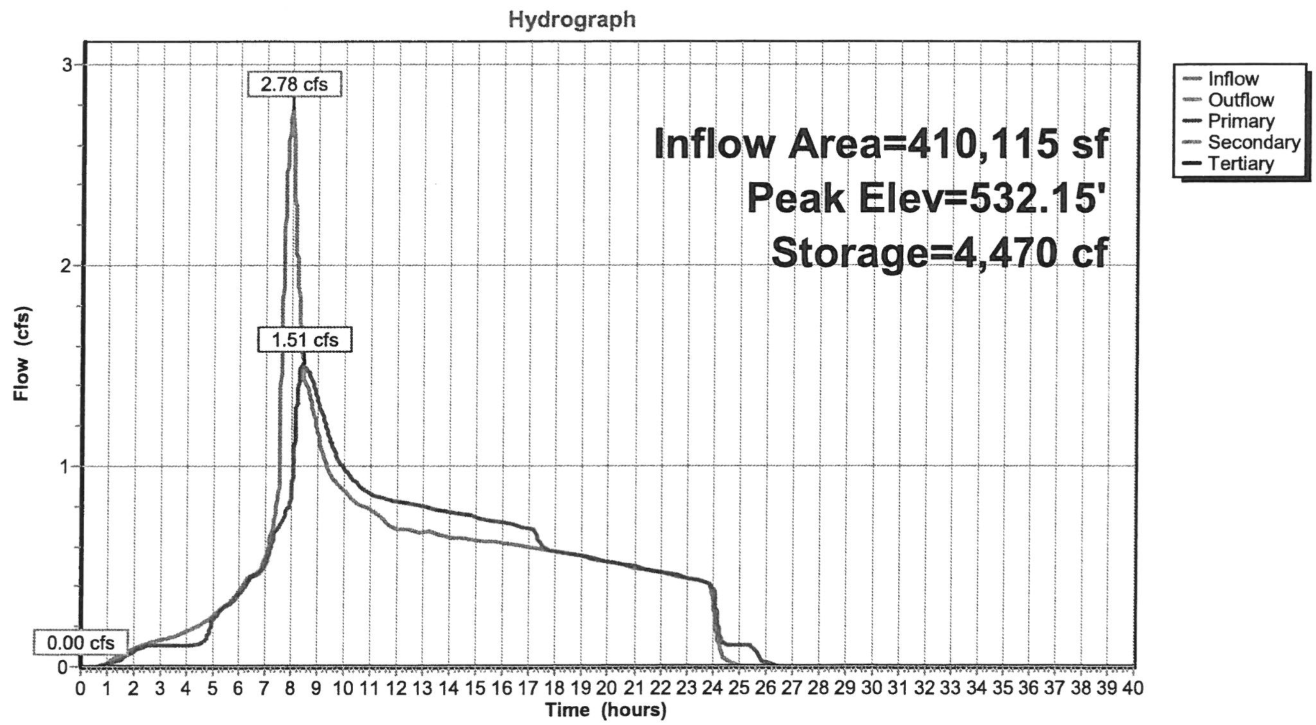
Device	Routing	Invert	Outlet Devices
#1	Primary	529.00'	**15.0" Round Culvert** L= 20.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 529.00' / 528.90' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.23 sf
#2	Device 1	527.82'	**4.4" Vert. Low Orifice** C= 0.600
#3	Device 2	530.50'	**2.000 in/hr Exfiltration over Surface area**
#4	Device 2	530.72'	**24.0" x 24.0" Horiz. Ditch Inlet #1** C= 0.600 Limited to weir flow at low heads
#5	Device 1	531.70'	**8.6" Vert. High Orifice** C= 0.600
#6	Secondary	532.87'	**24.0" x 24.0" Horiz. Ditch Inlet #2** C= 0.600 Limited to weir flow at low heads
#7	Tertiary	532.87'	**48.0" Horiz. Overflow Manhole** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=1.51 cfs @ 8.43 hrs HW=532.15' (Free Discharge)
└─**1=Culvert** (Passes 1.51 cfs of 9.39 cfs potential flow)
  ├─**2=Low Orifice** (Orifice Controls 0.90 cfs @ 8.55 fps)
  │ ├─**3=Exfiltration** (Passes < 0.14 cfs potential flow)
  │ └─**4=Ditch Inlet #1** (Passes < 23.03 cfs potential flow)
  └─**5=High Orifice** (Orifice Controls 0.61 cfs @ 2.28 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=530.50' (Free Discharge)
└─**6=Ditch Inlet #2** (Controls 0.00 cfs)

Tertiary OutFlow Max=0.00 cfs @ 0.00 hrs HW=530.50' (Free Discharge)
└─**7=Overflow Manhole** (Controls 0.00 cfs)

Pond 1P: Pond

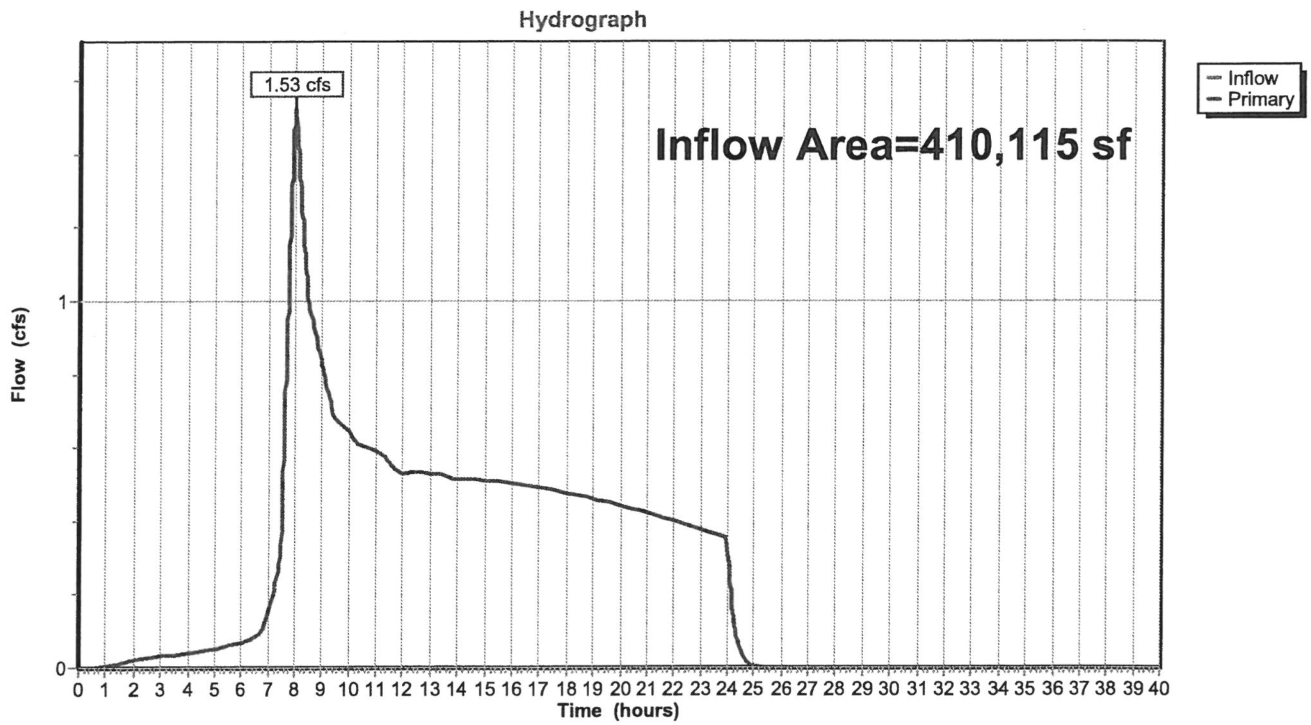


Summary for Link 104L: (new Link)

Inflow Area = 410,115 sf, 4.90% Impervious, Inflow Depth = 1.00" for 5-Year event
Inflow = 1.53 cfs @ 8.01 hrs, Volume= 34,154 cf
Primary = 1.53 cfs @ 8.01 hrs, Volume= 34,154 cf, Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs

Link 104L: (new Link)



Summary for Subcatchment 100: Pre-Developed Site

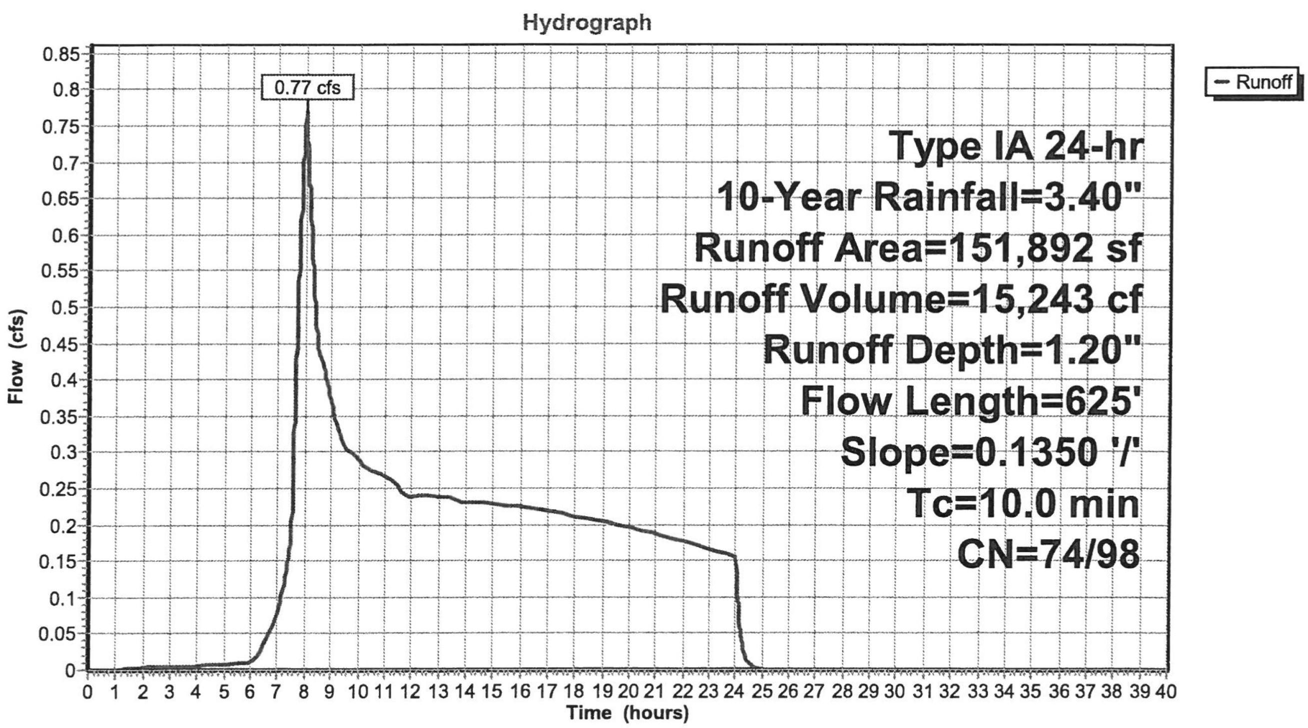
Runoff = 0.77 cfs @ 8.00 hrs, Volume= 15,243 cf, Depth= 1.20"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-Year Rainfall=3.40"

	Area (sf)	CN	Description
*	2,500	98	roofs
	149,392	74	>75% Grass cover, Good, HSG C
	151,892	74	Weighted Average
	149,392	74	98.35% Pervious Area
	2,500	98	1.65% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.2	100	0.1350	0.32		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.50"
4.8	525	0.1350	1.84		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
10.0	625	Total			

Subcatchment 100: Pre-Developed Site



Summary for Subcatchment 101: Post-Developed Site

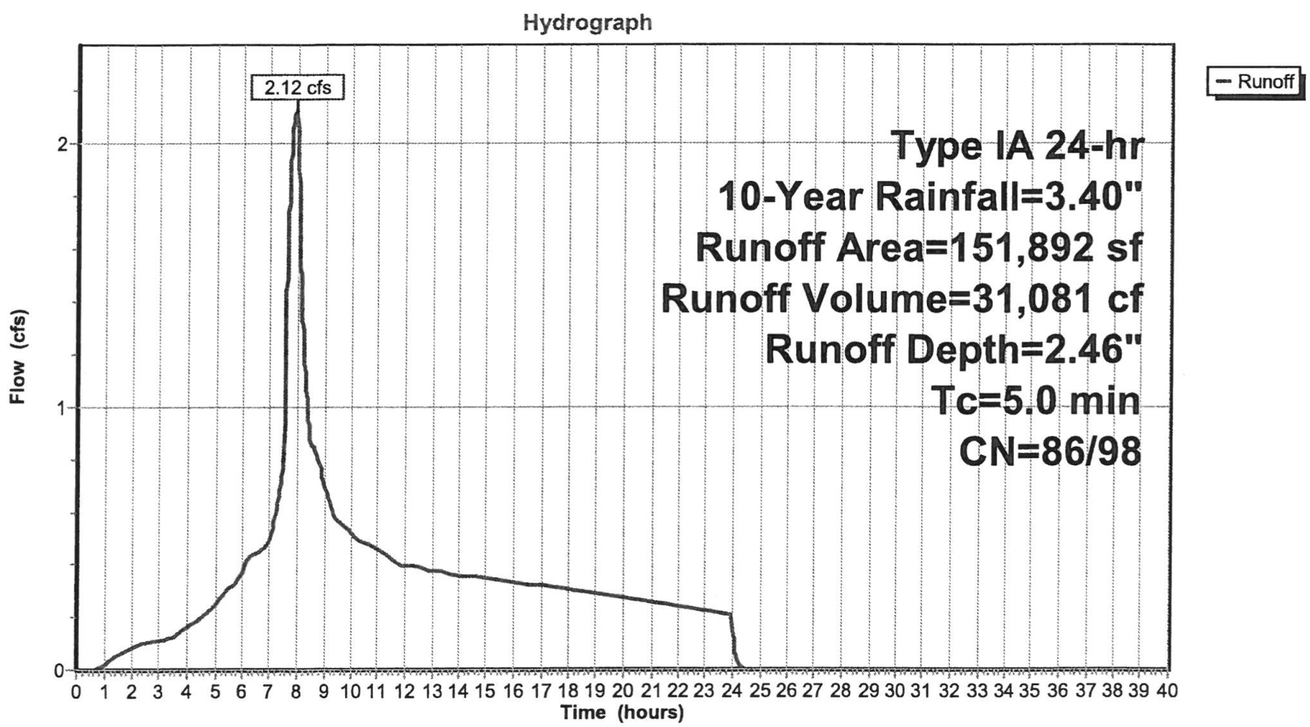
Runoff = 2.12 cfs @ 7.91 hrs, Volume= 31,081 cf, Depth= 2.46"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-Year Rainfall=3.40"

	Area (sf)	CN	Description
*	37,500	98	lots
*	7,602	98	sidewalks
*	13,394	98	ROW
	93,396	86	<50% Grass cover, Poor, HSG C
	151,892	91	Weighted Average
	93,396	86	61.49% Pervious Area
	58,496	98	38.51% Impervious Area

