Chapter 8: Stormwater Management Design Examples

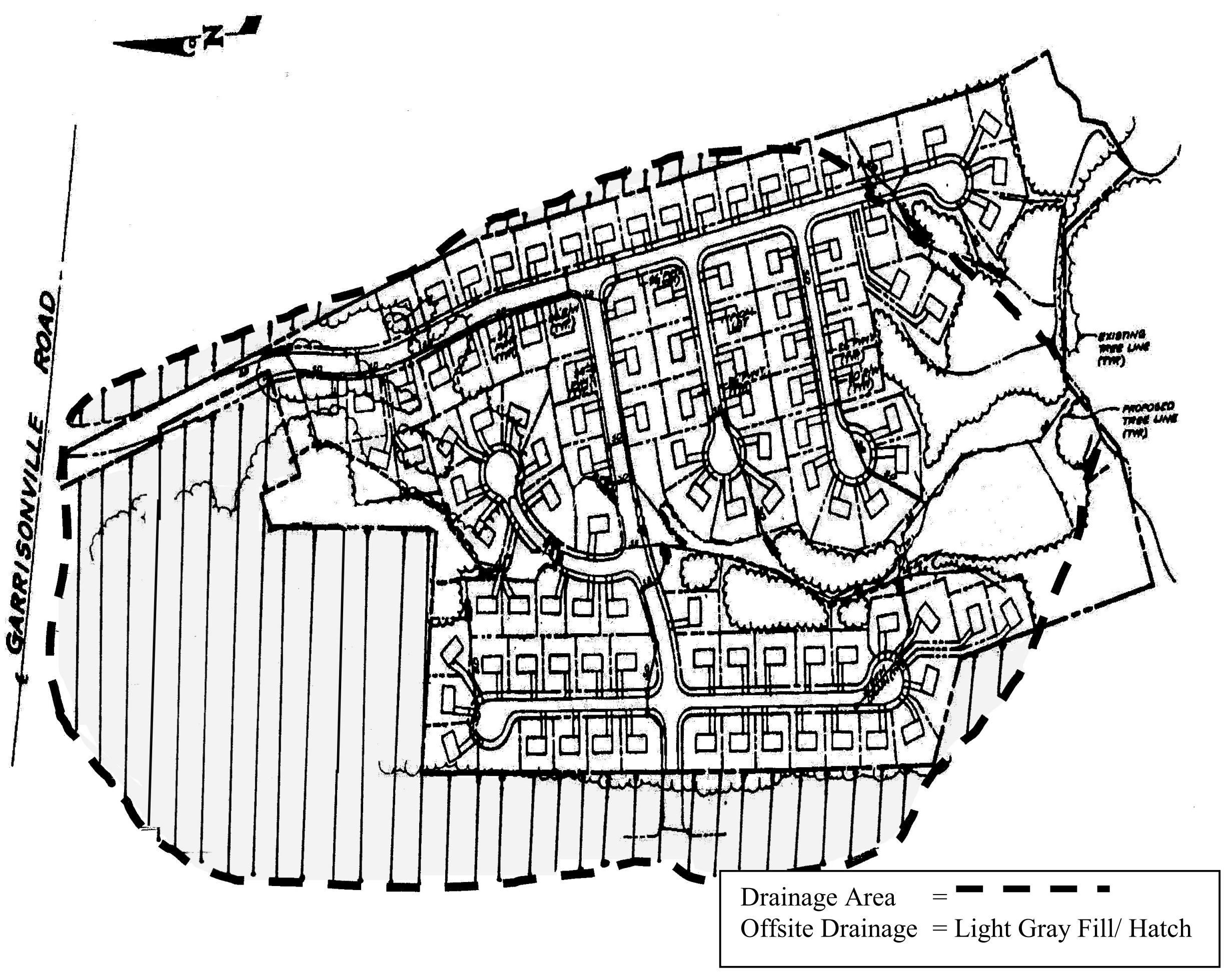
This chapter presents design examples for two hypothetical development sites in the State of New York. The first site, "Stone Hill Estates," is a residential development near Ithaca. The second is a commercial site in Albany. The chapter is divided into five sections, each of which focuses on a particular element of stormwater management design.

- Section 8.1 provides an example of detailed hydrology calculations at the residential site.
- Section 8.2 presents a pond design example based on the hydrology calculated in Section 8.1. This design example demonstrates the hydrologic and hydraulic computations to achieve water quality and water quantity control for stormwater management. Other specific dam design criteria such as soil compaction, structural appurtenances, embankment drainage, outlet design, gates, reservoir drawdown requirements, etc. are stated in Guidelines For Design of Dams.
- This design example in Section 8.2 requires an Article 15 Permit from NYS-DEC since the dam is 15 feet high measured from the top of dam to the low elevation at the downstream outlet, and the storage measured behind the structure to the top of the dam is 2.2 MG.
- Sections 8.3 through 8.5 present design examples for three practices on the commercial site: a sand filter, infiltration trench, and bioretention practice.

Section 8.1 Sizing Example - Stone Hill Estates

Following is a sizing example for the hypothetical "Stone Hill Estates," a 45-acre residential development in Ithaca, New York (Figure 8.1). The site also drains approximately 20 acres of off-site drainage, which is currently in a meadow condition. The site is on mostly C soils with some D soils.

Figure 8.1 Stone Hill Site Plan



<u>Base Data</u>
Location: Ithaca, NY
Site Area = 45.1 ac; Offsite Area = 20.0 ac (meadow)
Total Drainage Area (A) = 65.1
Measured Impervious Area=12.0 ac;
Site Soils Types: 78% "C", 22% "D"
Offsite Soil Type: 100% "C"
Zoning: Residential (½ acre lots)

<u>Hydrologic Data</u>

	Pre	Post	Ult.
CN	72	78	82
t_c (hr)	.46	.35	.35

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

The layout of the Stone Hill subdivision is shown on the previous page.

Water Quality Volume, WQ_v

- <u>Compute Impervious Cover</u>

  Use both on-site and off-site drainage:

  $$I = 12.0 \text{ acres}/65.1 \text{ acres} = 18.4\%$$

- <u>Compute Runoff Coefficient, R_v</u>

  $$R_v = 0.05 + (I)(0.009)$$
  $$= 0.05 + (18.4)(0.009) = 0.22$$

- <u>Compute WQ_v (Includes both on-site and off-site drainage)</u>

Use the 90% capture rule with 0.9" of rainfall. (From Figure 4.1)

$$WQ_v = (0.9")(R_v)(A)$$
$$= (0.9")(0.22)(65.1 \text{ ac})(1\text{ft}/12\text{in})$$
$$= \underline{1.07 \text{ ac-ft}}$$

Establish Hydrologic Input Parameters and Develop Site Hydrology (see Figures 8.2, 8.3, and 8.4)

Condition	**Area**	**CN**	**Tc**
	Ac		*hrs*
Pre-developed	65.1	72	0.46
Post-developed	65.1	78	0.35
Ultimate buildout*	65.1	82	0.35

*Zoned land use in the drainage area.

<u>Hydrologic Calculations</u>

Condition	**$Q_{1\text{-yr}}$**	**$Q_{1\text{-yr}}$**	**$Q_{10\text{-yr}}$**	$Q_{100\text{-yr}}$
Runoff	*inches*	*cfs*	*cfs*	*cfs*
Pre-developed	0.4	19	72	141
Post-developed	0.7	38	112	202
Ultimate buildout	NA	NA	NA	227

PEAK DISCHARGE SUMMARY				
JOB:	STONE HILL			EWB
DRAINAGE AREA NAME:	PRE DEVELOPMENT			21-Jan-97
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	Curve Number	AREA (In acres)
MEADOW		C	71	20.25 Ac.
MEADOW		D	78	7.95 Ac.
WOOD		C	70	15.09 Ac.
WOOD		D	77	1.81 Ac.
OFF-SITE MEADOW		C	71	20.00 Ac.
			AREA SUBTOTALS:	65.10 Ac.

Time of Concentration 2-Yr 24 Hr Rainfall = 2.7 In	Surface Cover Cross Section	Manning 'n' Wetted Per	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	dense grass	'n'=0.24	150 Ft.	3.80% 0.28 Hrs
Shallow Flow	UNPAVED		1300 Ft. 2.65 F.P.S.	2.70% 0.14 Hrs.
Channel Flow Hydraulic Radius =1.26	22.0 SqFt	'n'=0.040 17.5 Ft.	1100 Ft. 7.14 F.P.S.	2.70% 0.04 Hrs.

Total Area in Acres =	65.10 Ac.	Total Sheet Flow=	Total Shallow Flow=	Total Channel Flow =
Weighted CN =	72			
Time Of Concentration =	0.46 Hrs.	0.28 Hrs.	0.14 Hrs.	0.04 Hrs.
Pond Factor =	1	RAINFALL TYPE II		

STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.3 In.	0.4 In.	18.6 CFS	101,195 Cu. Ft.
2 Year	2.7 In.	0.6 In.	30.2 CFS	150,257 Cu. Ft.
10 Year	3.9 In.	1.4 In.	72 CFS	328,570 Cu. Ft.
100 Year	5.5 In.	2.6 In.	141 CFS	611,958 Cu. Ft.