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

Subcatchment 101: Post-Developed Site



Summary for Subcatchment 200: Upstream Flows

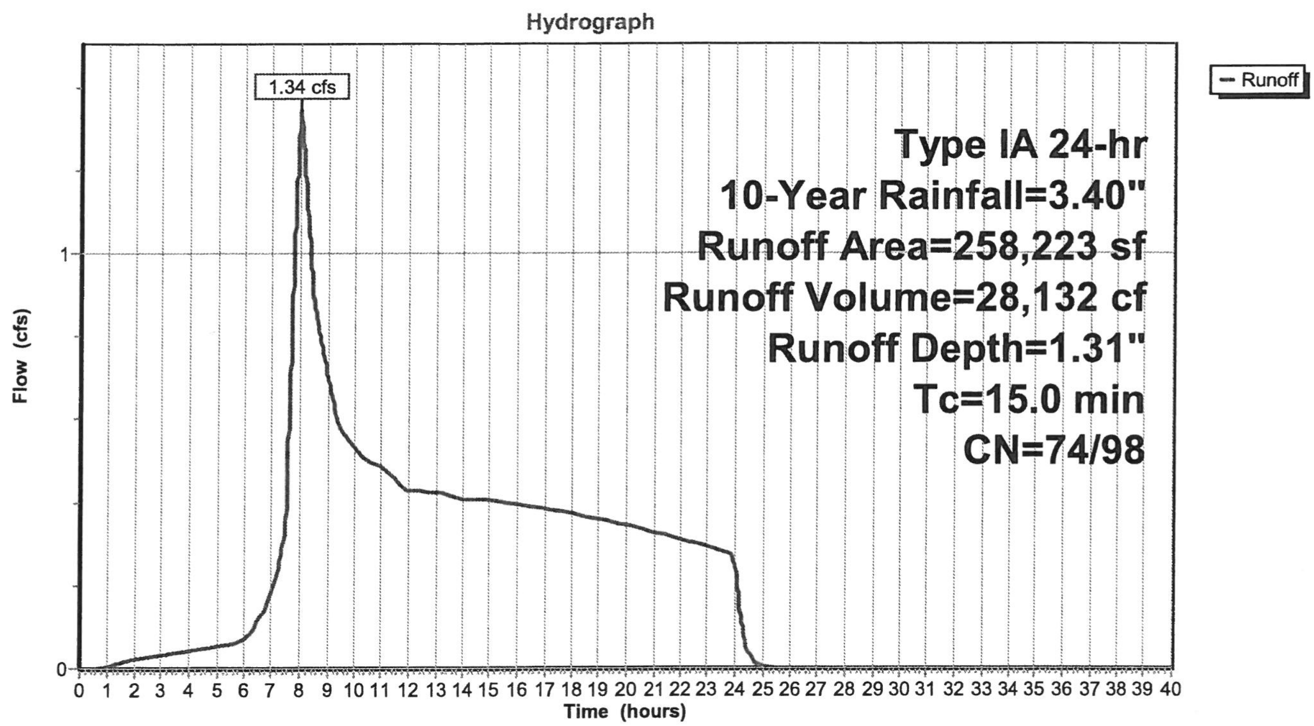
Runoff = 1.34 cfs @ 8.01 hrs, Volume= 28,132 cf, Depth= 1.31"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-Year Rainfall=3.40"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	240,635	74	>75% Grass cover, Good, HSG C
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	240,635	74	93.19% Pervious Area
	17,588	98	6.81% Impervious Area

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

Subcatchment 200: Upstream Flows



Summary for Subcatchment 201: Upstream Flows

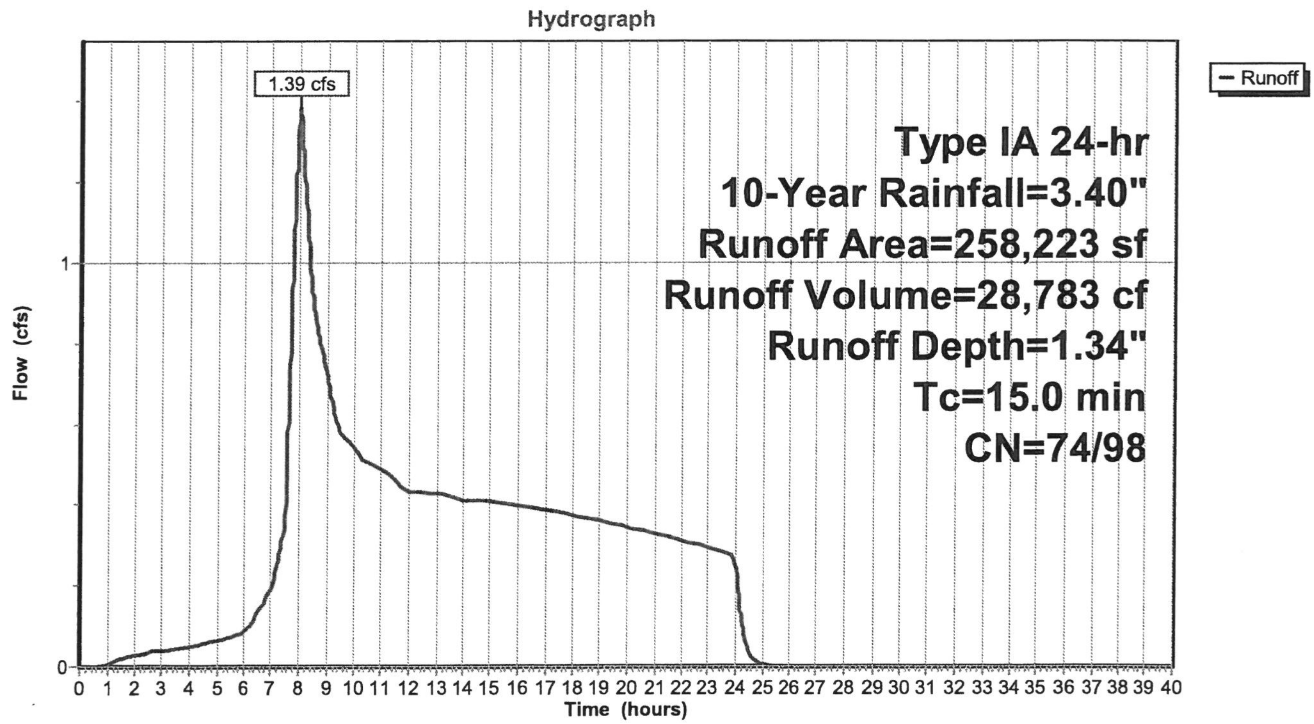
Runoff = 1.39 cfs @ 8.01 hrs, Volume= 28,783 cf, Depth= 1.34"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-Year Rainfall=3.40"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	236,718	74	>75% Grass cover, Good, HSG C
*	3,917	98	frontage new ROW
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	236,718	74	91.67% Pervious Area
	21,505	98	8.33% Impervious Area

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

Subcatchment 201: Upstream Flows



Summary for Pond 1P: Pond

```
Inflow Area =      410,115 sf, 19.51% Impervious,  Inflow Depth =  1.75"   for 10-Year event
Inflow      =      3.47 cfs @   8.00 hrs,  Volume=        59,864 cf
Outflow     =      2.11 cfs @   8.33 hrs,  Volume=        59,864 cf,  Atten= 39%,  Lag= 19.5 min
Primary     =      2.11 cfs @   8.33 hrs,  Volume=        59,864 cf
Secondary   =      0.00 cfs @   0.00 hrs,  Volume=             0 cf
Tertiary    =      0.00 cfs @   0.00 hrs,  Volume=             0 cf
```

Routing by Stor-Ind method, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Peak Elev= 532.42' @ 8.33 hrs Surf.Area= 3,040 sf Storage= 5,294 cf

Plug-Flow detention time= 44.9 min calculated for 59,850 cf (100% of inflow)
Center-of-Mass det. time= 45.0 min (815.7 - 770.8)

Volume	Invert	Avail.Storage	Storage Description
#1	530.50'	14,681 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
530.50	2,208	0	0
531.00	2,609	1,204	1,204
532.00	3,015	2,812	4,016
533.00	3,075	3,045	7,061
534.00	4,035	3,555	10,616
535.00	4,095	4,065	14,681

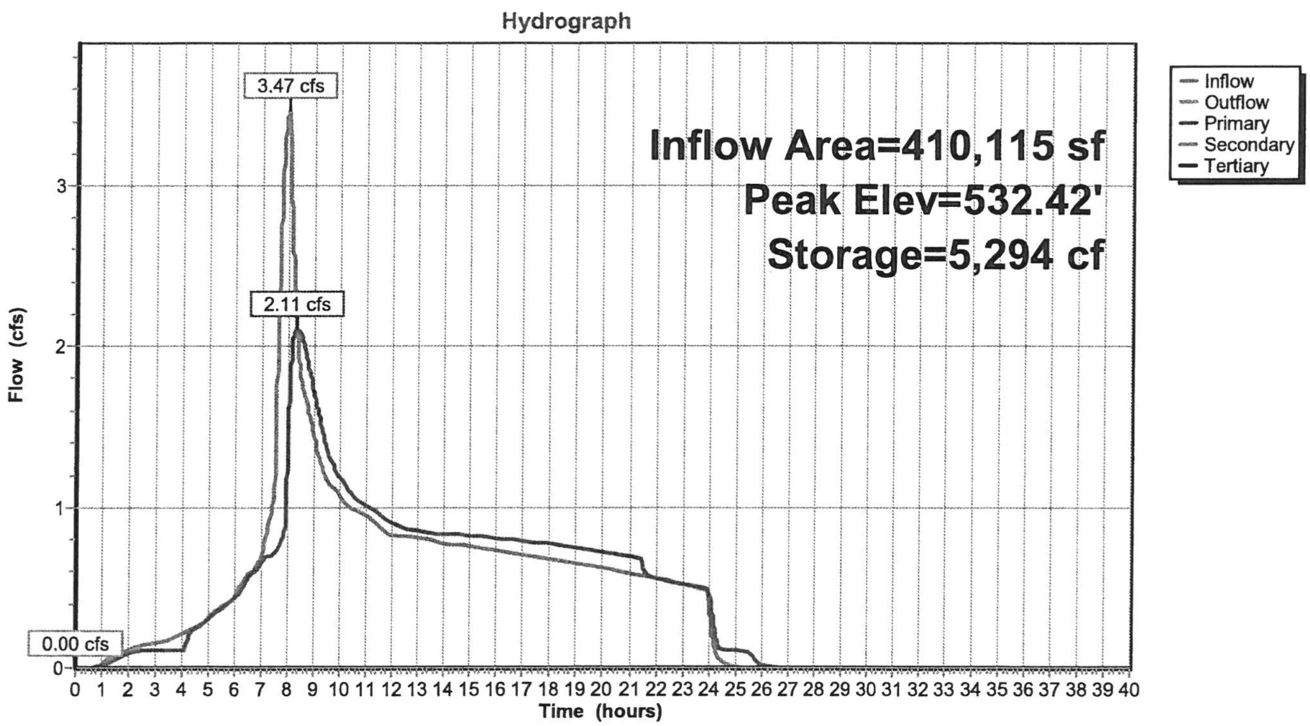
Device	Routing	Invert	Outlet Devices
#1	Primary	529.00'	**15.0" Round Culvert** L= 20.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 529.00' / 528.90' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.23 sf
#2	Device 1	527.82'	**4.4" Vert. Low Orifice** C= 0.600
#3	Device 2	530.50'	**2.000 in/hr Exfiltration over Surface area**
#4	Device 2	530.72'	**24.0" x 24.0" Horiz. Ditch Inlet #1** C= 0.600 Limited to weir flow at low heads
#5	Device 1	531.70'	**8.6" Vert. High Orifice** C= 0.600
#6	Secondary	532.87'	**24.0" x 24.0" Horiz. Ditch Inlet #2** C= 0.600 Limited to weir flow at low heads
#7	Tertiary	532.87'	**48.0" Horiz. Overflow Manhole** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=2.11 cfs @ 8.33 hrs HW=532.42' (Free Discharge)
└─**1=Culvert** (Passes 2.11 cfs of 9.88 cfs potential flow)
  ├─**2=Low Orifice** (Orifice Controls 0.94 cfs @ 8.91 fps)
  │ ├─**3=Exfiltration** (Passes < 0.14 cfs potential flow)
  │ └─**4=Ditch Inlet #1** (Passes < 25.13 cfs potential flow)
  └─**5=High Orifice** (Orifice Controls 1.17 cfs @ 2.90 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=530.50' (Free Discharge)
└─**6=Ditch Inlet #2** (Controls 0.00 cfs)

Tertiary OutFlow Max=0.00 cfs @ 0.00 hrs HW=530.50' (Free Discharge)
└─**7=Overflow Manhole** (Controls 0.00 cfs)

Pond 1P: Pond

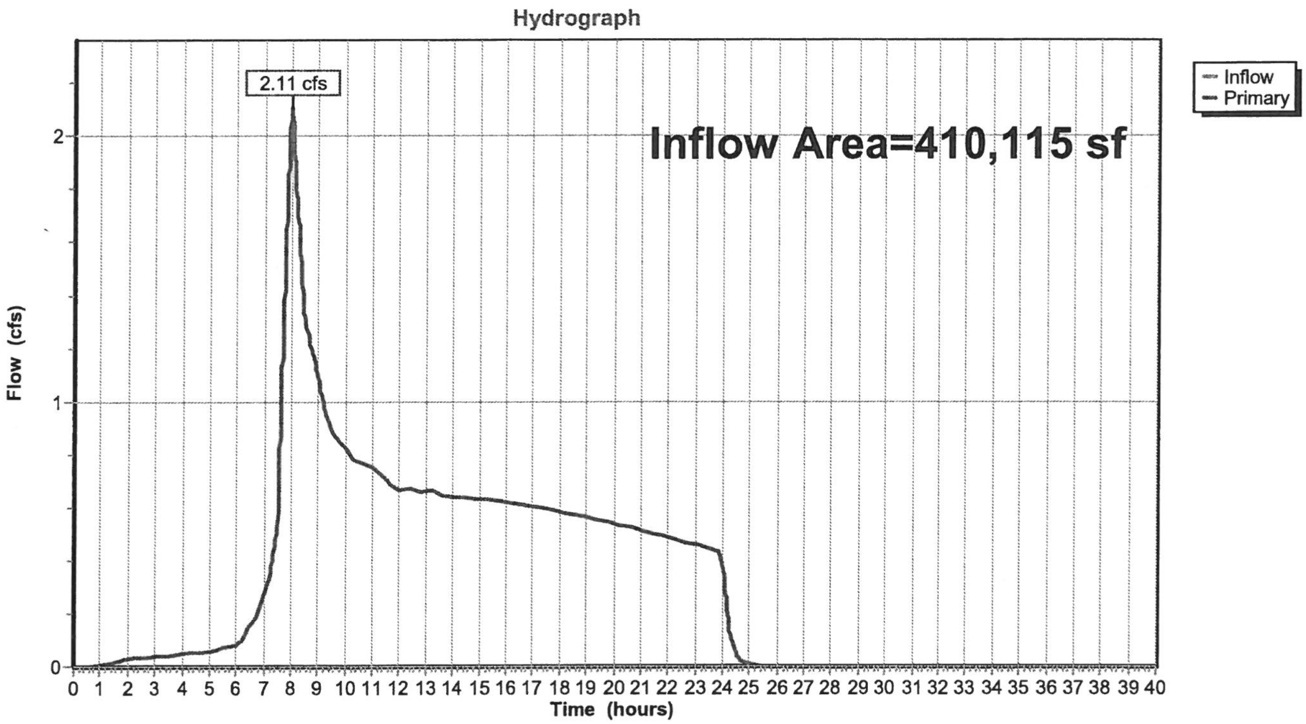


Summary for Link 104L: (new Link)

Inflow Area =	410,115 sf,	4.90% Impervious,	Inflow Depth = 1.27"	for 10-Year event
Inflow =	2.11 cfs @	8.00 hrs, Volume=	43,375 cf	
Primary =	2.11 cfs @	8.00 hrs, Volume=	43,375 cf,	Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs

Link 104L: (new Link)