Figure 8.2 Stone Hill Pre-Development Conditions

PEAK DISCHARGE SUMMARY				
JOB:	STONE HILL			EWB
DRAINAGE AREA NAME:	POST DEVELOPMENT			21-Jan-97
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	Curve	AREA (In acres)
MEADOW		C	71	0.16 Ac.
MEADOW		D	78	0.14 Ac.
WOOD		C	70	3.09 Ac.
WOOD		D	77	1.81 Ac.
IMPERVIOUS			98	12.00 Ac.
GRASS		C	74	20.09 Ac.
GRASS		D	80	7.81 Ac.
OFFSITE MEADOW		C	71	20.00 Ac.
		AREA SUBTOTALS:		65.10 Ac.
Time of Concentration 2-Yr 24 Hr Rainfall = 2.7 In	Surface Cover Cross Section	Manning 'n' Wetted Per	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	dense grass	'n'=0.24	100 Ft.	3.80% 0.20 Hrs
Shallow Flow (a)	UNPAVED		100 Ft. 1.98 F.P.S.	1.50% 0.01 Hrs.
(b)	PAVED		400 Ft. 2.03 F.P.S.	1.00% 0.05 Hrs.
Channel Flow (a) Hydraulic Radius =0.50	1.6 SqFt	'n'=0.013 3.2 Ft.	1550 Ft. 7.22 F.P.S.	1.00% 0.06 Hrs.
(b) Hydraulic Radius =1.42	12.0 SqFt	'n'=0.030 8.5 Ft.	350 Ft. 13.01 F.P.S.	4.30% 0.01 Hrs.
(c) Hydraulic Radius =1.26	22.0 SqFt	'n'=0.040 8.5 Ft.	300 Ft. 7.89 F.P.S.	3.30% 0.01 Hrs.
Total Area in Acres =	65.10 Ac.	Total Sheet Flow=	Total Shallow Flow=	Total Channel Flow =
Weighted CN =	78			
Time Of Concentration =	0.35 Hrs.	0.20 Hrs.	0.07 Hrs.	0.08 Hrs.
Pond Factor =	1	RAINFALL TYPE II		
STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.3 In.	0.7 In.	37.6 CFS	156,283 Cu. Ft.
2 Year	2.7 In.	0.9 In.	54.0 CFS	217,511 Cu. Ft.
10 Year	3.9 In.	1.8 In.	112 CFS	427,155 Cu. Ft.
100 Year	5.5 In.	3.1 In.	202 CFS	742,265 Cu. Ft.

Figure 8.3 Stone Hill Post-Development Conditions

	PEAK DISCHARGE SUMMARY			
JOB:	**STONE HILL**			EWB
DRAINAGE AREA NAME:	**ULTIMATE BUILDOUT**			**21-Jan-97**
COVER DESCRIPTION	**SOIL NAME**	**GROUP A,B,C,D?**	**Curve**	**AREA (In acres)**
MEADOW		C	71	0.16 Ac.
MEADOW		D	78	0.14 Ac.
WOOD		C	70	3.09 Ac.
WOOD		D	77	1.81 Ac.
IMPERVIOUS			98	12.00 Ac.
GRASS		C	74	20.09 Ac.
GRASS		D	80	7.81 Ac.
OFFSITE ULTIMATE				
SF RES (0.25 AC LOTS)		C	83	20.00 Ac.
			AREA SUBTOTALS:	**65.10 Ac.**
Time of Concentration	**Surface Cover**	**Manning 'n'**	**Flow Length**	**Slope**
2-Yr 24 Hr Rainfall = 2.7 In	**Cross Section**	**Wetted Per**	**Avg Velocity**	**Tt (Hrs)**
Sheet Flow	dense grass	**'n'=0.24**	**100 Ft.**	**3.80%**
				0.20 Hrs
Shallow Flow	**UNPAVED**		**100 Ft.**	**1.50%**
(a)			**1.98 F.P.S.**	**0.01 Hrs.**
	PAVED		**400 Ft.**	**1.00%**
(b)			**2.03 F.P.S.**	**0.05 Hrs.**
Channel Flow (a)		**'n'=0.013**	**1550 Ft.**	**1.00%**
Hydraulic Radius =0.50	1.6 SqFt	3.2 Ft.	**7.22 F.P.S.**	**0.06 Hrs.**
(b)		**'n'=0.030**	**350 Ft.**	**4.30%**
Hydraulic Radius =1.42	12.0 SqFt	8.5 Ft.	**13.01 F.P.S.**	**0.01 Hrs.**
(c)		**'n'=0.040**	**300 Ft.**	**3.30%**
Hydraulic Radius =1.26	22.0 SqFt	8.5 Ft.	**7.89 F.P.S.**	**0.01 Hrs.**
Total Area in Acres =	**65.10 Ac.**	**Total Sheet**	**Total Shallow**	**Total Channel**
Weighted CN =	**82**	**Flow=**	**Flow=**	**Flow =**
Time Of Concentration =	**0.35 Hrs.**	**0.20 Hrs.**	**0.07 Hrs.**	**0.08 Hrs.**
Pond Factor =	**1**	**RAINFALL TYPE II**		
STORM	***Precipitation* (P) inches**	***Runoff* (Q)**	**Qp, PEAK DISCHARGE**	**TOTAL STORM Volumes**
1 Year	**2.3 In.**	**0.9 In.**	**50.9 CFS**	201,772 Cu. Ft.
2 Year	**2.7 In.**	**1.1 In.**	**70.0 CFS**	271,097 Cu. Ft.
10 Year	**3.9 In.**	**2.1 In.**	**135 CFS**	500,458 Cu. Ft.
100 Year	**5.5 In.**	**3.5 In.**	**227 CFS**	834,167 Cu. Ft.

Figure 8.4 Stone Hill Ultimate Buildout Conditions

Compute Stream Channel Protection Volume, (Cp_v) (see Section 4.3 and Appendix B)

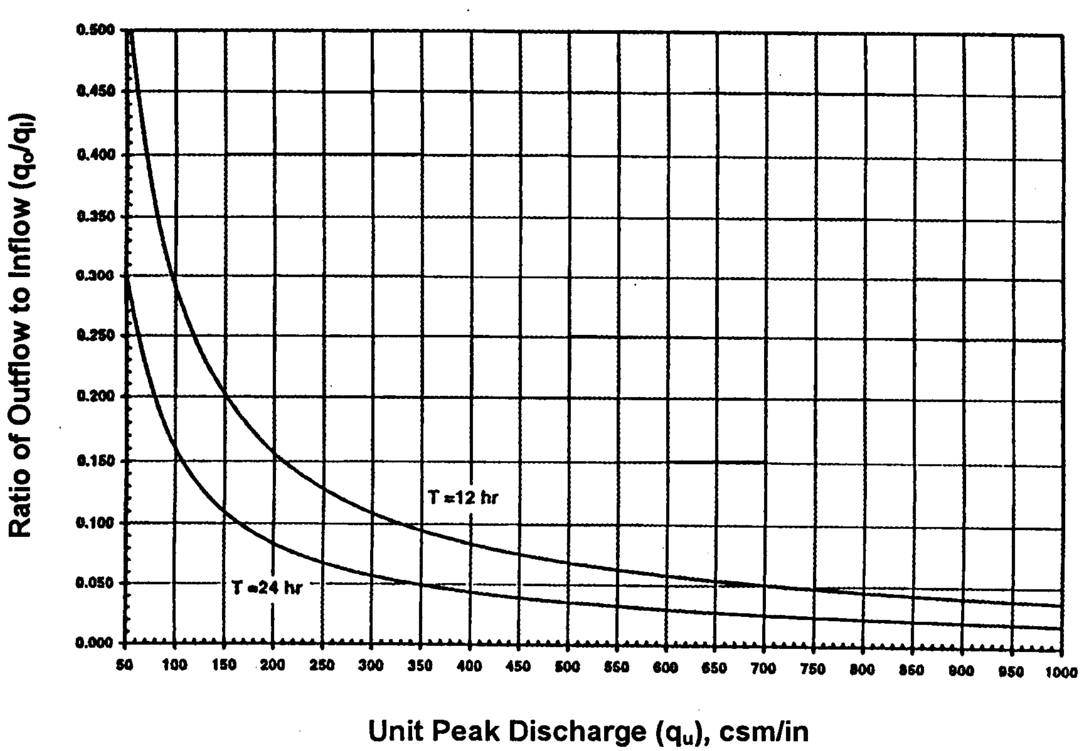
For stream channel protection, provide 24 hours of extended detention (T) for the one-year event.

<u>Compute Channel Protection Storage Volume</u>

First, determine the value of the unit peak discharge (q_u) using TR-55 and Type II Rainfall Distribution

- Initial abstraction (Ia) for CN of 78 is 0.564: [Ia = (200/CN - 2)]
- Ia/P = (0.564)/ 2.3 inches = 0.245
- T_c = 0.35 hours
- Using the above data, q_u = 570 csm/in (cubic feet per second per square mile per year)

Figure 8.5 Detention Time vs. Discharge Ratios (Source: MDE, 2000)



- Knowing q_u and T = 24 hours, find q_o/q_i using Figure 8.5 (also see methodology in Appendix B)
- Peak outflow discharge/peak inflow discharge $(q_o/q_i) = 0.035$
- $Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$ (from Appendix B)

  Where Vs equals channel protection storage (Cp_v) and Vr equals the volume of runoff in inches.
- Vs/Vr = 0.63 and, from figure 8.3, Q = 0.7"
- Solving for V_s

  $Vs = Cp_v = 0.63(0.7")(1/12)(65.1\ ac) = 2.4$ ac-ft (104,214 cubic feet)

<u>Define the Average Release Rate</u>

- The above volume, 2.4 ac-ft, is to be released over 24 hours
- $(2.4\ ac\text{-}ft \times 43{,}560\ ft^2/ac) / (24\ hrs \times 3{,}600\ sec/hr) = 1.2$ cfs

Compute Overbank Flood Protection Volume, (Qp_{10}) (see Section 4.4)