Summary for Subcatchment 100: Pre-Developed Site

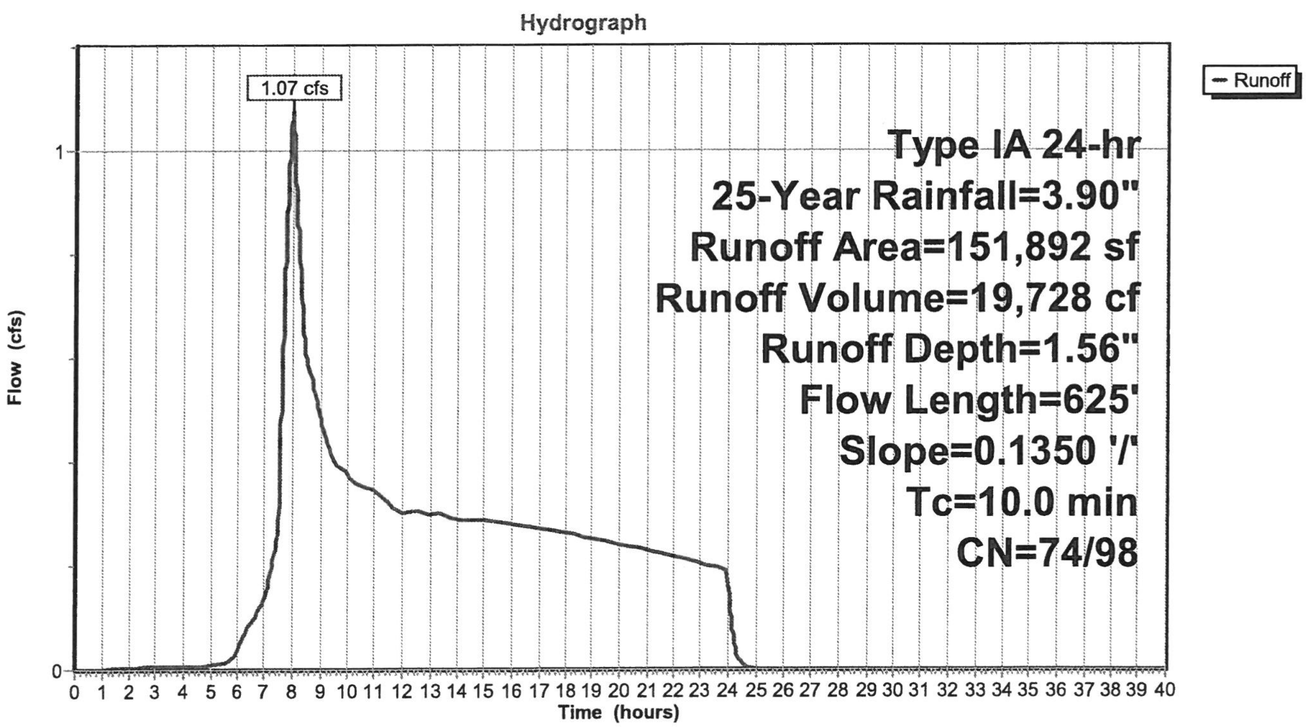
Runoff = 1.07 cfs @ 8.00 hrs, Volume= 19,728 cf, Depth= 1.56"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-Year Rainfall=3.90"

	Area (sf)	CN	Description
*	2,500	98	roofs
	149,392	74	>75% Grass cover, Good, HSG C
	151,892	74	Weighted Average
	149,392	74	98.35% Pervious Area
	2,500	98	1.65% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.2	100	0.1350	0.32		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.50"
4.8	525	0.1350	1.84		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
10.0	625	Total			

Subcatchment 100: Pre-Developed Site



Summary for Subcatchment 101: Post-Developed Site

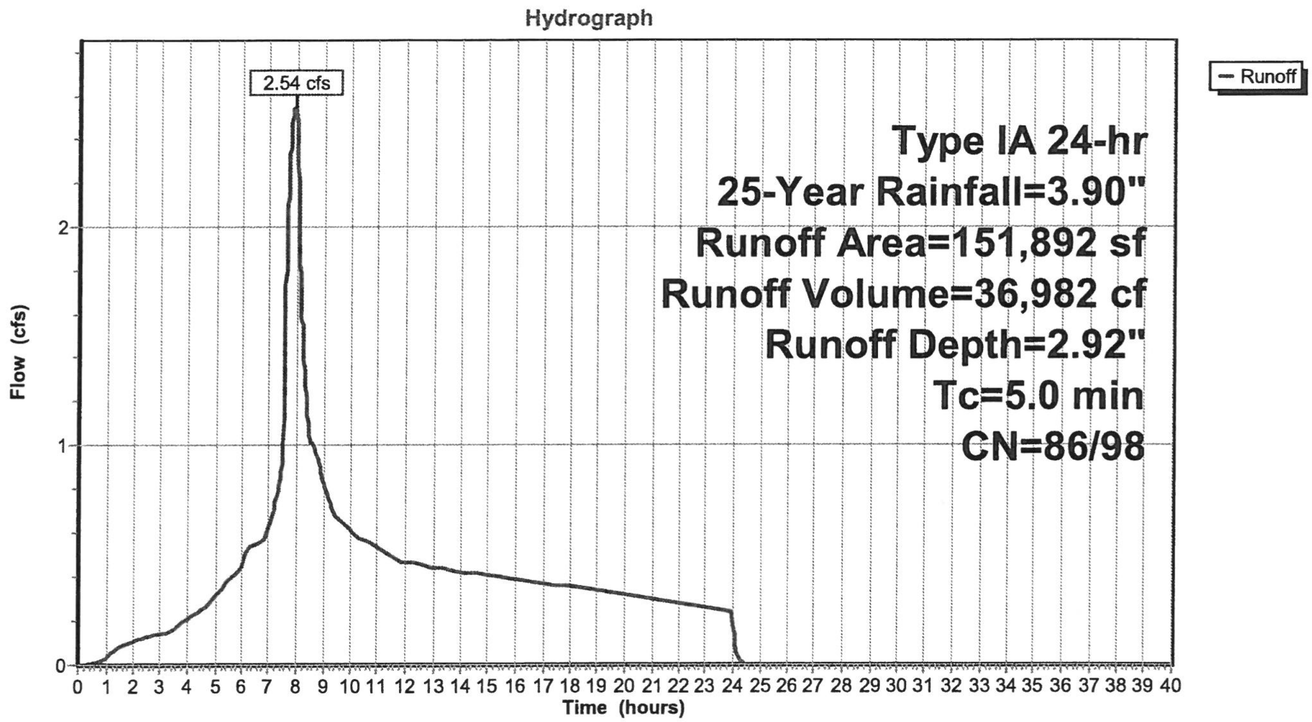
Runoff = 2.54 cfs @ 7.91 hrs, Volume= 36,982 cf, Depth= 2.92"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-Year Rainfall=3.90"

	Area (sf)	CN	Description
*	37,500	98	lots
*	7,602	98	sidewalks
*	13,394	98	ROW
	93,396	86	<50% Grass cover, Poor, HSG C
	151,892	91	Weighted Average
	93,396	86	61.49% Pervious Area
	58,496	98	38.51% Impervious Area

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

Subcatchment 101: Post-Developed Site



Summary for Subcatchment 200: Upstream Flows

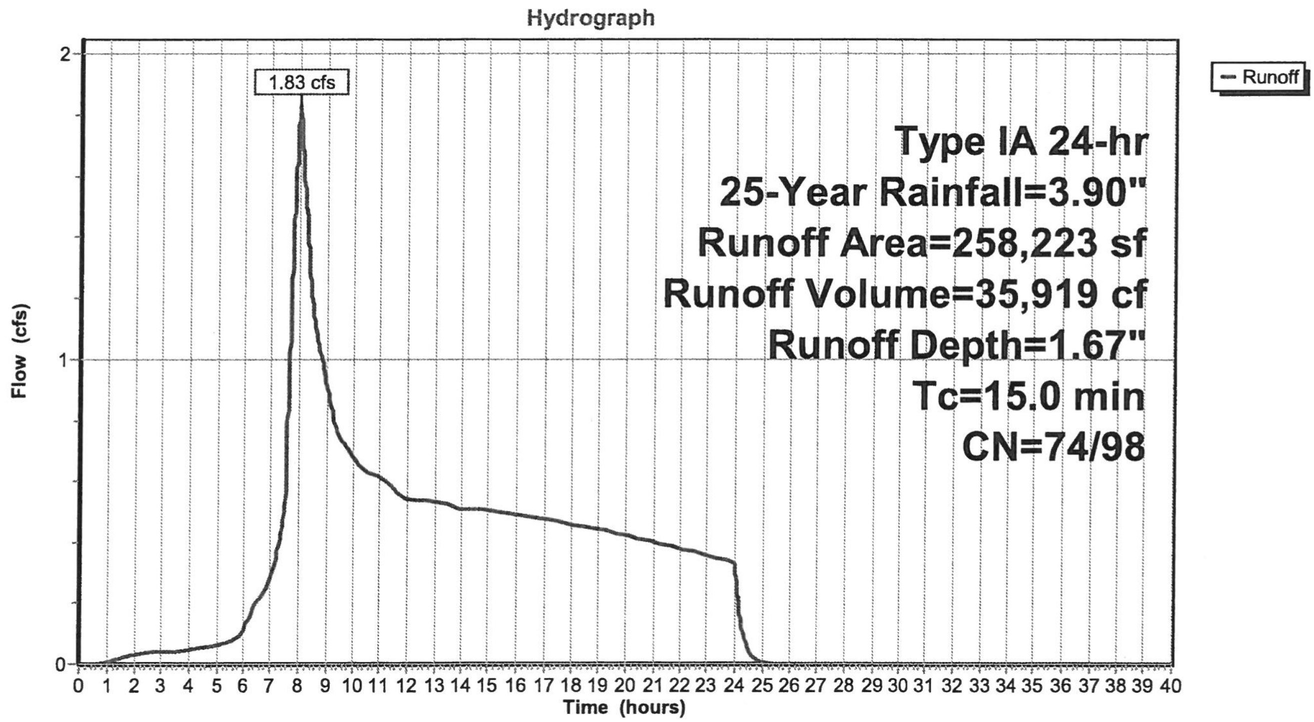
Runoff = 1.83 cfs @ 8.00 hrs, Volume= 35,919 cf, Depth= 1.67"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-Year Rainfall=3.90"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	240,635	74	>75% Grass cover, Good, HSG C
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	240,635	74	93.19% Pervious Area
	17,588	98	6.81% Impervious Area

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

Subcatchment 200: Upstream Flows



Summary for Subcatchment 201: Upstream Flows

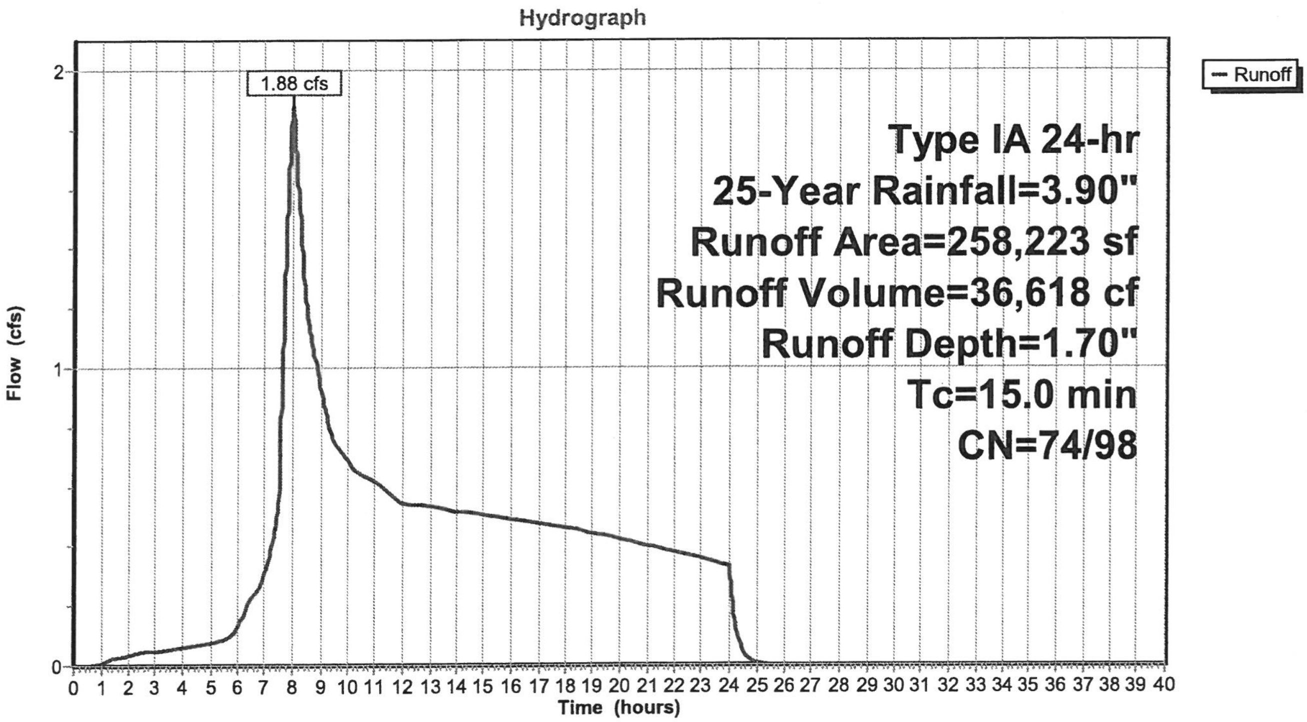
Runoff = 1.88 cfs @ 8.00 hrs, Volume= 36,618 cf, Depth= 1.70"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 25-Year Rainfall=3.90"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	236,718	74	>75% Grass cover, Good, HSG C
*	3,917	98	frontage new ROW
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	236,718	74	91.67% Pervious Area
	21,505	98	8.33% Impervious Area

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

Subcatchment 201: Upstream Flows



Summary for Pond 1P: Pond

Inflow Area =	410,115 sf, 19.51% Impervious,	Inflow Depth = 2.15"	for 25-Year event
Inflow =	4.36 cfs @ 8.00 hrs,	Volume=	73,600 cf
Outflow =	2.74 cfs @ 8.30 hrs,	Volume=	73,600 cf, Atten= 37%, Lag= 18.1 min
Primary =	2.74 cfs @ 8.30 hrs,	Volume=	73,600 cf
Secondary =	0.00 cfs @ 0.00 hrs,	Volume=	0 cf
Tertiary =	0.00 cfs @ 0.00 hrs,	Volume=	0 cf

Routing by Stor-Ind method, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Peak Elev= 532.86' @ 8.30 hrs Surf.Area= 3,067 sf Storage= 6,636 cf

Plug-Flow detention time= 50.0 min calculated for 73,582 cf (100% of inflow)
Center-of-Mass det. time= 50.0 min (813.8 - 763.8)

Volume	Invert	Avail.Storage	Storage Description
#1	530.50'	14,681 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
530.50	2,208	0	0
531.00	2,609	1,204	1,204
532.00	3,015	2,812	4,016
533.00	3,075	3,045	7,061
534.00	4,035	3,555	10,616
535.00	4,095	4,065	14,681

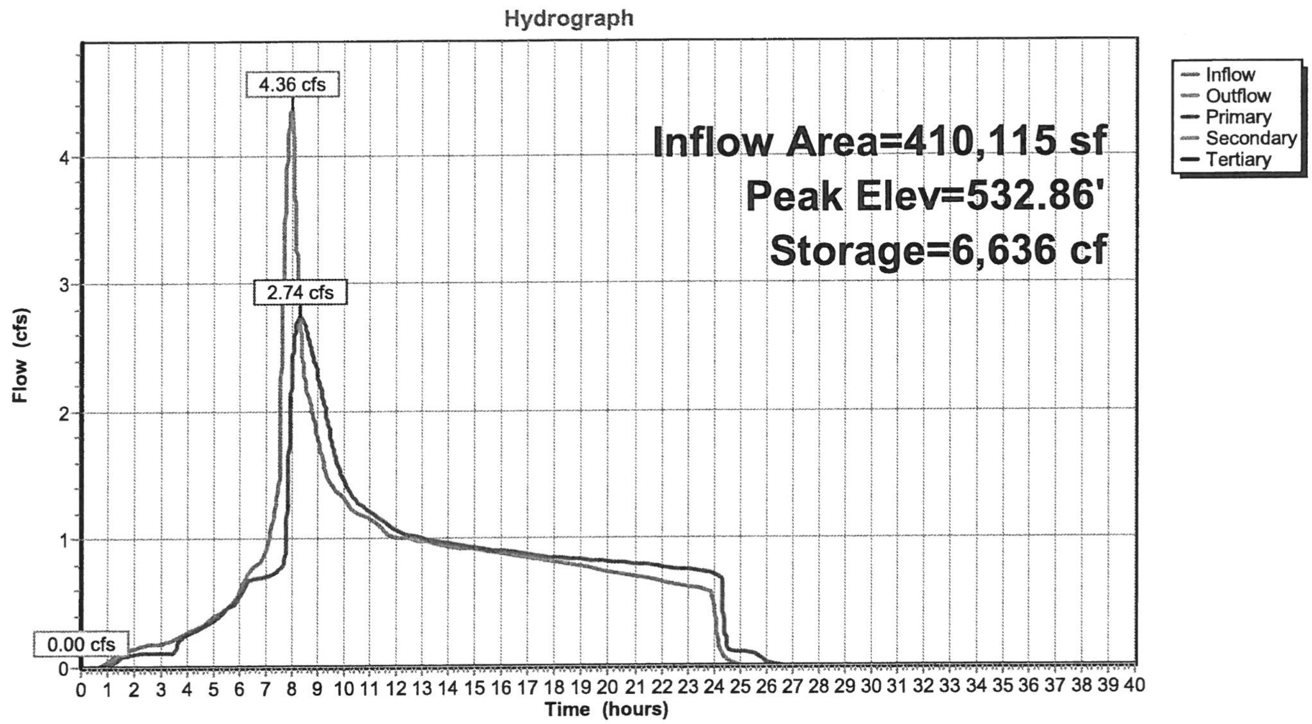
Device	Routing	Invert	Outlet Devices
#1	Primary	529.00'	**15.0" Round Culvert** L= 20.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 529.00' / 528.90' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.23 sf
#2	Device 1	527.82'	**4.4" Vert. Low Orifice** C= 0.600
#3	Device 2	530.50'	**2.000 in/hr Exfiltration over Surface area**
#4	Device 2	530.72'	**24.0" x 24.0" Horiz. Ditch Inlet #1** C= 0.600 Limited to weir flow at low heads
#5	Device 1	531.70'	**8.6" Vert. High Orifice** C= 0.600
#6	Secondary	532.87'	**24.0" x 24.0" Horiz. Ditch Inlet #2** C= 0.600 Limited to weir flow at low heads
#7	Tertiary	532.87'	**48.0" Horiz. Overflow Manhole** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=2.74 cfs @ 8.30 hrs HW=532.86' (Free Discharge)
↑—**1=Culvert** (Passes 2.74 cfs of 10.63 cfs potential flow)
  ↑—**2=Low Orifice** (Orifice Controls 1.00 cfs @ 9.46 fps)
    ↑—**3=Exfiltration** (Passes < 0.14 cfs potential flow)
    —**4=Ditch Inlet #1** (Passes < 28.18 cfs potential flow)
  —**5=High Orifice** (Orifice Controls 1.74 cfs @ 4.32 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=530.50' (Free Discharge)
↑—**6=Ditch Inlet #2** (Controls 0.00 cfs)

Tertiary OutFlow Max=0.00 cfs @ 0.00 hrs HW=530.50' (Free Discharge)
↑—**7=Overflow Manhole** (Controls 0.00 cfs)

Pond 1P: Pond

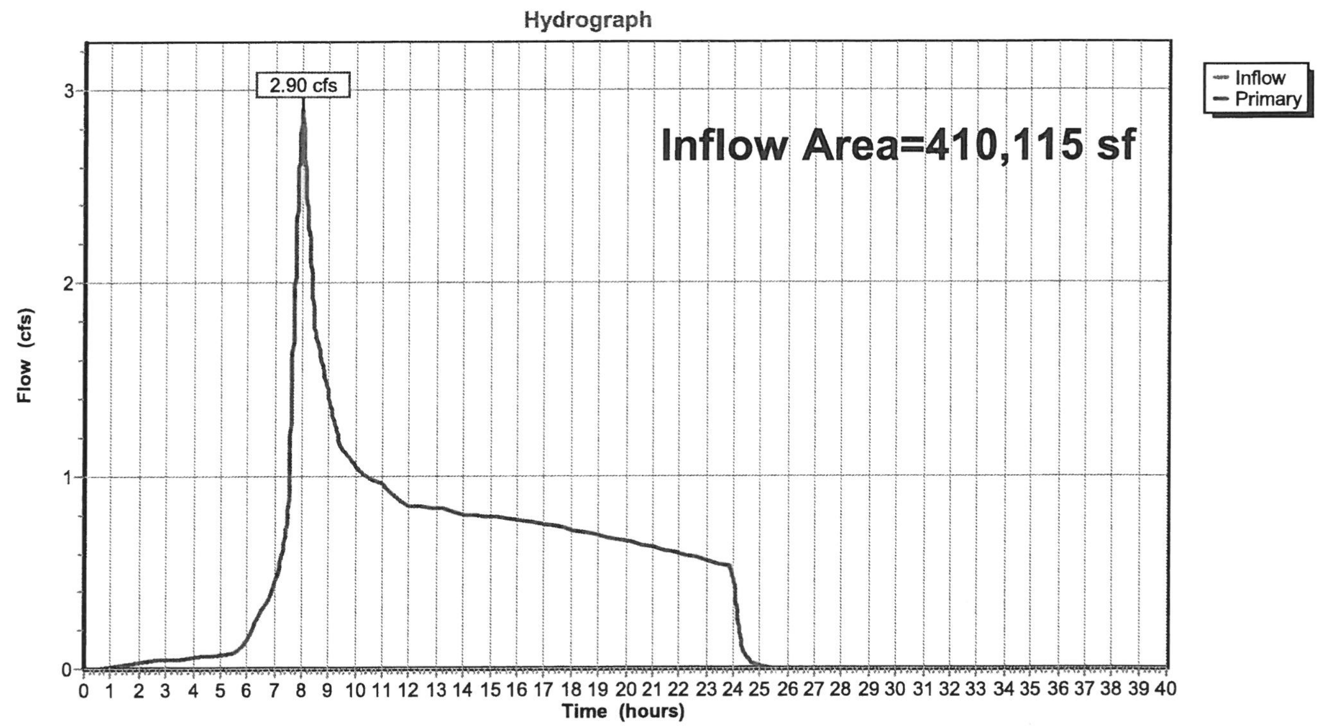


Summary for Link 104L: (new Link)

Inflow Area =	410,115 sf,	4.90% Impervious,	Inflow Depth = 1.63"	for 25-Year event
Inflow =	2.90 cfs @	8.00 hrs, Volume=	55,647 cf	
Primary =	2.90 cfs @	8.00 hrs, Volume=	55,647 cf, Atten= 0%, Lag= 0.0 min	

Primary outflow = Inflow, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs

Link 104L: (new Link)



Summary for Subcatchment 100: Pre-Developed Site

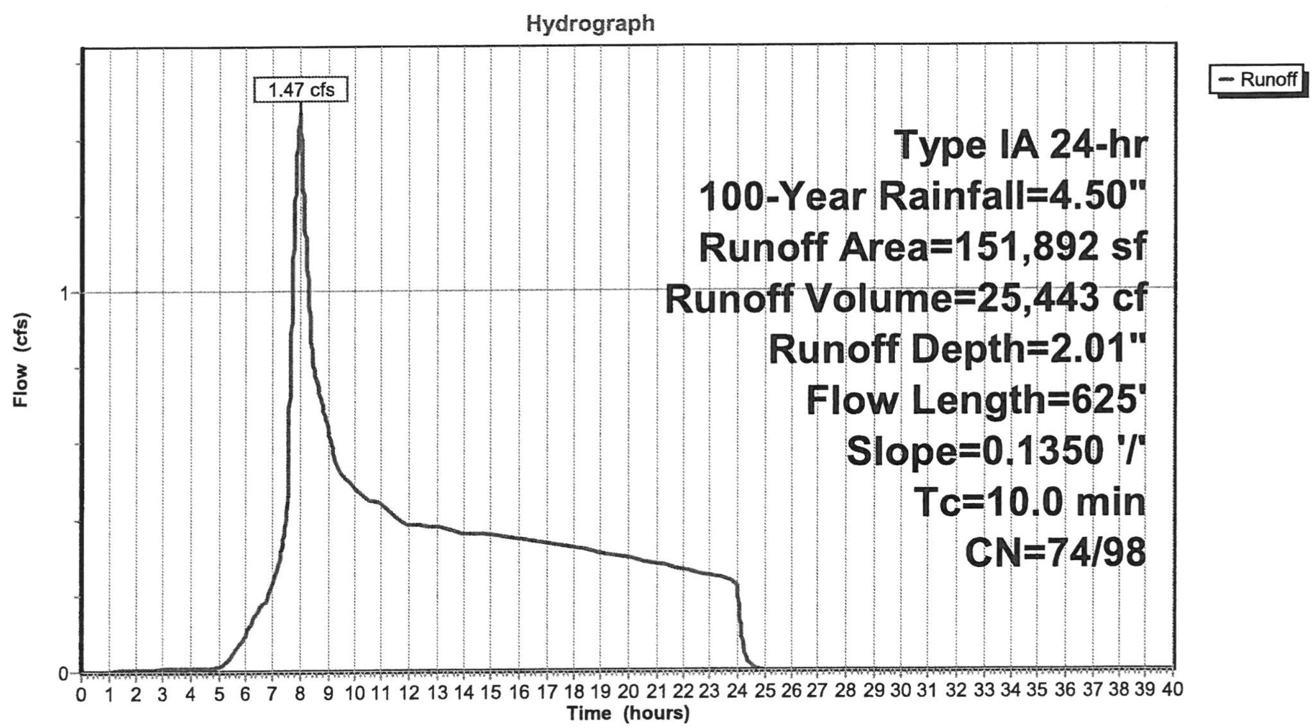
Runoff = 1.47 cfs @ 8.00 hrs, Volume= 25,443 cf, Depth= 2.01"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-Year Rainfall=4.50"

	Area (sf)	CN	Description
*	2,500	98	roofs
	149,392	74	>75% Grass cover, Good, HSG C
	151,892	74	Weighted Average
	149,392	74	98.35% Pervious Area
	2,500	98	1.65% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.2	100	0.1350	0.32		**Sheet Flow,** Grass: Short n= 0.150 P2= 2.50"
4.8	525	0.1350	1.84		**Shallow Concentrated Flow,** Woodland Kv= 5.0 fps
10.0	625	Total			

Subcatchment 100: Pre-Developed Site



Summary for Subcatchment 101: Post-Developed Site

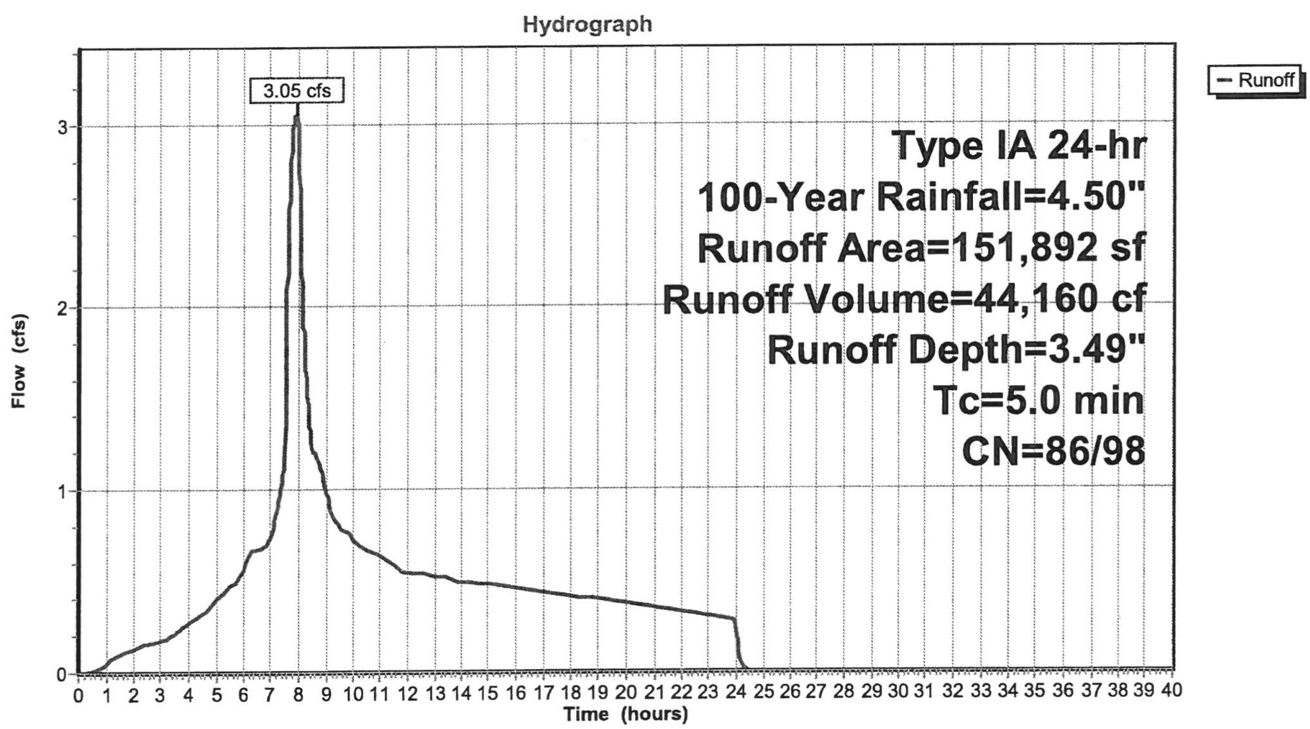
Runoff = 3.05 cfs @ 7.90 hrs, Volume= 44,160 cf, Depth= 3.49"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-Year Rainfall=4.50"

	Area (sf)	CN	Description
*	37,500	98	lots
*	7,602	98	sidewalks
*	13,394	98	ROW
	93,396	86	<50% Grass cover, Poor, HSG C
	151,892	91	Weighted Average
	93,396	86	61.49% Pervious Area
	58,496	98	38.51% Impervious Area