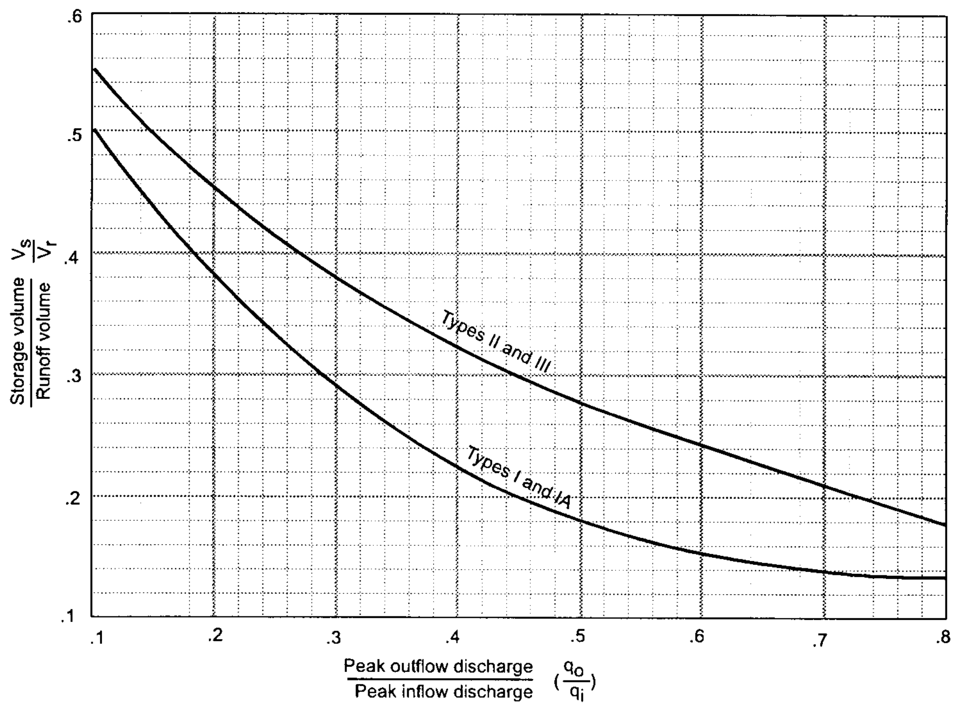
For both the overbank flood protection volume and the extreme flood protection volume, size is determined using the TR-55 "Short-Cut Method," which relates the storage volume to the required reduction in peak flow and storm inflow volume (Figure 8.6).

- For a q_i of of 112 cfs (post-developed), and an allowable q_o of 72 cfs (pre-developed), the value of $(q_o)/(q_i)$ is 0.64
- Using figure 8.6, and a post-developed curve number of 78, Vs/Vr = 0.23
- Using a total storm runoff volume of 427,155 cubic feet (9.8 acre-feet), the required storage (Vs) is:

  $V_s = Qp_v = 0.23(427{,}155)/43{,}560 = 2.26$ acre-feet

Figure 8.6 Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III

Source: TR-55, 1986



While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 10-year storm. So, for preliminary sizing purposes, add 15% to the required volume for the 10-year storm. $Q_{p\text{-}10} = 2.23 \times 1.15 = 2.59$ ac-ft.

Compute Extreme Flood Protection Volume, (Q_f)

Extreme flood protection is calculated using the same methodology as overbank protection.

- For a Q_{in} of, and an allowable Q_{out} of, and a runoff volume of the Vs necessary for 100-year control is, under a developed CN of 78. Note that 5.5 inches of rain fall during this event, with approximately 3.1 inches of runoff.
- While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 100-year storm. So, for preliminary sizing purposes add 15% to the required volume for the 100-year storm. $Q_{f\text{-}100} = 3.53 \times 1.15 = 4.06$ ac-ft.

Analyze Safe Passage of 100-Year Design Storm (Qf)

If peak discharge control of the 100-year storm is not required, it is still necessary to provide safe passage for the 100-year event under ultimate buildout conditions (Q_{ult} = 227 cfs).

Section 8.2 Pond Design Example

Following is a step-by-step design example for an extended detention pond (P-3) applied to Stone Hill Estates, which is described in detail in Section 8.1 along with design treatment volumes. This example continues with the design to develop actual design parameters for the constructed facility.

Step 1. Compute preliminary runoff control volumes.

The volume requirements were determined in Section 8.1. Table 8.1 provides a summary of the storage requirements.

Table 8.1. Summary of General Storage Requirements for Stone Hill Estates

Symbol	*Category*	Volume Required (ac- ft)	*Notes*
WQ_v	Water Quality Volume	1.07	
Cp_v	Stream Protection	2.4	Average ED release rate is 1.2 cfs over 24 hours
Q_p	Peak Control	2.6	10-year, in this case
Q_f	Flood Control	4.1	

Step 2. Determine if the development site and conditions are appropriate for the use of a stormwater pond.

The drainage area to the pond is 65.1 acres. Existing ground at the proposed pond outlet is 619 MSL. Soil boring observations reveal that the seasonally high water table is at elevation 618. The underlying soils are SC (sandy clay) and are suitable for earthen embankments and to support a wet pond without a liner. The stream invert at the adjacent stream is at elevation 616.

Step 3. Confirm local design criteria and applicability.

There are no additional requirements for this site.

Step 4. Determine pretreatment volume.

Size wet forebay to treat 10% of the WQ_v. (10%)(1.07 ac-ft) = **0.1 ac-ft**
(forebay volume is included in WQ_v as part of permanent pool volume)

Step 5. Determine permanent pool volume and ED volume.

Size permanent pool volume to contain 50% of WQ_v:
$0.5 \times$ (1.07 ac-ft) = **0.54 ac-ft**. (includes 0.1 ac-ft of forebay volume)

Size ED volume to contain 50% of WQ_v: $0.5 \times$ (1.07 ac-ft) = **0.54 ac-ft**

NOTE

THIS DESIGN APPROACH ASSUMES THAT ALL OF THE ED VOLUME WILL BE IN THE POND AT ONCE. WHILE THIS WILL NOT BE THE CASE, SINCE THERE IS A DISCHARGE DURING THE EARLY STAGES OF STORMS, THIS CONSERVATIVE APPROACH ALLOWS FOR ED CONTROL OVER A WIDER RANGE OF STORMS, NOT JUST THE TARGET RAINFALL.

Step 6. Determine pond location and preliminary geometry. Conduct pond grading and determine storage available for WQv permanent pool and WQv-ED if applicable.

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond. Storage must be provided for the permanent pool (including sediment forebay), extended detention (WQ_v-ED), Cp_v-ED, 10-year storm, 100-year storm, plus sufficient additional storage to pass the ultimate condition 100-year storm with required freeboard. An elevation-storage table and curve is prepared using the average area method for computing volumes. See Figure 8.7 for pond location on site, Figure 8.8 for grading and Figure 8.9 for Elevation-Storage Data.

Figure 8.7 Pond Location on Site

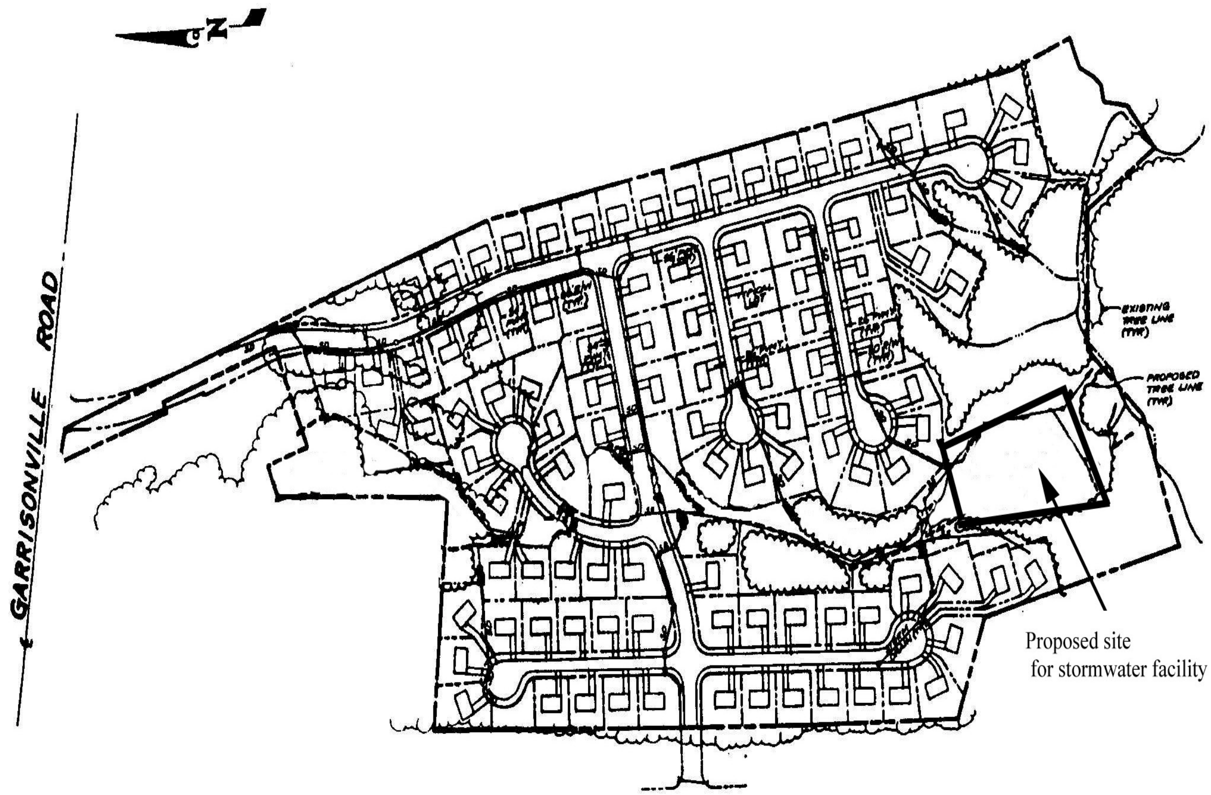


Figure 8.8 Plan View of Pond Grading (Not to Scale)

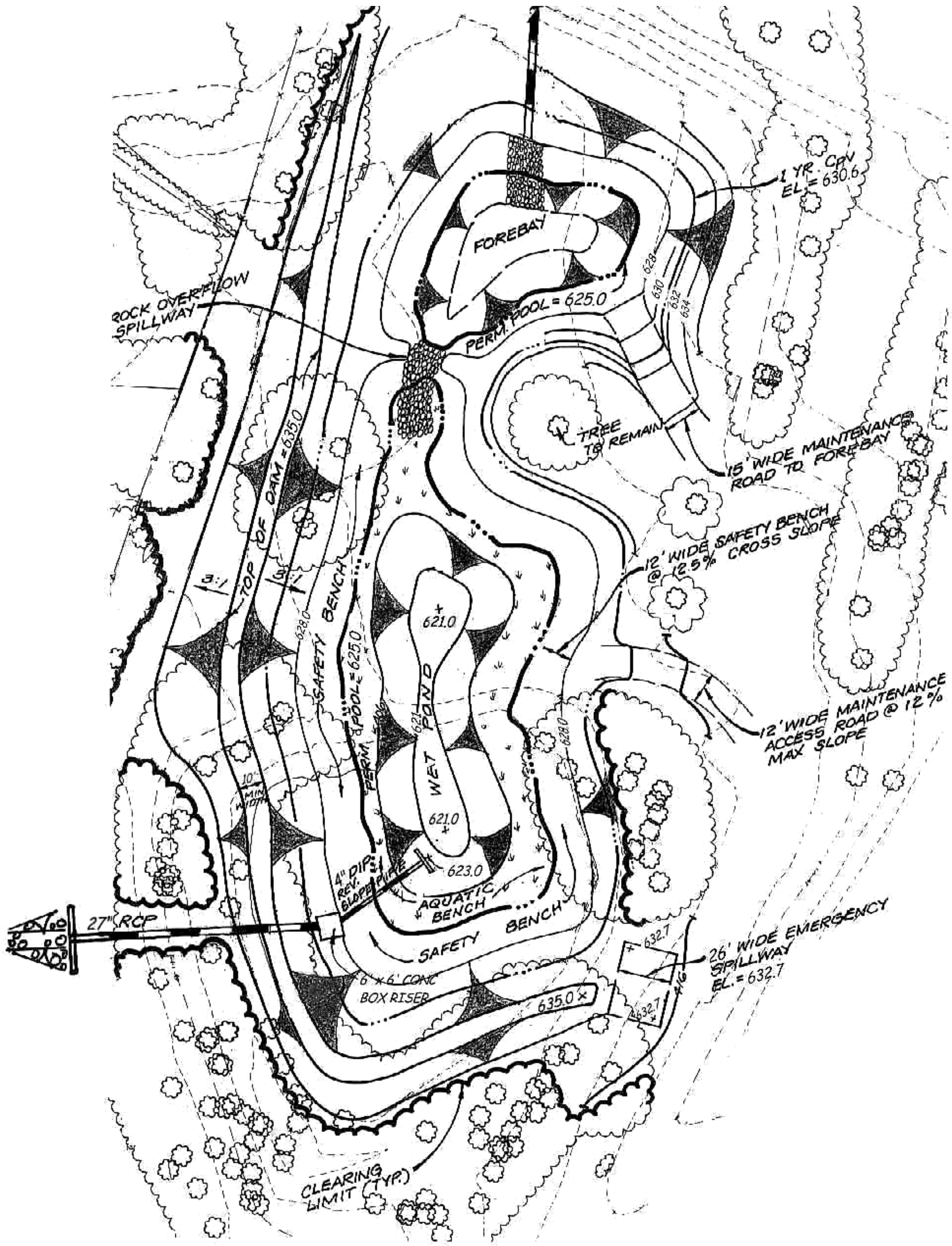
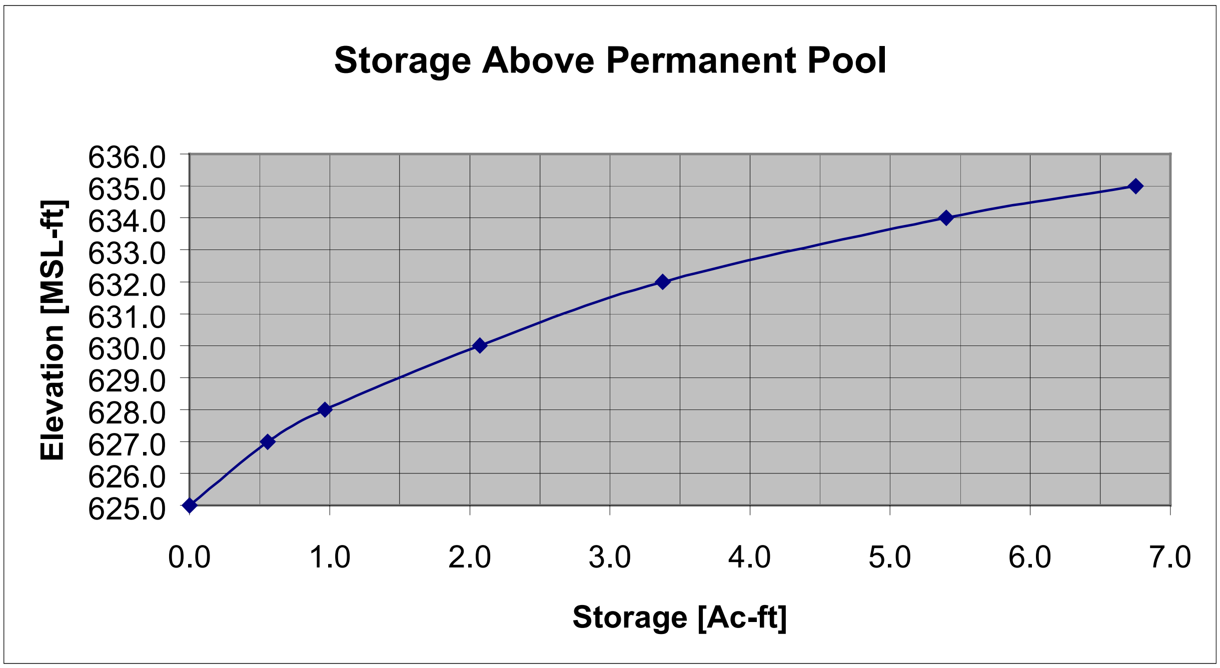


Figure 8.9 Storage-Elevation Table/Curve

Elevation MSL	Area ft^2	Average Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Cumulative Volume ac-ft	Volume Above Permanent Pool ac-ft
621.0	3150						
624.0	8325	5738	3	17213	17213	0.40	
625.0	10400	9363	1	9363	26575	0.61	0
627.0	13850	12125	2	24250	50825	1.17	0.56
628.0	21850	17850	1	17850	68675	1.58	0.97
630.0	26350	24100	2	48200	116875	2.68	2.07
632.0	30475	28413	2	56825	173700	3.99	3.38
634.0	57685	44080	2	88160	261860	6.01	5.40
635.0	60125	58905	1	58905	320765	7.36	6.75



Set basic elevations for pond structures

- The pond bottom is set at elevation 621.0
- Provide gravity flow to allow for pond drain, set riser invert at 620.5
- Set barrel outlet elevation at 620.0

Set water surface and other elevations

- Required permanent pool volume = 50% of WQv = 0.54 ac-ft. From the elevation-storage table, read elevation 625.0 (0.61 ac-ft > 0.54 ac-ft) site can accommodate it and it allows a small safety factor for fine sediment accumulation – OK

Set permanent pool wsel = 625.0

- Forebay volume provided in single pool with volume = 0.1 ac-ft - OK
- Required extended detention volume (WQ_v-ED)= 0.54 ac-ft. From the elevation-storage table (volume above permanent pool), read elevation 627.0 (0.56 ac-ft > 0.54 ac-ft) OK. Set ED wsel = 627.0

Note: Total storage at elevation 627.0 = 1.17 ac-ft (greater than required WQ_v of 1.07 ac-ft)

Compute the required WQ_v-ED orifice diameter to release 0.54 ac-ft over 24 hours

- Avg. ED release rate = (0.54 ac-ft)(43,560 ft^2/ac)/(24 hr)(3600 sec/hr) = 0.27 cfs
- Invert of orifice set at wsel = 625.0
- Average head = (627.0 - 625.0)/ 2 = 1.0'
- Use orifice equation to compute cross-sectional area and diameter

  $Q = CA(2gh)^{0.5}$, for Q=0.27 cfs h = 1.0 ft; C = 0.6 = discharge coefficient. Solve for A

  $A = 0.27 \text{ cfs} / [(0.6)((2)32.2 \text{ ft/s}^2)(1.0 \text{ ft}))^{0.5}]$ $A = 0.057 \text{ ft}^2$, $A = \pi d^2 / 4$;

  dia. = 0.26 ft = 3.2", say 3.0 inches

  <u>Use 4" pipe with 4" gate valve to achieve equivalent diameter</u>

Compute the stage-discharge equation for the 3.0" dia. WQv orifice

- $Q_{WQv\text{-}ED} = CA(2gh)^{0.5} = (0.6)(0.052 \text{ ft}^2)[((2)(32.2 \text{ ft/s}^2))^{0.5}](h^{0.5})$,
- <u>$Q_{WQv\text{-}ED} = (0.25)\, h^{0.5}$, where: h = wsel - 625.125</u>

  (Note: Account for one half of orifice diameter when calculating head)

Step 7. Compute ED orifice size, and compute release rate for Cpv-ED control and establish Cpv elevation.