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

Subcatchment 101: Post-Developed Site



Summary for Subcatchment 200: Upstream Flows

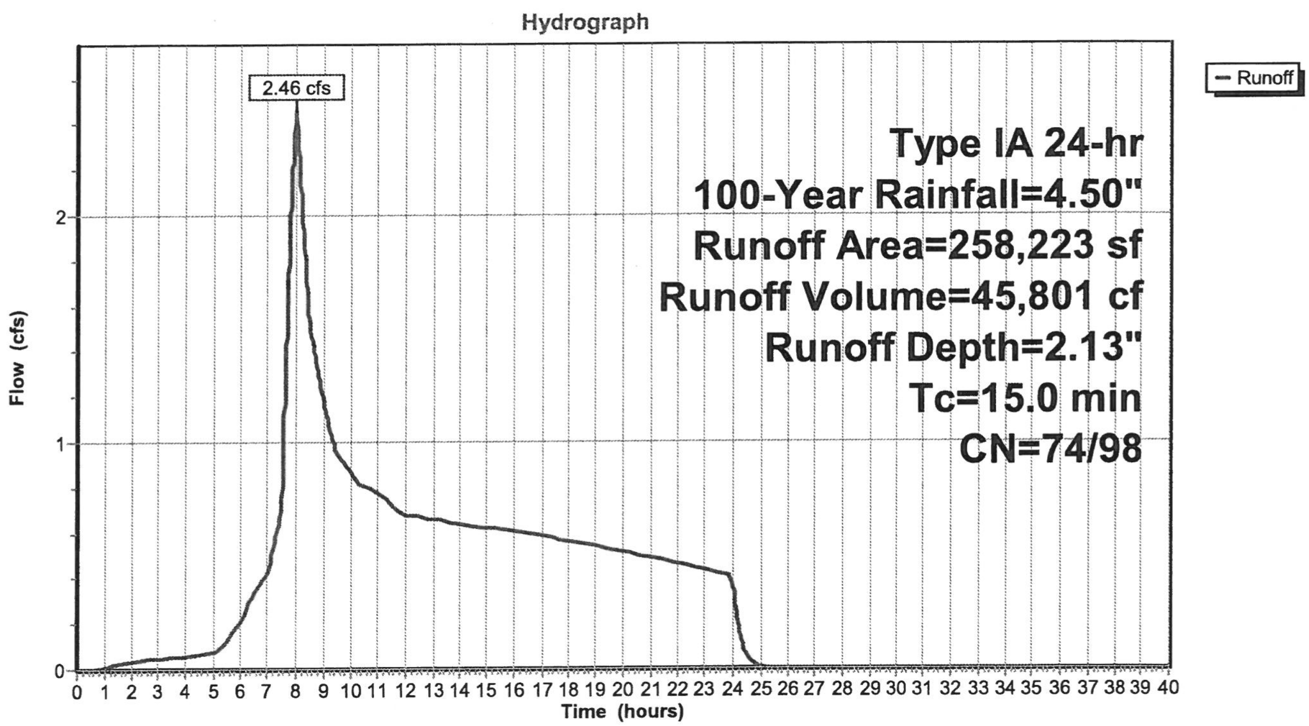
Runoff = 2.46 cfs @ 8.00 hrs, Volume= 45,801 cf, Depth= 2.13"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-Year Rainfall=4.50"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	240,635	74	>75% Grass cover, Good, HSG C
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	240,635	74	93.19% Pervious Area
	17,588	98	6.81% Impervious Area

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

Subcatchment 200: Upstream Flows



Summary for Subcatchment 201: Upstream Flows

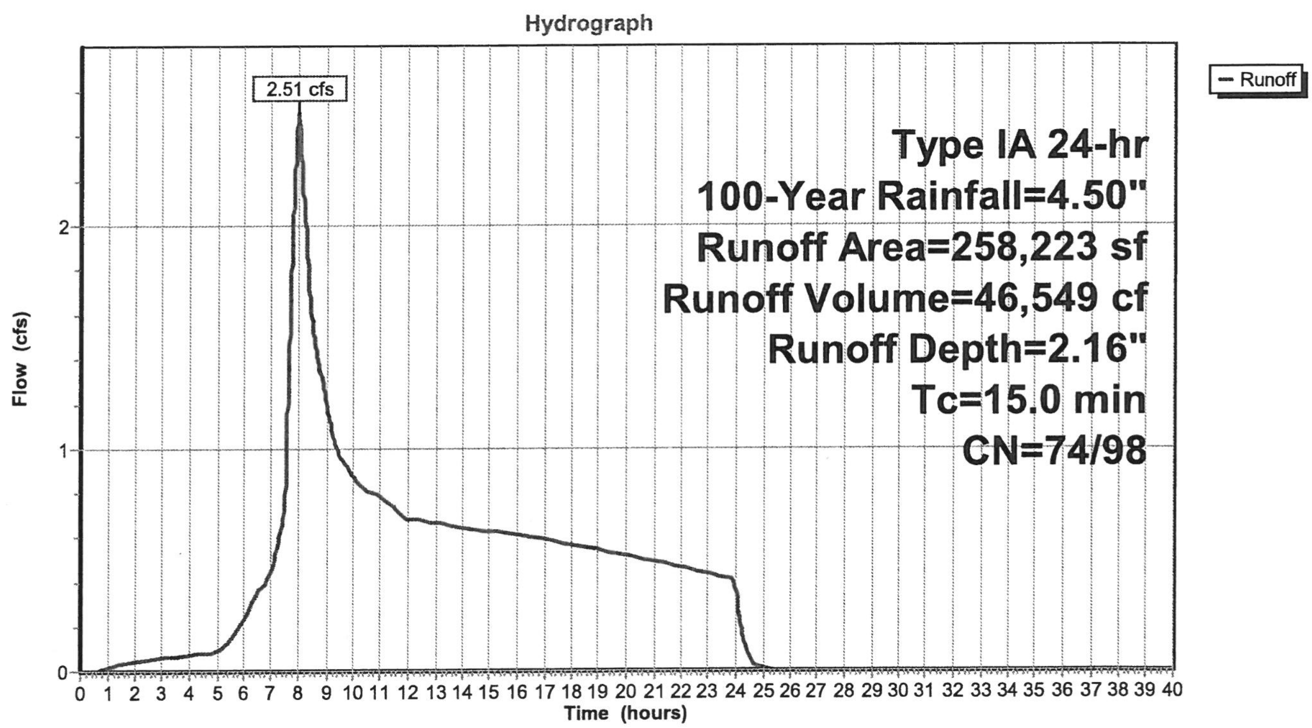
Runoff = 2.51 cfs @ 8.00 hrs, Volume= 46,549 cf, Depth= 2.16"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Type IA 24-hr 100-Year Rainfall=4.50"

	Area (sf)	CN	Description
*	7,500	98	3 lots, impervious area
	236,718	74	>75% Grass cover, Good, HSG C
*	3,917	98	frontage new ROW
*	10,088	98	Ex. ROW
	258,223	76	Weighted Average
	236,718	74	91.67% Pervious Area
	21,505	98	8.33% Impervious Area

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

Subcatchment 201: Upstream Flows



Summary for Pond 1P: Pond

Inflow Area = 410,115 sf, 19.51% Impervious, Inflow Depth = 2.65" for 100-Year event
Inflow = 5.49 cfs @ 8.00 hrs, Volume= 90,709 cf
Outflow = 5.40 cfs @ 8.01 hrs, Volume= 90,709 cf, Atten= 2%, Lag= 0.7 min
Primary = 2.88 cfs @ 8.01 hrs, Volume= 88,046 cf
Secondary = 0.98 cfs @ 8.01 hrs, Volume= 1,036 cf
Tertiary = 1.54 cfs @ 8.01 hrs, Volume= 1,627 cf

Routing by Stor-Ind method, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs
Peak Elev= 532.98' @ 8.01 hrs Surf.Area= 3,074 sf Storage= 7,005 cf

Plug-Flow detention time= 48.9 min calculated for 90,709 cf (100% of inflow)
Center-of-Mass det. time= 48.9 min (805.3 - 756.3)

Volume	Invert	Avail.Storage	Storage Description
#1	530.50'	14,681 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
530.50	2,208	0	0
531.00	2,609	1,204	1,204
532.00	3,015	2,812	4,016
533.00	3,075	3,045	7,061
534.00	4,035	3,555	10,616
535.00	4,095	4,065	14,681

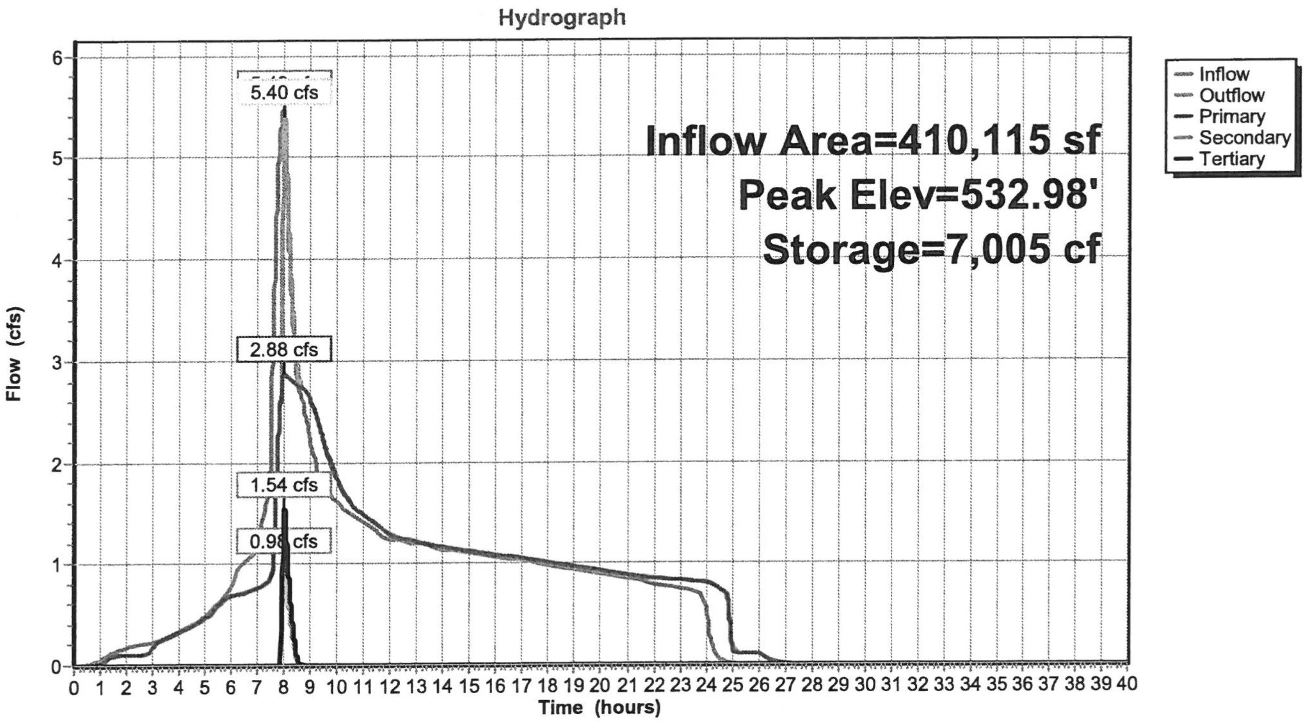
Device	Routing	Invert	Outlet Devices
#1	Primary	529.00'	**15.0" Round Culvert** L= 20.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 529.00' / 528.90' S= 0.0050 '/' Cc= 0.900 n= 0.013, Flow Area= 1.23 sf
#2	Device 1	527.82'	**4.4" Vert. Low Orifice** C= 0.600
#3	Device 2	530.50'	**2.000 in/hr Exfiltration over Surface area**
#4	Device 2	530.72'	**24.0" x 24.0" Horiz. Ditch Inlet #1** C= 0.600 Limited to weir flow at low heads
#5	Device 1	531.70'	**8.6" Vert. High Orifice** C= 0.600
#6	Secondary	532.87'	**24.0" x 24.0" Horiz. Ditch Inlet #2** C= 0.600 Limited to weir flow at low heads
#7	Tertiary	532.87'	**48.0" Horiz. Overflow Manhole** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=2.88 cfs @ 8.01 hrs HW=532.98' (Free Discharge)
↑**1=Culvert** (Passes 2.88 cfs of 10.83 cfs potential flow)
↑**2=Low Orifice** (Orifice Controls 1.01 cfs @ 9.61 fps)
↑**3=Exfiltration** (Passes < 0.14 cfs potential flow)
4=Ditch Inlet #1 (Passes < 28.96 cfs potential flow)
5=High Orifice (Orifice Controls 1.87 cfs @ 4.63 fps)

Secondary OutFlow Max=0.97 cfs @ 8.01 hrs HW=532.98' (Free Discharge)
↑**6=Ditch Inlet #2** (Weir Controls 0.97 cfs @ 1.09 fps)

Tertiary OutFlow Max=1.53 cfs @ 8.01 hrs HW=532.98' (Free Discharge)
↑**7=Overflow Manhole** (Weir Controls 1.53 cfs @ 1.09 fps)

Pond 1P: Pond

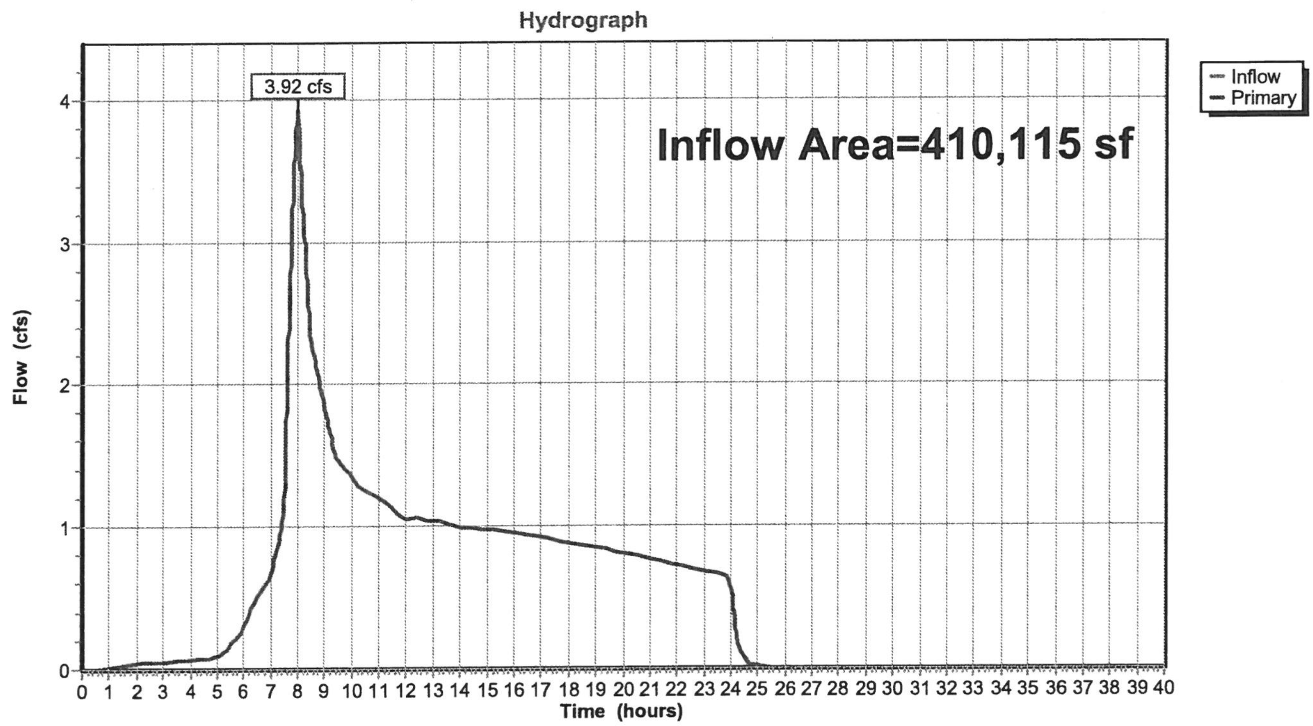


Summary for Link 104L: (new Link)

Inflow Area =	410,115 sf,	4.90% Impervious,	Inflow Depth = 2.08"	for 100-Year event	
Inflow =	3.92 cfs @	8.00 hrs, Volume=	71,244 cf		
Primary =	3.92 cfs @	8.00 hrs, Volume=	71,244 cf,	Atten= 0%,	Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-40.00 hrs, dt= 0.01 hrs