Set the Cp_v pool elevation

- Required Cp_v storage = 2.4 ac-ft (see Table 1).
- From the elevation-storage table, read elevation 630.6 (this includes the WQ_v).
- Set Cp_v wsel = 630.6

Size Cp_v orifice

- Size to release average of 1.2 cfs.
- Set invert of orifice at wsel = 627.0
- Average WQ_v-ED orifice release rate is 0.41 cfs, based on average head of 2.74' ((630.6 – 625.125)/2)
- Cp_v-ED orifice release = 1.2 -0.41 = 0.79 cfs
- Head = (630.6 - 627.0)/2 = 1.8'

Use orifice equation to compute cross-sectional area and diameter

- $Q = CA(2gh)^{0.5}$, for h = 1.8'
  - $A = 0.79 \text{ cfs} / [(0.6)((2)(32.2'/s^2)(1.8'))^{0.5}]$
  - $A = 0.12 \text{ ft}^2$, $A = \pi d^2 / 4$;
  - dia. = 0.39 ft = 4.7"
  - Use 6" pipe with 6" gate valve to achieve equivalent diameter

Compute the stage-discharge equation for the 4.7" dia. Cp_v orifice

- $Q_{Cpv\text{-}ED} = CA(2gh)^{0.5} = (0.6)\ (0.12 \text{ ft}^2)\ [((2)\ (32.2'/s^2))^{0.5}]\ (h^{0.5})$,
- $Q_{Cpv\text{-}ED} = (0.58)\ (h^{0.5})$, where: h = wsel – 627.2

(Note: Account for one half of orifice diameter when calculating head)

Step 8. Calculate Qp_{10} (10 year storm) release rate and water surface elevation.

In order to calculate the 10 year release rate and water surface elevation, the designer must set up a stage-storage-discharge relationship for the control structure for each of the low flow release pipes (WQ_v-ED and Cp_v-ED) plus the 10 year storm.

Develop basic data and information

- The 10 year pre-developed peak discharge = 72 cfs,
- The post developed inflow = 112 cfs, from Table 1,
- From previous estimate Q_{p-10} = 2.26 ac-ft. Adding 15% to account for ED storage yields a preliminary volume of 2.56 ac-ft.
- From elevation-storage table (Figure 8.9), read elevation 631.0.
- Size 10 year slot to release 72 cfs at elevation 631.0.

@ wsel 631.0:

- WQ_v-ED orifice releases 0.61 cfs,
- Cp_v-ED orifice releases 1.13 cfs, therefore;
- Allowable Q_{p-10} = 72 cfs - (.61 + 1.13) = 70.26 cfs, say 70.3 cfs.
- Set weir crest elevation at wsel = 630.6
- Max head = (631.0 – 630.6) = 0.4'

Use weir equation to compute slot length

- $Q = CLh^{3/2}$
- $L = 70.3 \text{ cfs} / (3.1)(0.4^{3/2}) = 89.6 \text{ ft}$
- This weir length is impractical, so adjust max head (and therefore slot height) to 1.5' and recalculate weir length.
- $L = 70.3 \text{ cfs} / (3.1)(1.5^{3/2}) = 12.3 \text{ ft}$
- <u>Use three 5ft x 1.5 ft slots for 10-year release</u> (opening should be slightly larger than needed so as to have the barrel control before slot goes from weir flow to orifice flow).
- Maximum $Q = (3.1)(15)(1.5)^{3/2} = 85.4$ cfs

Check orifice equation using cross-sectional area of opening

- $Q = CA(2gh)^{0.5}$, for h = 0.75' (For orifice equation, h is from midpoint of slot)
- $A = 3\,(5.0')\,(1.5') = 22.5\text{ft}^2$
- $Q = 0.6\,(22.5\text{ft}^2)\,[(64.4)(0.75)]^{0.5} = 93.8$ cfs > 85.4 cfs, so use weir equation

$$Q_{10} = (3.1)(15')\,h^{3/2}\ ,\ \underline{Q_{10} = (46.5)\,h^{3/2}\text{, where } h = \text{wsel} - 630.6}$$

- Size barrel to release approximately 70.3 cfs at elevation 632.1 (630.6 + 1.5)
- Check inlet condition: (use FHWA culvert charts)

H_w = 632.1-620.5 = 11.6 ft

- Try 27" diameter RCP, Using FHWA Chart ("Headwater Depth for Concrete Pipe Culverts with Inlet Control") with entrance condition 1
- H_w / D = 11.6/2.25 = 5.15, Discharge = 69 cfs
- Check outlet condition (use NRCS pipe flow equation from NEH Section 5 ES-42):
- $Q = a\,[(2gh)/(1+k_m+k_pL)]^{0.5}$

where:

	Q	=	discharge in cfs
	a	=	pipe cross sectional area in ft^2
	g	=	acceleration of gravity in ft/sec^2
	h	=	head differential (wsel - downstream centerline of pipe or tailwater elev.)
	k_m	=	coefficient of minor losses (use 1.0)
	k_p	=	pipe friction loss coef. (= $5087n^2/d^{4/3}$, d in inches, n is Manning's n)
	L	=	pipe length in ft

h = 632.1 - (620.0 + 1.125) = 10.98'

for 27" RCP, approximately 70 feet long:

$Q = 4.0\,[(64.4)\,(10.98) / (1+1+(0.0106)\,(70))]^{0.5} = 64.2$ cfs

64.2 cfs < 69 cfs, so barrel is outlet controlled and use outlet equation

$Q = 19.4\,(h)^{0.5}$, where h = wsel – 621.125

Note: pipe will control flow before high stage inlet reaches max head.

Complete stage-storage-discharge summary (Figure 8.10) up to preliminary 10-year wsel (632.1) and route 10 year post-developed condition inflow using computer software (e.g., TR-20). Pond routing computes 10-year wsel at 632.5 with discharge = 65.4 cfs < 72 cfs, OK (see Figure 8.11).

Figure 8.10 Stage-Storage-Discharge Summary

Elevation MSL	Storage ac-ft	Low Flow WQv-ED 3.0" eq dia		Riser Cpv-ED 4.7" eq. dia		Riser High Stage Slot Orifice		Riser High Stage Slot Weir		27" Barrel Inlet		27" Barrel Pipe		Emergency Spillway 26' earthen 3:1		Total Discharge
		H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	Q cfs
625.0	0.00	0	0													0.00
625.5	0.14	0.4	0.15													0.15
626.0	0.28	0.9	0.23													0.23
626.5	0.42	1.4	0.29													0.29
627.0	0.56	1.9	0.34	0.0	0.00											0.34
627.5	0.77	2.4	0.39	0.3	0.32											0.70
628.0	0.97	2.9	0.42	0.8	0.52											0.94
629.0	1.52	3.9	0.49	1.8	0.78											1.27
629.5	1.80	4.4	0.52	2.3	0.88											1.40
630.0	2.07	4.9	0.55	2.8	0.97											1.52
630.6	2.40	5.5	0.58	3.4	1.07	-	-	0.0	0.0							1.65
631.0	2.73	5.9	0.61	3.8	1.13	-	-	0.4	11.8							13.5
632.1	3.45	7.0	0.66	4.9	1.28	0.75	94	1.5	85.4	11.6	69.0	11.0	64.2			64.2
632.5	3.80	7.4	0.68	5.3	1.34	0.95	106	-	-	12.0	70.0	11.4	65.4			65.4
632.7	4.10	7.6	0.69	5.5	1.36	1.05	111	-	-	12.2	71.0	11.6	66.0	0.0	0.0	66.0
633.3	4.70	-	-	-	-	-	-	-	-	12.8	72.0	12.2	67.6	0.6	26.0	93.6
634.0	5.40	-	-	-	-	-	-	-	-	13.5	73.0	12.9	69.6	1.3	95.0	164.6
635.0	6.75	-	-	-	-	-	-	-	-	14.5	86.0	13.9	72.2	2.3	251.0	323.2

Note: Adequate outfall protection must be provided in the form of a riprap channel, plunge pool, or combination to ensure non-erosive velocities.

Step 9. Calculate Qp_{100} (100-year storm) release rate and water surface elevation, size emergency spillway, calculate 100-year water surface elevation.

In order to calculate the 100-year release rate and water surface elevation, the designer must continue with the stage-storage-discharge relationship (Figure 8.10) for the control riser and emergency spillway.

Develop basic data and information

- The 100 year pre-developed peak discharge = 141 cfs,
- The post developed inflow = 202 cfs, from Table 1,
- From previous estimate $Q_{p\text{-}100}$ = 3.53 ac-ft. Adding 15% to account for ED storage yields a preliminary volume of 4.06 ac-ft.
- From elevation-storage table (Figure 8.10), read elevation 632.8, say 633.0.

The 10-year wsel is at 632.5. Set the emergency spillway at elevation at 632.7 (this allows for some additional storage above the 10-yr wsel) and use design information and criteria for Earth Spillways (not included in this manual).

- Size 100 year spillway to release 141 cfs at elevation 633.0.
- @ wsel 633.0:

- Outflow from riser structure is controlled by barrel (under outlet control), from Figure 8.11, read Q = 67.6 cfs at wsel = 633.3. Assume Q = 67 cfs at wsel = 633.0.
- Set spillway invert at wsel = 632.7
- Try 26' wide vegetated emergency spillway with 3:1 side slopes.
- Finalize stage-storage-discharge relationships and perform pond routing

Pond routing (TR-20) computes 100-year wsel at 633.76 with discharge = 140.95 cfs < 141 cfs, OK (see Figure 8.11).

Note: this process of sizing the emergency spillway and storage volume determination is usually iterative. This example reflects previous iterations at arriving at an acceptable design solution.

Step 10. Check for safe passage of Qp_{100} under ultimate buildout conditions and set top of embankment elevation.

The safety design of the pond embankment requires that the 100-year discharge, based on ultimate buildout conditions be able to pass safely through the emergency spillway with sufficient freeboard (one foot). This criteria does not mean that the ultimate buildout peak discharge be attenuated to pre-development rates.

From previous hydrologic modeling we know that:

- The 100 year ultimate buildout peak discharge = 227 cfs,
- The ultimate buildout composite curve number is 82.

Using TR-20 or equivalent routing model, determine peak wsel. Pond routing computes 100-year wsel at 634.0 with discharge = 192 cfs (Figure 8.12).

Therefore, with one foot of freeboard, the minimum embankment elevation is 635.0. Table 8.2 provides a summary of the storage, stage, and discharge relationships determined for this design example. See Figure 8.13 for a schematic of the riser.

Table 8.2 Summary of Controls Provided					
Control Element	**Type/Size of Control**	**Storage Provided**	**Elevation**	**Discharge**	**Remarks**
Units		Acre-feet	MSL	cfs	
Permanent Pool		0.61	625.0	0	part of WQ_v
Forebay	submerged berm	0.1	625.0	0	included in permanent pool vol.
Extended Detention (WQ_v-ED)	4" pipe, sized to 3.0" equivalent diameter	0.56	627.0	0.25	part of $WQ_{V.}$, vol. above perm. pool, discharge is average release rate over 24 hours
Channel Protection (Cp_v-ED)	6" pipe sized to 4.7" equivalent diameter	2.4	630.6	1.2	volume above perm. pool, discharge is average release rate over 24 hours
Overbank Protection (Q_{p-10})	Three 5’ x 1.5’ slots on a 6’ x 6’ riser, 30"barrel.	2.5	632.5	65.4	volume above perm. pool
Extreme Storm (Q_{f-100})	26' wide earth spillway	4.0	633.8	140.9	volume above perm. pool
Extreme Storm Ultimate Buildout	26' wide earth spillway	NA	634	192.0	Set minimum embankment height at 635.0

Figure 8.11 TR-20 Model Input and Output

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20                    FULLPRINT                        NOPLOTS
TITLE      New York Manual Wet ED Example 1/01          EWB
TITLE      Post Developed Conditions Routing for 1, 10, and 100
 3 STRUCT       1
 8                      625.0        0.0          0.0
 8                      625.5        0.15         0.14
 8                      626.0        0.23         0.28
 8                      626.5        0.29         0.42
 8                      627.0        0.34         0.56
 8                      627.5        0.70         0.77
 8                      628.0        0.94         0.97
 8                      629.0        1.27         1.52
 8                      629.5        1.40         1.80
 8                      630.0        1.52         2.07
 8                      630.6        1.65         2.40
 8                      631.0        13.50        2.73
 8                      632.1        64.20        3.45
 8                      632.7        66.00        4.10
 8                      633.3        93.60        4.70
 8                      634.0        165.0        5.40
 8                      635.0        35230        6.75
 9 ENDTBL
 6 RUNOFF 1     1     2 0.102        78.0         0.35         1 1   0 0 1
 6 RESVOR 2     1 2   3 625.0                                  1 1       1
   ENDATA
 7 INCREM 6             0.1
 7 COMPUT 7     1     1 0.0          2.3          1.0          2 2   1  01
   ENDCMP 1
 7 COMPUT 7     1     1 0.0          3.9          1.0          2 2   1  10
   ENDCMP 1
 7 COMPUT 7     1     1 0.0          5.5          1.0          2 2   1  99
   ENDCMP 1
   ENDJOB 2

*******************************END OF 80-80 LIST********************************
```

```
TR20 XEQ  1/22/**              New York Manual Wet ED Example 1/01         EWB                                      JOB  1   SUMMARY
     REV 09/01/83              Post Developed Conditions Routing for 1, 10, and 100                                          PAGE   8
```

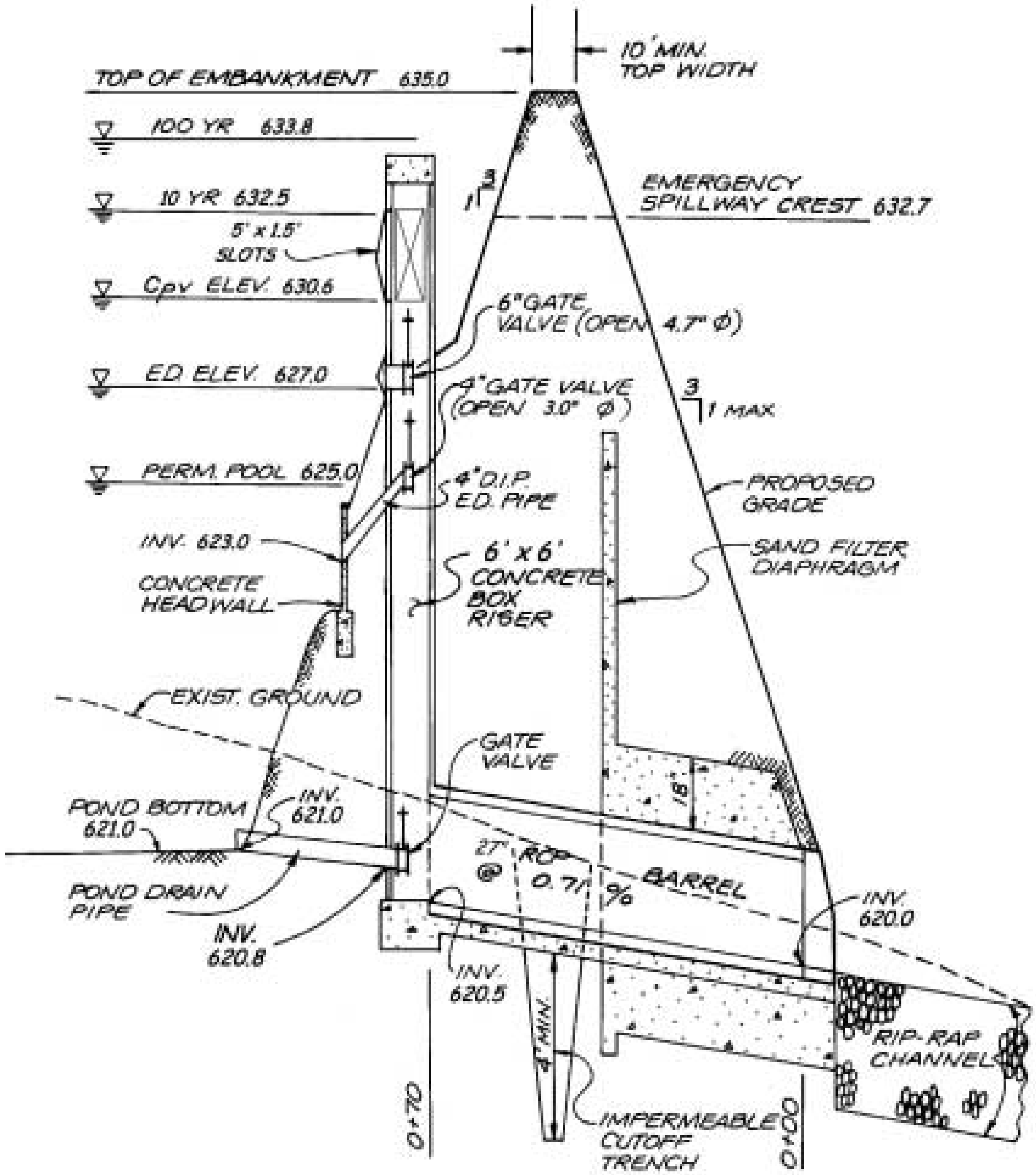
SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE CSM)
ALTERNATE 1 STORM 1													
STRUCTURE 1	RUNOFF	.10	2	2	.10	.0	2.30	24.00	.66	---	12.13	40.62	398.2
STRUCTURE 1	RESVOR	.10	2	2	.10	.0	2.30	24.00	.40	630.31	18.00?	1.59?	15.6
ALTERNATE 1 STORM 10													
STRUCTURE 1	RUNOFF	.10	2	2	.10	.0	3.90	24.00	1.81	---	12.11	118.47	161.5
STRUCTURE 1	RESVOR	.10	2	2	.10	.0	3.90	24.00	1.49	632.51	12.34	65.43	41.5
ALTERNATE 1 STORM 99													
STRUCTURE 1	RUNOFF	.10	2	2	.10	.0	5.50	24.00	3.14	---	12.11	206.59	025.4
STRUCTURE 1	RESVOR	.10	2	2	.10	.0	5.50	24.00	2.80	633.76	12.29	140.95	381.9

Figure 8.12 TR-20 Model Input and Output for Ultimate Buildout Conditions

```
TR20 XEQ  1/22/**              New York Manual Wet ED Example 1/01          EWB                                      JOB  1   SUMMARY
     REV 09/01/83              Ultimate Buildout Conditions for 100-yr                                                     PAGE   4


SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
                  (A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
                   A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/    STANDARD              RAIN  ANTEC  MAIN        PRECIPITATION                              PEAK DISCHARGE
STRUCTURE    CONTROL   DRAINAGE  TABLE  MOIST  TIME   -------------------------  RUNOFF    --------------------------------------
   ID       OPERATION    AREA      #    COND  INCREM  BEGIN    AMOUNT  DURATION  AMOUNT    ELEVATION   TIME        RATE      RATE
                       (SQ MI)                 (HR)   (HR)      (IN)     (HR)     (IN)       (FT)      (HR)        (CFS)     (CSM)