Link 104L: (new Link)



Appendix D:

BASIN ID	SQUARE FEET	ACRES
101	151,892 SF	3.49
200	258,223 SF	5.93

SHEET 1 OF

EMERIO Design

6445 SW FALLBROOK PLACE, SUITE 100
BEAVERTON, OREGON 97008
TEL: (503) 746-8812
FAX: (503) 639-9592
www.emeriodesign.com

REVISIONS

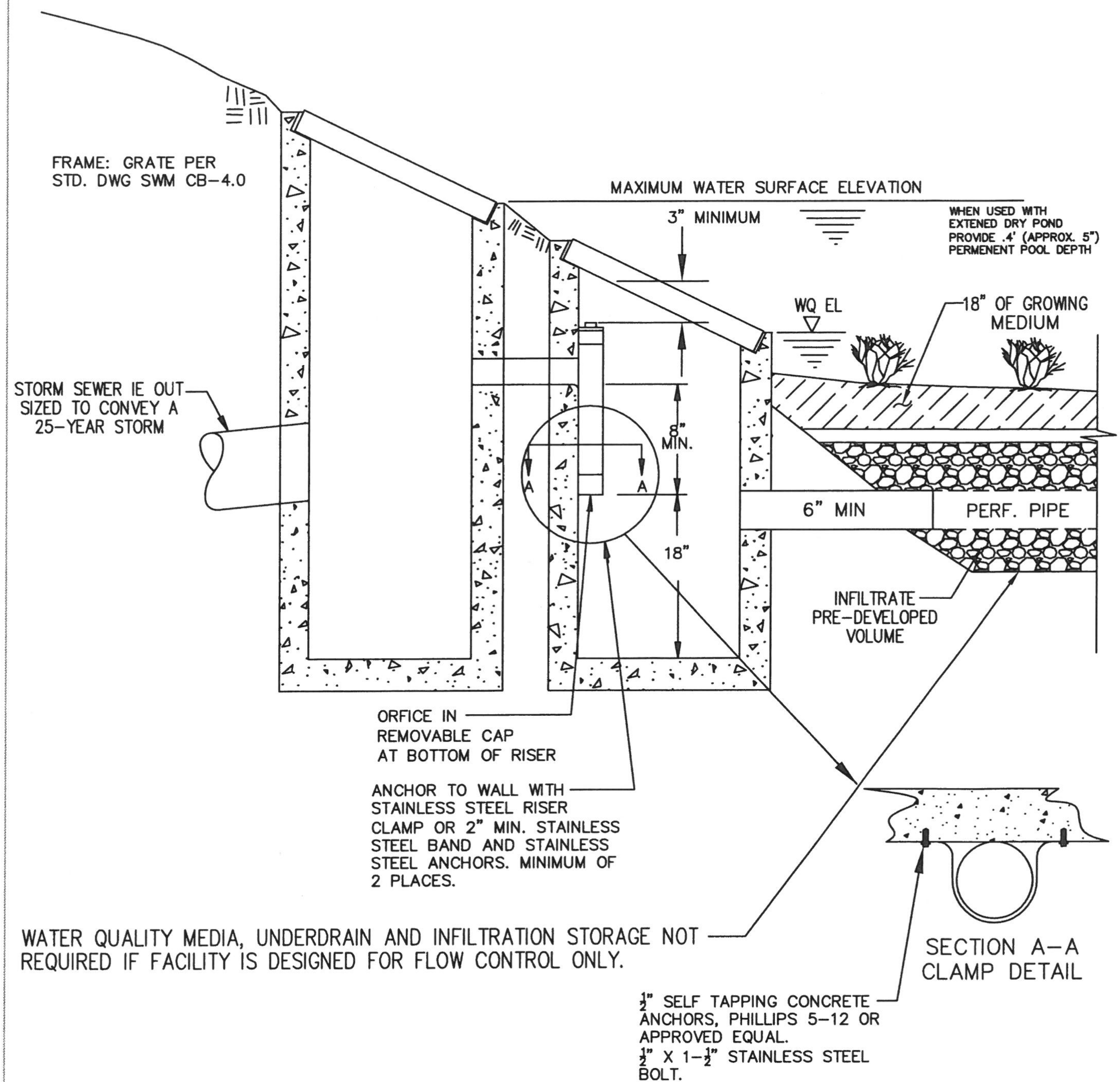
NO.	DATE	DESCRIPTION
0	6/2019	PLANNING 1ST SUBMITTAL

POST-DEVELOPED BASIN MAP

23000 23010 SW BLAND CIRCLE

TAX MAP 21E35B
TAX LOT 400 & 404
WEST LINN, OREGON

FILE: P:\0540-001 23000 23010 SW Bland Cir\docs\civil\storm\CAD\0540-001stormwater_exhibit, Layout: 01 POST-DEVELOPED BASIN MAP, Plot Date: 7/5/2019 3:15 PM, by: Josh Ayers



NOTES:

1. CONNECTING PIPE AND TEE SHALL BE 4", 6", OR 8" AWWA C-900 OR ASTM 3034 PVC,AND ONE SIZE LARGER THAN THE ORIFICE OPENING.
2. MAXIMUM ORIFICE OPENING SHALL BE 6" DIAMETER.
3. STRUCTURES TYPE AND SIZE SHALL CONFORM WITH DETAIL SWM FC-5.0.
4. FRAME AND GRATE SHALL CONFORM TO CATCH BASIN-FRAME AND GRATE (DETAIL SWM CB-4.0).
5. SUBMERGED ORFICE AND RISER SHALL BE SECURED FLUSH AGAINST WALL OF STRUCTURE AS APPROVED.
6. MAINTAINANCE ACCESS REQUIRED TO WITHIN 10' OF CENTER OF BOTH STRUCTURES AND EDGE OF MAINTENANCE ACCESS ROAD.

WATER ENVIRONMENT SERVICES www.co.clackamas.or.us/wes	CLACKAMAS COUNTY 150 BEAVERCREEK ROAD OREGON CITY, OR 97045	APPROVAL DATE: 2013 SCALE: N.T.S. FLOW CONTROL- STRUCTURE TYPE 3	STANDARD DRAWING SWM FC-6.0

GeoPacific
Engineering, Inc.

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Updated September 13, 2019
Project No. 18-5089

Ben Looney
Growth Commercial Capitol
511 Main Street #101
Oregon City, Oregon 97045
Via email: ben@growthcc.com

CC: Steve Miller, Emerio Design Via email: stevem@emeriodesign.com

SUBJECT: GEOTECHNICAL REPORT
23000 & 23010 BLAND CIRCLE SUBDIVISION
T2S R1E SECTION 35B TAX LOTS 201, 400, & 404
WEST LINN, OREGON

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above-referenced project. The purpose of our investigation was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-6777, dated October 31, 2018, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The subject site is located on the north side of Bland Circle in West Linn, Clackamas County, Oregon (Figure 1). The property totals approximately 3.5 acres in size and topography is gently to moderately sloping to the southwest at grades of approximately 5 to 20 percent. The site is currently occupied by two homes. Vegetation consists primarily of short grasses and dense to sparse trees.

It is our understanding that the proposed development includes 15 lots for single family homes, new streets, and associated underground utilities (Figure 2). A water quality facility is planned in the southwestern portion of the project. The southern home will be retained and the northern structure will be removed. The grading plan provided for our review indicates maximum cuts will be on the order of 12 feet or less and fills will be minimal. The water quality facility will incorporate retaining walls.

REGIONAL AND LOCAL GEOLOGIC SETTING

The subject site lies within the Willamette Valley/Puget Sound lowland, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. A series of discontinuous faults subdivide the Willamette Valley into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands, while down-warped structural blocks form sedimentary basins.

The site is located on a south facing slope at elevations of approximately 530 to 620 feet above sea level. The subject site is underlain by the Miocene aged (about 14.5 to 16.5 million years ago) Columbia River Basalt Formation, which are a thick sequence of lava flows which form the crystalline basement of the Tualatin Valley (Beeson et al., 1989; Gannett and Caldwell, 1998). The basalts are composed of dense, finely crystalline rock that is commonly fractured along blocky and columnar vertical joints. Individual basalt flow units typically range from 25 to 125 feet thick and interflow zones are typically vesicular, scoriaceous, brecciated, and sometimes include sedimentary rocks.

REGIONAL SEISMIC SETTING

At least three major fault zones capable of generating damaging earthquakes are thought to exist in the vicinity of the subject site. These include the Portland Hills Fault Zone, the Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone.

Portland Hills Fault Zone

The Portland Hills Fault Zone is a series of NW-trending faults that include the central Portland Hills Fault, the western Oatfield Fault, and the eastern East Bank Fault. These faults occur in a northwest-trending zone that varies in width between 3.5 and 5.0 miles. The combined three faults vertically displace the Columbia River Basalt by 1,130 feet and appear to control thickness changes in late Pleistocene (approx. 780,000 years) sediment (Madin, 1990). The Portland Hills Fault occurs along the Willamette River at the base of the Portland Hills, and is approximately 4.6 miles northeast of the site. The East Bank Fault is oriented roughly parallel to the Portland Hills Fault, on the east bank of the Willamette River, and is located approximately 8.6 miles northwest of the site. The Oatfield Fault occurs along the western side of the Portland Hills, and is approximately 3.8 miles northeast of the site. The Oatfield Fault is considered to be potentially seismogenic (Wong, et al., 2000). Madin and Mabey (1996) indicate the Portland Hills Fault Zone has experienced Late Quaternary (last 780,000 years) fault movement; however, movement has not been detected in the last 20,000 years. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000). No historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, but in 1991 a M3.5 earthquake occurred on a NW-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is assumed to be potentially active (Geomatrix Consultants, 1995).

Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW-trending faults that lies approximately 15.6 miles southwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek Fault or Newberg Fault (the fault closest to the subject site); however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner et al. 1992; Geomatrix Consultants, 1995).

Cascadia Subduction Zone

The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 4 cm per year (Goldfinger et al., 1996). A growing body of geologic evidence suggests that prehistoric subduction zone earthquakes have occurred (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington, (2) burial of subsided tidal marshes by tsunami wave deposits, (3) paleoliquefaction features, and (4) geodetic uplift patterns on the Oregon coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years with the last event occurring 300 years ago (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). The inferred seismogenic portion of the plate interface lies roughly along the Oregon coast at depths of between 20 and 40 miles.

SUBSURFACE CONDITIONS

Our site-specific exploration for this report was conducted on November 19, 2018. A total of 4 exploratory test pits were excavated with a small to medium sized trackhoe to depths of 5.5 to 8 feet at the approximate locations indicated on Figure 2. It should be noted that test pit locations were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate.

A GeoPacific Engineering Geologist continuously monitored the field exploration program and logged the test pits. Soils observed in the explorations were classified in general accordance with the Unified Soil Classification System (USCS). Rock hardness was classified in accordance with Table 1, modified from the ODOT Rock Hardness Classification Chart. During exploration, our geologist also noted geotechnical conditions such as soil consistency, moisture and groundwater conditions. Logs of test pits are attached to this report. The following report sections are based on the exploration program and summarize subsurface conditions encountered at the site.

Table 1. Rock Hardness Classification Chart

ODOT Rock Hardness Rating	Field Criteria	Unconfined Compressive Strength	Typical Equipment Needed For Excavation
Extremely Soft (R0)	Indented by thumbnail	<100 psi	Small excavator
Very Soft (R1)	Scratched by thumbnail, crumbled by rock hammer	100-1,000 psi	Small excavator
Soft (R2)	Not scratched by thumbnail, indented by rock hammer	1,000-4,000 psi	Medium excavator (slow digging with small excavator)
Medium Hard (R3)	Scratched or fractured by rock hammer	4,000-8,000 psi	Medium to large excavator (slow to very slow digging), typically requires chipping with hydraulic hammer or mass excavation)
Hard (R4)	Scratched or fractured w/ difficulty	8,000-16,000 psi	Slow chipping with hydraulic hammer and/or blasting
Very Hard (R5)	Not scratched or fractured after many blows, hammer rebounds	>16,000 psi	Blasting

Undocumented Fill: Undocumented fill was not encountered in our explorations. Our reconnaissance indicates that approximately 5 feet of undocumented fill may be present in the vicinity of the existing driveway of the home at 23010 Bland Circle, as indicated on Figure 2. We anticipate other areas of fill may be present in the vicinity of the existing homes.

Topsoil Horizon: Directly underlying the ground surface in test pits TP-1 through TP-4 was a topsoil horizon consisting of light brown, moderately to highly organic silt (OL-ML). The topsoil horizon was generally loose, contained many fine roots, and extended to a depth of 9 to 12 inches.

Residual Soil: Underlying the topsoil horizon in test pits TP-1 through TP-4 was clayey silt (ML) to silty clay (CL) residual soil resulting from in-place weathering of the underlying Columbia River Basalt Formation. The light reddish brown silty clay to clayey silt contained trace weathered basalt fragments and was generally characterized by a very stiff consistency. In test pits TP-1 through TP-3, the residual soil extended to a depth of 2.5 to 6.5 feet and beyond the maximum depth of exploration in test pit TP-4.

Columbia River Basalt Formation: Underlying the residual soil in test pits TP-1 through TP-3 was weathered basalt belonging to the Columbia River Basalt Formation. Generally, the gray basalt was extremely soft (R0) to soft (R2) with trace light reddish brown silty clay to clayey silt matrix. Practical refusal was encountered on soft (R2) to medium hard (R3) basalt at a depth of 5.5 to 8 feet in test pits TP-1 through TP-3. A larger machine would likely be able to excavate deeper depths. Table 2 presents the depths at which rock was first encountered in test pits and the depth at which practical refusal was achieved with a small to medium sized trackhoe equipped with rock teeth.

Table 2. Depth of Basalt Bedrock Encountered in Explorations

Test Pit	Depth Rock First Encountered	Depth of Practical Refusal on Medium Hard (R3) Basalt
TP-1	2.5'	5.5'
TP-2	6.5'	8'
TP-3	5.5'	6'

Soil Moisture and Groundwater

On November 19, 2018, neither static groundwater nor groundwater seepage was encountered in test pits excavated to a maximum depth of 8 feet below the ground surface. Regional groundwater mapping indicates that static groundwater is present at a depth of approximately 220 to 280 feet below the ground surface (Snyder, 2008). Experience has shown that temporary storm related perched groundwater within the near surface soils often occur over fine-grained native deposits such as those beneath the site during the wet season and particularly in mottled soils such as were identified in the test pits. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors.

INFILTRATION TESTING

Infiltration testing was not performed due to encountering basalt bedrock. GeoPacific does not recommend infiltrating into bedrock due to limited storage volume.

CONCLUSIONS AND RECOMMENDATIONS

Our investigation indicates that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and sufficient geotechnical monitoring is incorporated into the construction phases of the project. In our opinion, the greatest geotechnical issue for project completion is the depth of the bedrock beneath the site. Weathered basalt bedrock was encountered throughout the site and basalt was first encountered at depths of 2.5 to 6.5 feet. Practical refusal was encountered on medium hard (R3) basalt in test pits TP-1 through TP-3 at depths of 5.5 to 8 feet. A larger excavator may be able to achieve greater depths; however, difficult excavating conditions should be expected.

Site Preparation

Areas of proposed buildings, new streets, and areas to receive fill should be cleared of vegetation and any organic and inorganic debris. Existing buried structures, should be demolished and any cavities structurally backfilled. Inorganic debris and organic materials from clearing should be removed from the site. Existing fill and any organic-rich topsoil should then be stripped from construction areas of the site or where engineered fill is to be placed. Fill was not encountered in our explorations; however, our reconnaissance indicates that fill is likely present in the vicinity of the existing driveway of the southern home as indicated on Figure 2. Other areas of fill are likely present in the vicinity of the existing homes.