   ALTERNATE    1   STORM   99
   ___________________________

STRUCTURE  1  RUNOFF       .10      2      2     .10     .0      5.50    24.00     3.53       ---      12.10       230.71    2261.9
STRUCTURE  1  RESVOR       .10      2      2     .10     .0      5.50    24.00     3.19     634.00     12.22       191.83    1880.7
```

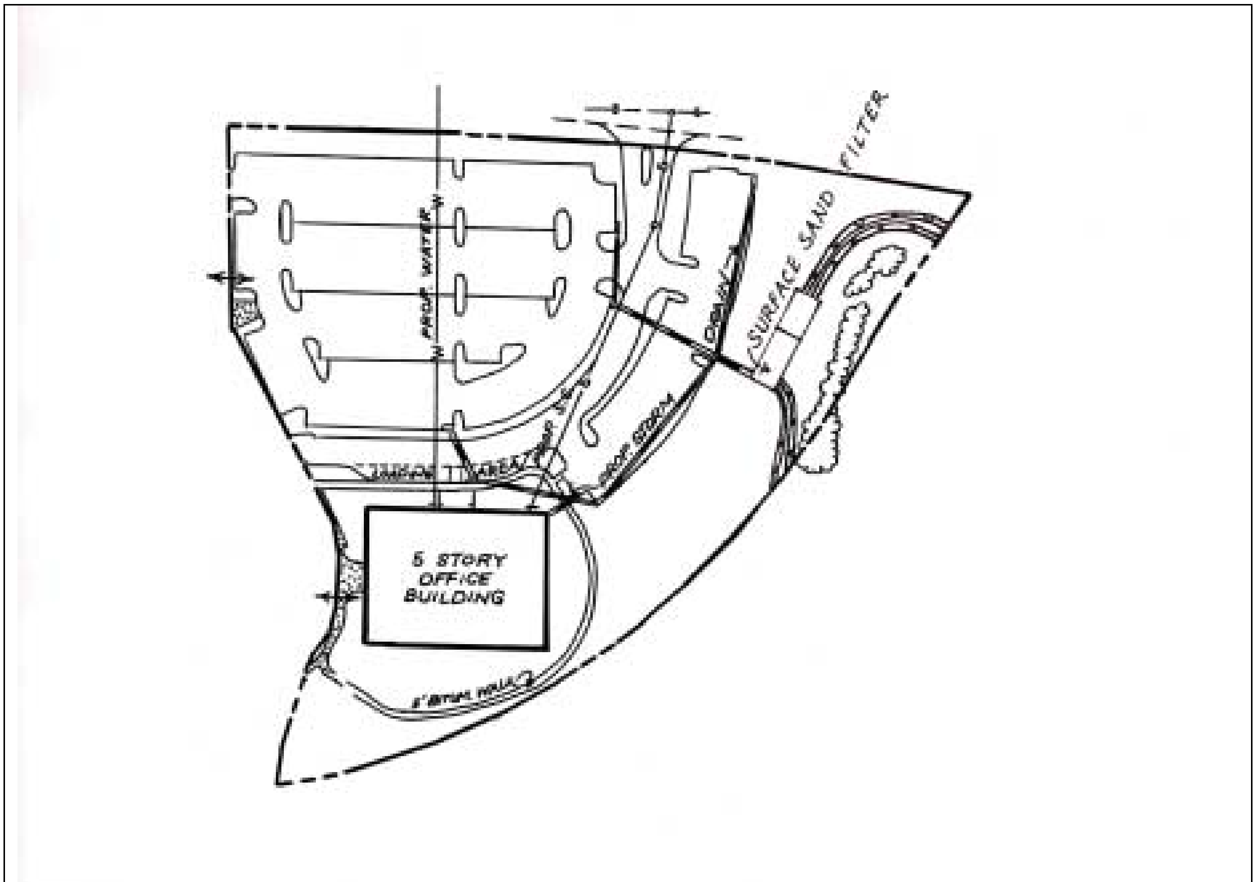
Figure 8.13 Profile of Principle Spillway



Section 8.3 Sand Filter Design Example

This design example focuses on the design of a sand filter for a 4.5-acre catchment of Lake Center, a hypothetical commercial site located in Albany, NY. A five-story office building and associated parking are proposed within the catchment. The layout is shown in Figure 8.14. The catchment has 3.05 acres of impervious cover, resulting in 68% impervious cover. The pre-developed site is a mixture of forest and meadow. On-site soils are predominantly HSG "B" soils.

Figure 8.14 Lake Center Site Plan



Base Data
Location: Albany, NY
Site Area = Total Drainage Area (A) = 4.50 ac
Impervious Area = 3.05 ac; or I =3.05/4.50 = 68%
Soils Type "B"

Hydrologic Data

	Pre	**Post**
CN	58	83
t_c (hr)	.44	.10

This step-by-step example will focus on meeting the water quality requirements. Channel protection control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. For an example of detailed sizing calculations, consult section 8.1. In general, the primary function of sand filters is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 10-yr peak discharge is provided to appropriately size the necessary by-pass flow splitter. Where quantity control is required, bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

Step 1.	**Compute design volumes using the Unified Stormwater Sizing Criteria.**

Water Quality Volume, WQ_v

Select the Design Storm

Consulting Figure 4.1 of this document, use 1.0" as the 90% rainfall event for Albany.

Compute Runoff Coefficient, R_v

$R_v = 0.05 + (68)(0.009) = 0.66$

Compute WQ_v

$$WQ_v = (1.0")(R_v)(A) / 12$$

$$= (1.0")(0.66)(4.5\text{ ac})(43{,}560\text{ft}^2/\text{ac})(1\text{ft}/12\text{in})$$

$$= \underline{10{,}781}\text{ ft}^3 = \underline{0.25\text{ ac-ft}}$$

Develop Site Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations (see Table 8.3)

Note: For this design example, the 10-year peak discharge is given and will be used to size the bypass flow splitter. Any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations.

Table 8.3 Site Hydrology					
Condition	**CN**	**Q_1**	**Q_2**	**Q_{10}**	**Q_{100}**
		cfs	*cfs*	*cfs*	*cfs*
Pre-developed	58	0.2	0.4	3	9
Post-Developed	83	7	10	19	36

Step 2. Determine if the development site and conditions are appropriate for the use of a surface sand filter.

Site Specific Data:

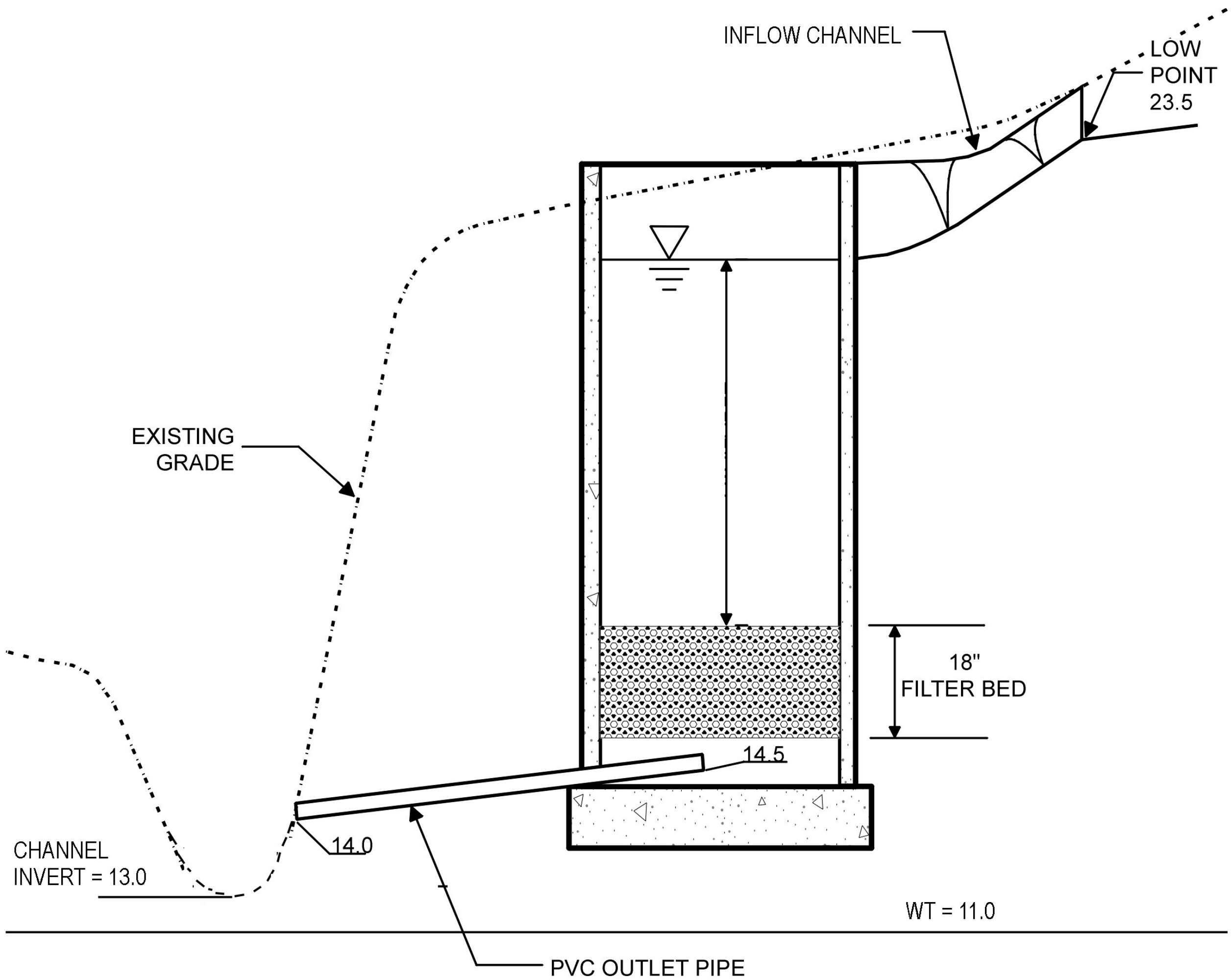
Existing ground elevation at practice location is 222.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 211.0 feet. Adjacent drainage channel invert is at 213.0 feet.

Step 3. Compute available head, & peak discharge (Q_{wq}).

- Determine available head (See Figure 8.15)

The low point at the parking lot is 223.5. Subtract 2' to pass the Q_{10} discharge (221.5) and a half foot for the inflow channel to the facility (221.0). The low point at the channel invert is 213.0. Set the outfall underdrain pipe 1.0' above the drainage channel invert and add 0.5' to this value for the drain slope (214.5). Add to this value 8" for the gravel blanket over the underdrains, and 18" for the sand bed (216.67). The total available head is 221.0 - 216.67 or 4.33 feet. Therefore, the available average depth (h_f) = 4.33' / 2 = 2.17'.

Figure 8.15 Available Head Diagram



- Compute Peak Water Quality Discharge:

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. The Small Storm Hydrology Method presented in Appendix B was followed to calculate a modified curve number and subsequent peak discharge associated with the 1.0-inch rainfall. Calculation steps are provided below.

Compute modified CN for 1.0" rainfall

P = 1.0"

$Q_a = WQ_v \div \text{area} = (10{,}781\ ft^3 \div 4.5\ ac \div 43{,}560\ ft^2/ac \times 12\ in/ft) = 0.66"$

CN $= 1000/[10+5P+10Q_a-10(Q_a^2+1.25*Q_a*P)^{½}]$

$= 1000/[10+5*1.0+10*0.66-10(0.66^2+1.25*0.66*1.0)^{1/2}]$

$= 96.4$

Use CN = 96

For CN = 96 and the t_c = 0.1 hours, compute the Q_{wq} for a 1.0" storm. With the CN = 96, a 1.0" storm will produce 0.6" of runoff. From TR-55 Chapter 2, Hydrology, I_a = 0.083, therefore:

$I_a/P = 0.083/1.0 = 0.083$.

From TR-55 Chapter 4 q_u = 1000 csm/in, and

Q_{wq} = (1000 csm/in) (4.5 ac/640ac/sq mi.) (0.66") = 4.6 cfs.

Step 4. Size the flow diversion structure.

Assume that flows are diverted to a diversion structure (Figure 8.16). First, size a low-flow orifice to pass the water quality storm (Q_p = 4.6 cfs).

$Q = CA(2gh)^{1/2}$; 4.6 cfs = (0.6) (A) [(2) (32.2 ft/s^2) (1.5')]$^{1/2}$

A = 0.77 sq ft = $\pi d^2/4$: d = 0.99' or 12"

Size the 10-year overflow as follows:
The 10-year wsel is initially set at 223.0. Use a concrete weir to pass the 10-year flow (19.0 cfs), minus the flow carried by the low flow orifice, into a grassed overflow channel using the Weir equation. Assume 2' of head to pass this event. Overflow channel should be designed to provide sufficient energy dissipation (e.g., riprap, plunge pool, etc.) so that there will be non-erosive velocities.

Determine the flow from the low-flow orifice (Q_{lf}). Assume 3.5' of head (1.5' plus 2' for the 10-year head):

Q_{lf}=(0.6) (A) [(2) (32.2 ft/s^2) (3.5')]$^{1/2}$

$A = \pi (1')^2/4$

$= 0.78$ sf

So,

Q_{lf} = (0.6) (0.78) [(2) (32.2 ft/s^2) (3.5')]$^{1/2}$

= 7.0 cfs

Thus, determine the flow passed to the through the channel as:

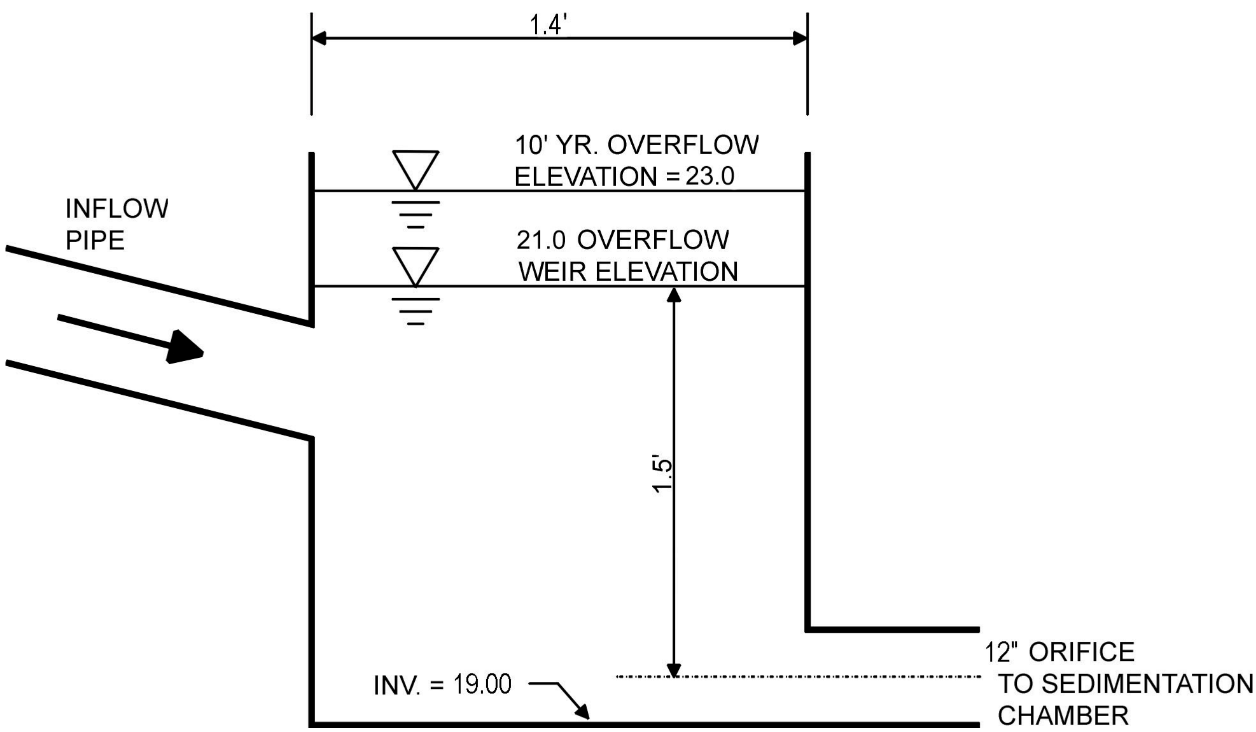
$Q = CLH^{3/2}$

(19-7) = 3.1 (L) (2')$^{1.5}$

L = 1.4' which sets the minimum length of the flow diversion overflow weir.

Weir wall elev. = 21.0. Set low flow invert at 21.0 - [1.5' + (0.5*12"*1ft/12")] = 19.00.

Figure 8.16 Flow Diversion Structure



Step 5. Size filtration bed chamber (see Figure 8.17).

From Darcy's Law: $A_f = WQ_v\ (d_f) / [k\ (h_f + d_f)\ (t_f)]$

where d_f = 18" or 1.5' (Filter thickness)

k = 3.5 ft/day (Flow-through rate)

h_f = 2.17' (Average head on filter)

t_f = 40 hours (Drain time)

A_f = (10,781 cubic feet) (1.5') / [3.5 (2.17' + 1.5') (40hr/24hr/day)]

A_f = 755 sq ft; filter is 20' by 40' (= 800 sq ft)

Step 6. Size sedimentation chamber.

Size the sedimentation chamber as wet storage with a 2.5' depth. Determine the pretreatment volume as:

P_v = (0.25) (10,781 cf)

= 2,695 cf

Therefore,

A_s = (2,695 cf)/(2.5')

= 1,078 sf (Use 20'X55' or 1,100 sf)

Step 7. Compute V_{min}.

V_{min} = ¾(WQ_v) or 0.75 (10,781 cubic feet) = 8,086 cubic feet

Step 8. Compute volume within practice.

Volume within filter bed (V_f): $V_f = A_f (d_f) (n)$; n = 0.4 for sand

V_f = (800 sq ft) (1.5') (0.4) = 480 cf

temporary storage above filter bed ($V_{f\text{-temp}}$): $V_{f\text{-temp}} = 2h_fA_f$

$V_{f\text{-temp}}$ = 2 (2.17') (800 sq ft) = 3,472 cf

Compute storage in the sedimentation chamber (V_s):

V_s = (2.5')(1,100 sf)+4.33'(1,100 sf) = 7,513 cf

$V_f + V_{f\text{-temp}} + V_s$ = 480 cf + 3,472 cf + 7,513 cf = 11,465 cf

11,465 > 8,086 OK.

Pass flow through to the distribution chamber using a 12" orifice with an inverted elbow (see Figure 8.17).

Step 9. Compute sedimentation chamber and filter bed overflow weir sizes.

Assume overflow that needs to be handled is equivalent to the 12" orifice discharge under a head of 3.5 ft (i.e., the head in the diversion chamber associated with the 10-year peak discharge).

$Q = CA(2gh)^{1/2}$

$Q = 0.6(0.79\ ft^2)[(2)(32.2\ ft/s^2)(3.5\ ft)]^{1/2}$

Q = 7.1cfs

Size the overflow weir from the sediment chamber and the filtration chamber to pass 7.1 cfs (this assumes no attenuation within the practice).