Organic-rich topsoil should then be stripped from native soil areas of the site. The estimated depth range necessary for removal of topsoil in cut and fill areas is approximately 6 to 9 inches, respectively. The final depth of soil removal will be determined on the basis of a site inspection after the stripping/excavation has been performed. Stripped topsoil should preferably be removed from the site due to the high density of the proposed development. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative.

Any remaining undocumented fills and subsurface structures (tile drains, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be removed and the excavations backfilled with engineered fill.

Once stripping of a particular area is approved, the area must be ripped or tilled to a depth of 12 inches, moisture conditioned, root-picked, and compacted in-place prior to the placement of engineered fill or crushed aggregate base for pavement. Exposed subgrade soils should be evaluated by the geotechnical engineer. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded scraper or dump truck. For smaller areas where access is restricted, the subgrade should be evaluated by probing the soil with a steel probe. Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition, over-excavated and replaced with engineered fill (as described below), or stabilized with rock prior to placement of engineered fill. The depth of overexcavation, if required, should be evaluated by the geotechnical engineer at the time of construction.

Engineered Fill

All grading for the proposed development should be performed as engineered grading in accordance with the applicable building code at time of construction with the exceptions and additions noted herein. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 95% of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

Site earthwork will be impacted by soil moisture and shallow groundwater conditions. Earthwork in wet weather would likely require extensive use of cement or lime treatment, or other special measures, at a considerable additional cost compared to earthwork performed under dry-weather conditions.

Keyways and Benching For Engineered Fill on Slopes

Engineered fill to be placed in sloping areas inclining steeper than 20% grade should be constructed on a keyway and benches in accordance with the typical design shown in Figure 3. Keyways should have a minimum depth of 2 feet and minimum width of 10 feet. Additional removals of potentially unstable soils may be required depending on conditions observed during construction. Both benches and keyways should be roughly horizontal in the down slope direction, but may slope up to 20% grade along topographic contour. Keyways sloping more than 20% grade along topographic contour should be benched.

The keyway should include a subdrain consisting of a minimum 3-inch diameter, ADS Heavy Duty grade (or equivalent), perforated plastic pipe enveloped in a minimum of 3 cubic feet per lineal foot of 2"-½", open-graded gravel drain rock wrapped with geotextile filter fabric (Mirafi 140N or equivalent). GeoPacific should inspect keyways, subdrains and benching prior to fill placement. Areas of potential seepage observed during construction may require a rock blanket drain in the keyway bottom.

We recommend that permanent fill and cut slopes be constructed no steeper than 2H:1V (50% grade). Fill slopes should be overbuilt a minimum of 3 feet horizontally beyond finish grade and then trimmed back to finish grade as shown on Figure 3 in order to achieve a well compacted slope face.

Excavating Conditions and Utility Trenches

We anticipate that on-site soils can be excavated using conventional heavy equipment such as scrapers and trackhoes. Weathered basalt bedrock was encountered in test pits throughout the site at depths of 2.5 to 6.5 feet and practical refusal was encountered on medium hard (R3) basalt at depths of 5.5 to 8 feet in test pits TP-1 through TP-3. A larger excavator may be able to achieve greater depths; however, difficult excavating conditions should be expected.

All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926), or be shored. The existing native soil is classified as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. This cut slope inclination is applicable to excavations above groundwater seepage zones only. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions.

Saturated soils and groundwater may be encountered in utility trenches, particularly during the wet season. We anticipate that dewatering systems consisting of ditches, sumps and pumps would be adequate for control of perched groundwater. Regardless of the dewatering system used, it should be installed and operated such that in-place soils are prevented from being removed along with the groundwater.

Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321. We recommend that trench backfill be compacted to at least 95% of the maximum dry density obtained by Modified Proctor ASTM D1557 or equivalent. Initial backfill lift thickness for a ¾"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

Erosion Control Considerations

During our field exploration program, we did not observe soil types that would be considered highly susceptible to erosion except in areas of moderately sloping topography. In our opinion, the primary concern regarding erosion potential will occur during construction, in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw wattles and silt fences. If used, these erosion control devices should be in place and remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

Wet Weather Earthwork

Soils underlying the site are likely to be moisture sensitive and may be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact fill to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications:

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Material used as engineered fill should consist of clean, granular soil containing less than 5 percent fines. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement;
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials;
- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved; and
- Geotextile silt fences, straw wattles, and fiber rolls should be strategically located to control erosion.

If cement or lime treatment is used to facilitate wet weather construction, GeoPacific should be contacted to provide additional recommendations and field monitoring.

Pavement Design

For design purposes, we used an estimated resilient modulus of 9,000 for compacted native soil. Table 3 presents our recommended minimum pavement section for dry weather construction.

Table 3. Recommended Minimum Dry-Weather Pavement Section

Material Layer	Light-duty Public Streets	Private Driveways	Compaction Standard
Asphaltic Concrete (AC)	3 in.	2.5 in.	92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾"-0 (leveling course)	2 in.	2 in.	95% of Modified Proctor AASHTO T-180
Crushed Aggregate Base 1½"-0	8 in.	6 in.	95% of Modified Proctor AASHTO T-180
Subgrade	12 in.	12 in.	95% of Standard Proctor AASHTO T-99 or equivalent

Any pockets of organic debris or loose fill encountered during ripping or tilling should be removed and replaced with engineered fill (see *Site Preparation* Section). In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving. If pavement areas are to be constructed during wet weather, the subgrade and construction plan should be reviewed by the project geotechnical engineer at the time of construction so that condition-specific recommendations can be provided. The moisture sensitive subgrade soils make the site a difficult wet weather construction project.

During placement of pavement section materials, density testing should be performed to verify compliance with project specifications. Generally, one subgrade, one base course, and one asphalt compaction test is performed for every 100 to 200 linear feet of paving.

Spread Foundations

The proposed residential structures may be supported on shallow foundations bearing on competent undisturbed, native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 inches below exterior grade. The recommended minimum widths for continuous footings supporting wood-framed walls without masonry are 12 inches for single-story, 15 inches for two-story, and 18 inches for three-story structures. Minimum foundation reinforcement should consist of a No. 4 bar at the tops of stem walls, and a No. 4 bar at the bottom of footings. Concrete slab-on-grade reinforcement should consist of No. 4 bars placed on 24-inch centers in a grid pattern.

The anticipated allowable soil bearing pressure is 1,500 lbs/ft^2 for footings bearing on competent, native soil and/or engineered fill. A maximum chimney and column load of 30 kips is recommended for the site. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. For heavier loads, the geotechnical engineer should be consulted. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.40, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Footing excavations should penetrate through topsoil and any loose soil to competent subgrade that is suitable for bearing support. All footing excavations should be trimmed neat, and all loose or softened soil should be removed from the excavation bottom prior to placing reinforcing steel bars. Due to the moisture sensitivity of on-site native soils, foundations constructed during the wet weather season may require overexcavation of footings and backfill with compacted, crushed aggregate.

Our recommendations are for house construction incorporating raised wood floors and conventional spread footing foundations. If living space of the structures will incorporate basements, a geotechnical engineer should be consulted to make additional recommendations for retaining walls, water-proofing, underslab drainage and wall subdrains. After site development, a Final Soil Engineer's Report should either confirm or modify the above recommendations.

Permanent Below-Grade Walls

Lateral earth pressures against below-grade retaining walls will depend upon the inclination of any adjacent slopes, type of backfill, degree of wall restraint, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. At-rest soil pressure is exerted on a retaining wall when it is restrained against rotation. In contrast, active soil pressure will be exerted on a wall if its top is allowed to rotate or yield a distance of roughly 0.001 times its height or greater.

If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained wall, an at-rest equivalent fluid pressure of 55 pcf should be used in design, again assuming level backfill against the wall. These values assume that drainage provisions are incorporated, free draining gravel backfill is used, and hydrostatic pressures are not allowed to develop against the wall.

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 6.5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 320 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

A coefficient of friction of 0.42 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12 to 18-inch wide zone of sand and gravel containing less than 5 percent passing the No. 200 sieve against the walls. A 3-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

Wall drains are recommended to prevent detrimental effects of surface water runoff on foundations – not to dewater groundwater. Drains should not be expected to eliminate all potential sources of water entering a basement or beneath a slab-on-grade. An adequate grade to a low point outlet drain in the crawlspace is required by code. Underslab drains are sometimes added beneath the slab when placed over soils of low permeability and shallow, perched groundwater.

Water collected from the wall drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Down spouts and roof drains should not be connected to the wall drains in order to reduce the potential for clogging. The drains should include clean-outs to allow periodic maintenance and inspection. Grades around the proposed structure should be sloped such that surface water drains away from the building.

GeoPacific should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

Structures should be located a horizontal distance of at least 1.5H away from the back of the retaining wall, where H is the total height of the wall. GeoPacific should be contacted for additional foundation recommendations where structures are located closer than 1.5H to the top of any wall.

Seismic Design

The Oregon Department of Geology and Mineral Industries (Dogami), Oregon HazVu: 2019 Statewide GeoHazards Viewer indicates that the site is in an area where *very strong* ground shaking is anticipated during an earthquake. Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2015 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2014). We recommend Site Class C be used for design per the OSSC, Table 1613.5.2 and as defined in ASCE 7, Chapter 20, Table 20.3-1. Design values determined for the site using the Applied Technology Council (ATC) 2019 Hazards By Location Online Tool are summarized in Table 4, presented on the following page, and are based upon existing soil conditions.

Table 4. Recommended Earthquake Ground Motion Parameters (2010 ASCE-7)

Parameter	Value
Location (Lat, Long), degrees	45.360, -122.654
Mapped Spectral Acceleration Values (MCE):	
Peak Ground Acceleration PGA_M	0.414
Short Period, S_s	0.951 g
1.0 Sec Period, S_1	0.409 g
Soil Factors for Site Class D:	
F_a	1.019
F_v	1.391
Residential Site Value = 2/3 x F_a x S_s	0.647 g
Residential Seismic Design Category	C

Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to earthquake shaking. Soil liquefaction is generally limited to loose, granular soils located below the water table. According to the Oregon HazVu: Statewide Geohazards Viewer, the subject site is regionally characterized as having no risk of soil liquefaction (DOGAMI:HazVu, 2019).

Footing and Roof Drains

Construction should include typical measures for controlling subsurface water beneath the homes, including positive crawlspace drainage to an adequate low-point drain exiting the foundation, visqueen covering the exposed ground in the crawlspace, and crawlspace ventilation (foundation vents). The homebuyers should be informed and educated that some slow flowing water in the crawlspaces is considered normal and not necessarily detrimental to the home given these other design elements incorporated into its construction. Appropriate design professionals should be consulted regarding crawlspace ventilation, building material selection and mold prevention issues, which are outside GeoPacific's area of expertise.

Down spouts and roof drains should collect roof water in a system separate from the footing drains to reduce the potential for clogging. Roof drain water should be directed to an appropriate discharge point and storm system well away from structural foundations. Grades should be sloped downward and away from buildings to reduce the potential for ponded water near structures.

If the proposed structures will have a raised floor, and no concrete slab-on-grade floors in living spaces are used, perimeter footing drains would not be required based on soil conditions encountered at the site and experience with standard local construction practices. Where it is desired to reduce the potential for moist crawl spaces, footing drains may be installed. If concrete slab-on-grade floors are used, perimeter footing drains should be installed as recommended below.

Where necessary, perimeter footing drains should consist of 3 or 4-inch diameter, perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. In our opinion, footing drains may outlet at the curb, or on the back sides of lots where sufficient fall is not available to allow drainage to meet the street.

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and their consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. The checklist attached to this report outlines recommended geotechnical observations and testing for the project. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, GeoPacific attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, expressed or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

GeoPacific Engineering, Inc.

CERTIFIED OREGON ELIZABETH K. RAPP No. E2190 ENGINEERING GEOLOGIST exp 2/1/20

REGISTERED PROFESSIONAL ENGINEER 14743PE OREGON JAN. 23, 1990 JAMES D. IMBRIE

EXPIRES: 06/30/2021

Beth K. Rapp, C.E.G.
Senior Engineering Geologist

James D. Imbrie, P.E., G.E.
Principal Geotechnical Engineer

Attachments: References
Checklist of Recommended Geotechnical Testing and Observation
Figure 1 – Vicinity Map
Figure 2 – Site and Exploration Plan
Figure 3 – Fill Slope Detail
Test Pit Logs (TP-1 – TP-4)

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CHECKLIST OF RECOMMENDED GEOTECHNICAL TESTING AND OBSERVATION

Item No.	Procedure	Timing	By Whom	Done
1	Preconstruction meeting	Prior to beginning site work	Contractor, Developer, Civil and Geotechnical Engineers	
2	Fill removal from site or sorting and stockpiling	Prior to mass stripping	Soil Technician/ Geotechnical Engineer	
3	Stripping, aeration, and root-picking operations	During stripping	Soil Technician	
4	Compaction testing of engineered fill (90% of Modified Proctor)	During filling, tested every 2 vertical feet	Soil Technician	
5	Compaction testing of trench backfill (95% of Standard Proctor)	During backfilling, tested every 4 vertical feet for every 200 lineal feet	Soil Technician	
6	Street Subgrade Compaction (95% of Standard Proctor)	Prior to placing base course	Soil Technician	
7	Base course compaction (95% of Modified Proctor)	Prior to paving, tested every 200 lineal feet	Soil Technician	
8	AC Compaction (92% (bottom lift) / 92% (top lift) of Rice)	During paving, tested every 200 lineal feet	Soil Technician	
9	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

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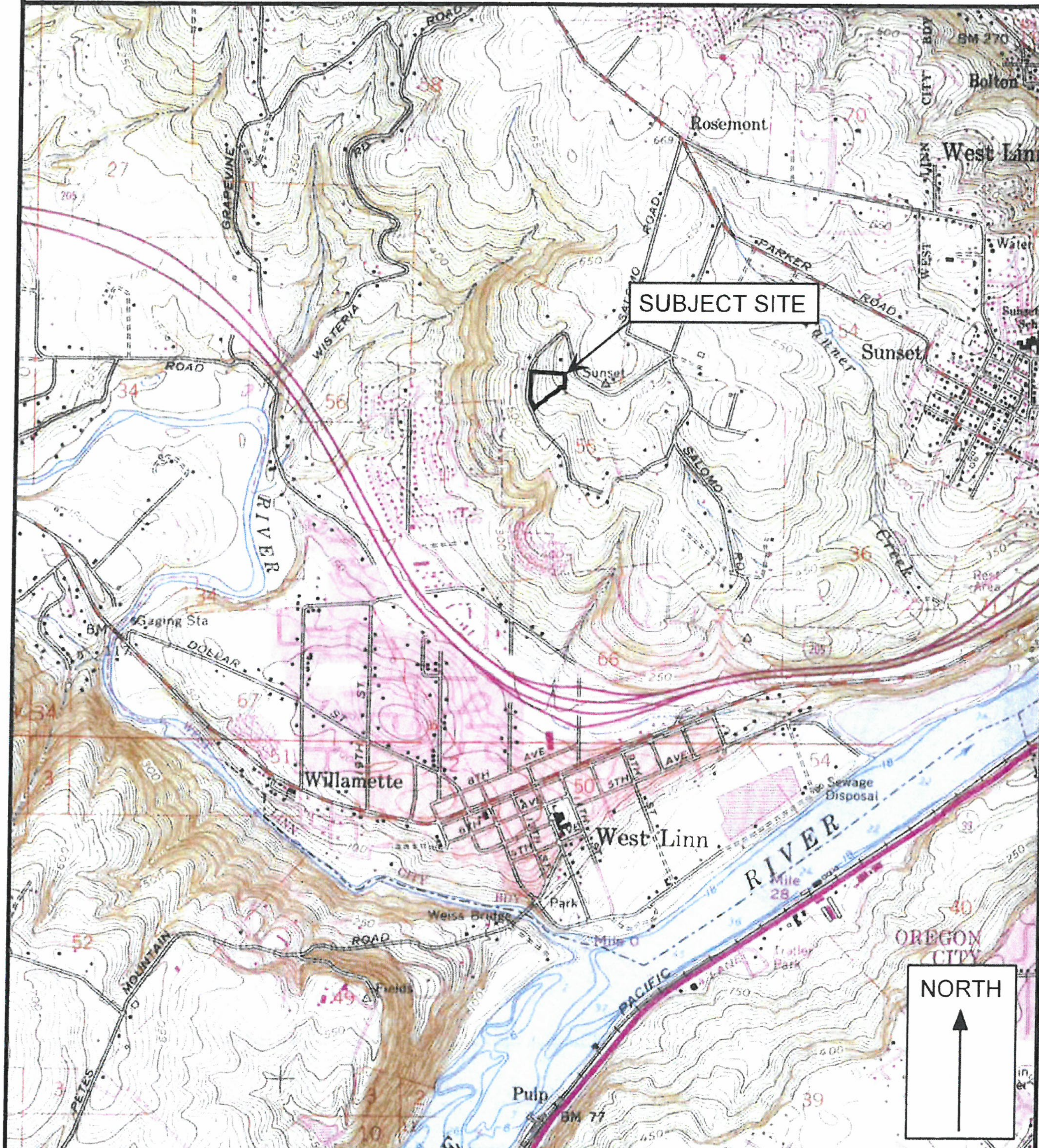
VICINITY MAP



Legend

Approximate Scale 1 in = 2,000 ft

Date: 9/13/2019
Drawn by: EKR

Base map: U.S. Geological Survey 7.5 minute Topographic Map Series, Canby, Oregon Quadrangle, 1961 (Photorevised 1985).

Project: 23000 & 23010 Bland Circle West Linn, Oregon	Project No. 18-5089	FIGURE 1

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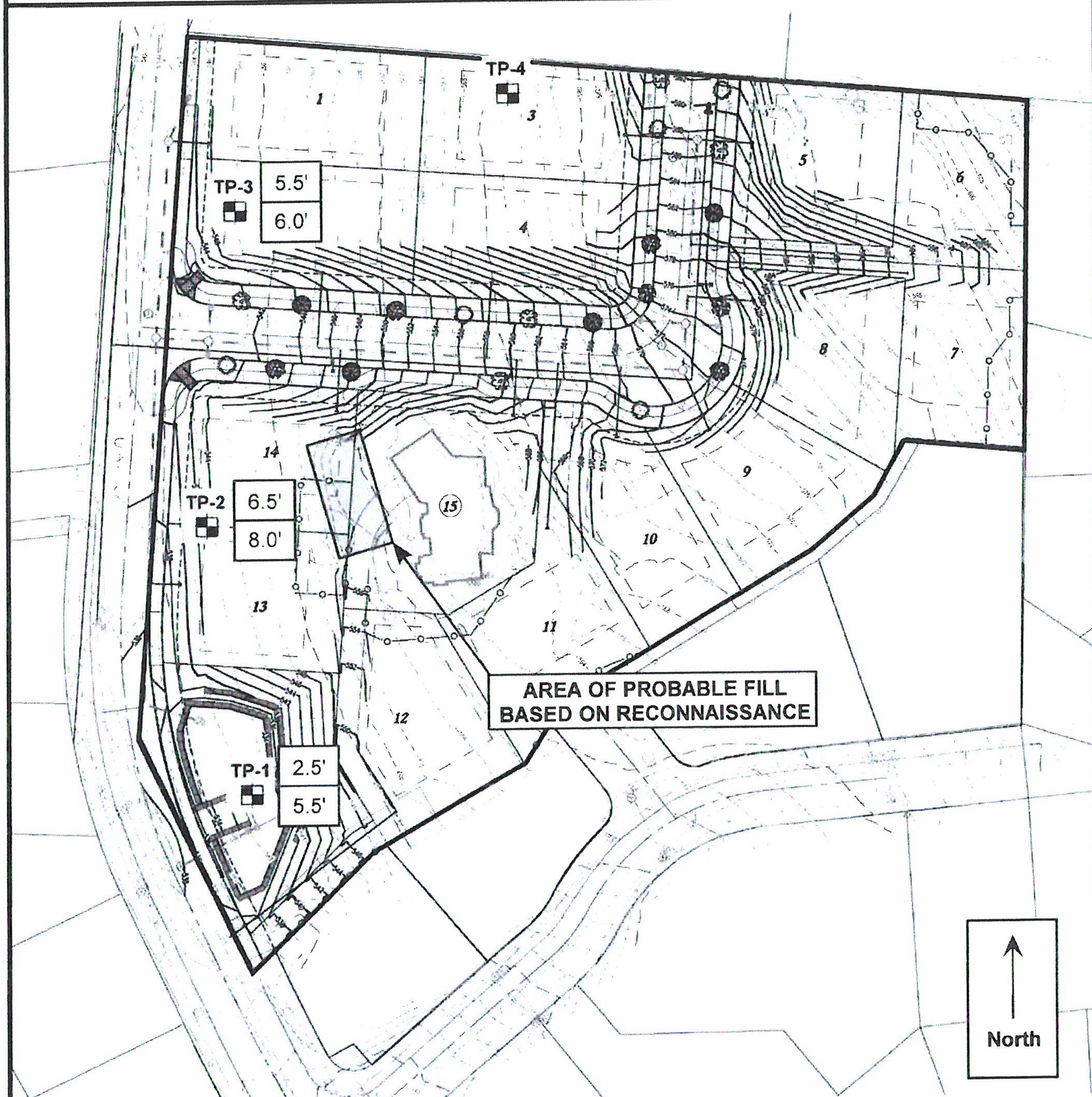
SITE PLAN AND EXPLORATION LOCATIONS



Legend

TP-1 Test Pit Designation and Approximate Location

2.5'	2.5' = Depth at Which Rock is First Encountered
5.5'	5.5' = Depth of Practical Refusal on Rock
	>10' = Depth of Practical Refusal Exceeds Depth of Exploration

Date: 9/13/2019
Drawn by: EKR

0 80'
APPROXIMATE SCALE 1"=80'

Project: 23000 & 23010 Bland Circle West Linn, Oregon	Project No. 18-5089	FIGURE 2

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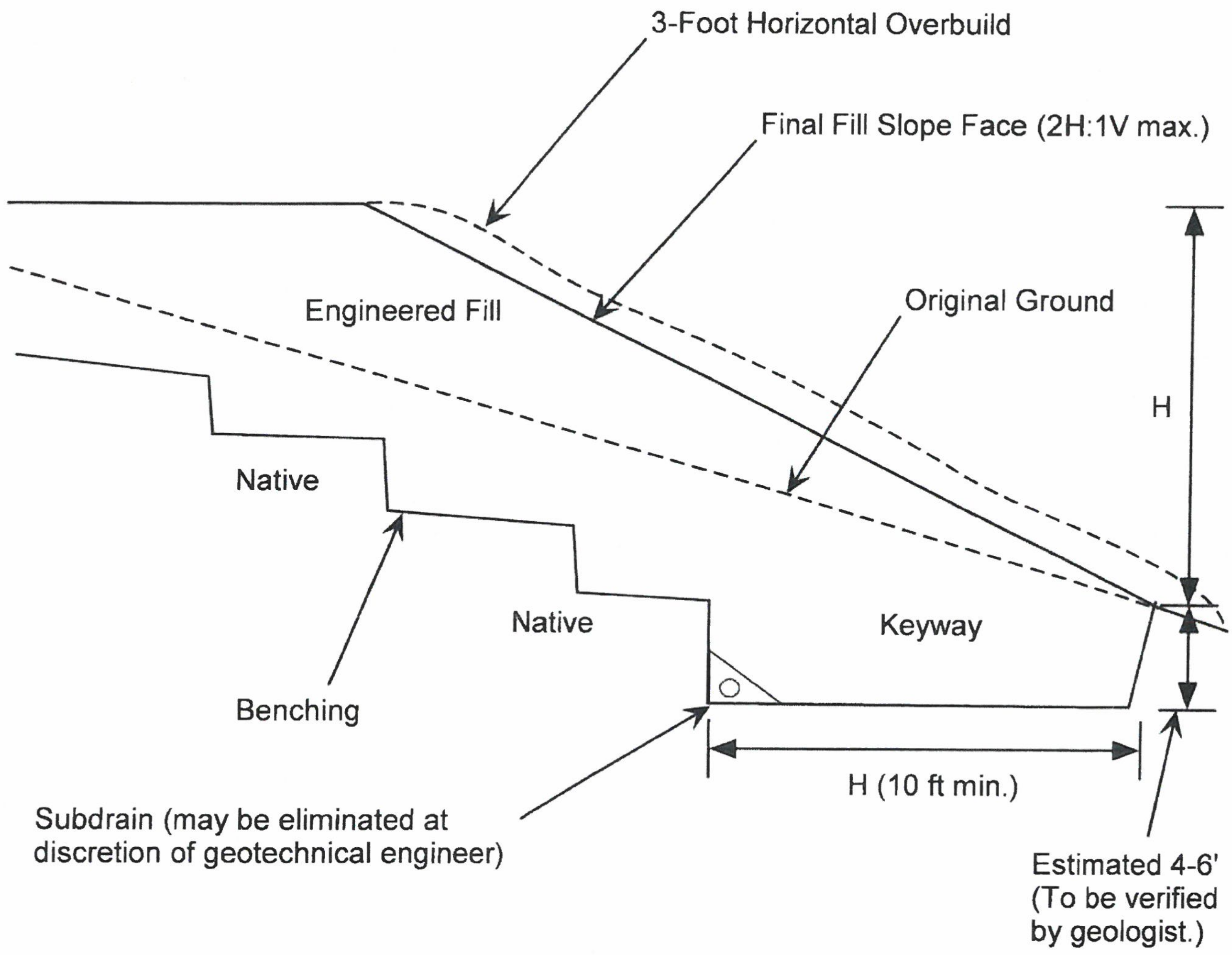
FILL SLOPE DETAIL

TYPICAL KEYWAY, BENCHING & FILL SLOPE DETAIL



Recommended subdrain is minimum 3-inch-diameter ADS Heavy Duty grade (or equivalent), perforated plastic pipe enveloped in a minimum of 3 cubic feet per lineal foot of 2" to 1/2" open-graded gravel drain rock wrapped with geotextile filter fabric (Mirafi 140N or equivalent).

Project: 23000 & 23010 Bland Circle West Linn, Oregon	Project No. 18-5089	FIGURE 3

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TEST PIT LOG

Project: 23000 & 23010 Bland Circle
West Linn, Oregon

Project No. 18-5089

Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1						Moderately organic SILT (OL-ML), brown, loose, fine roots throughout, moist (Topsoil Horizon)
1	2.5					Very stiff, silty CLAY (CL) to clayey SILT (ML), light reddish brown, subtle orange and gray mottling, trace fine roots throughout, damp to moist (Residual Soil)
1.5	4.5					
2	4.5					
3	4.5					Extremely soft (R0) to very soft (R1), highly weathered BASALT, trace light reddish brown silty clay to clayey silt matrix, gray, trace black staining, vesicular, fractured, damp to moist (Columbia River Basalt Formation)
4	4.5					
5						
6						Practical Refusal on Soft (R2) Basalt at 5.5 Feet.
7						Note: No seepage or groundwater encountered.
8						
9						
10						
11						
12						

LEGEND

Bag Sample (100 to 1,000 g) · Bucket Sample (5 Gal Bucket) · Shelby Tube Sample · Seepage · Water Bearing Zone · Water Level at Abandonment

Date Excavated: 11/19/2018
Logged By: B. Rapp
Surface Elevation:

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TEST PIT LOG

Project: 23000 & 23010 Bland Circle West Linn, Oregon	Project No. 18-5089	Test Pit No. **TP-2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0 – 0.8						Moderately organic SILT (OL-ML), brown, loose, fine roots throughout, damp to moist (Topsoil Horizon)
1	4.5					Very stiff, silty CLAY (CL) to clayey SILT (ML), trace gray basalt fragments, light reddish brown, subtle orange and gray mottling, trace fine roots to 2.5 feet, damp to moist (Residual Soil)
2	4.5					
3	4.5					
4	4.5					
5						
6						
6.5 – 8						Extremely soft (R0) to very soft (R1), highly weathered BASALT, trace light reddish brown silty clay to clayey silt matrix, gray, trace black staining, vesicular, damp to moist (Columbia River Basalt Formation)
8						Practical Refusal on Soft (R2) Basalt at 8 Feet. Note: No seepage or groundwater encountered.
9						
10						
11						
12						

LEGEND

Bag Sample (100 to 1,000 g) | Bucket Sample (5 Gal Bucket) | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/19/2018
Logged By: B. Rapp
Surface Elevation:

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TEST PIT LOG

Project: 23000 & 23010 Bland Circle West Linn, Oregon	Project No. 18-5089	Test Pit No. **TP-3**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1						Moderately to highly organic SILT (OL-ML), brown, loose, fine roots throughout, damp to moist (Topsoil Horizon)
1	1.0					Very stiff, silty CLAY (CL) to clayey SILT (ML), trace gray basalt fragments below 5 feet, light reddish brown, subtle orange and gray mottling, trace fine roots to 3 feet, damp to moist (Residual Soil)
1.5	4.5					
2	4.5					
3	4.5					
4	4.5					
5						
5.5–6						Soft (R2) BASALT, trace light reddish brown silty clay to clayey silt matrix, gray, trace black staining, vesicular, damp to moist (Columbia River Basalt Formation)
7						Practical Refusal on Soft (R2) to Medium Hard (R3) Basalt at 6 Feet.
8						Note: No seepage or groundwater encountered.
9						
10						
11						
12						

LEGEND

Bag Sample (100 to 1,000 g) | Bucket Sample (5 Gal. Bucket) | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/19/2018
Logged By: B. Rapp
Surface Elevation:

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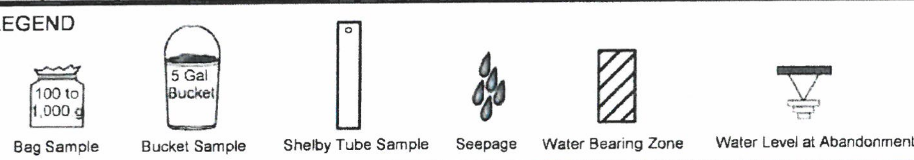
TEST PIT LOG

Project: 23000 & 23010 Bland Circle West Linn, Oregon	Project No. 18-5089	Test Pit No. TP-4

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.6						Moderately organic SILT (OL-ML), dark brown, loose, fine roots throughout, moist (Topsoil Horizon)
1	2.0					Very stiff, silty CLAY (CL) to clayey SILT (ML), trace subrounded to subangular gray basalt fragments below 6 feet, light reddish brown, subtle orange and gray mottling, damp to moist (Residual Soil)
1.5	4.5					
2	4.5					
3	4.5					
4	4.5					
5						
6						
7						
8						Test Pit Terminated at 8 Feet.
9						Note: No seepage or groundwater encountered.
10						
11						
12						

LEGEND



Date Excavated: 11/19/2018
Logged By: B. Rapp
Surface Elevation: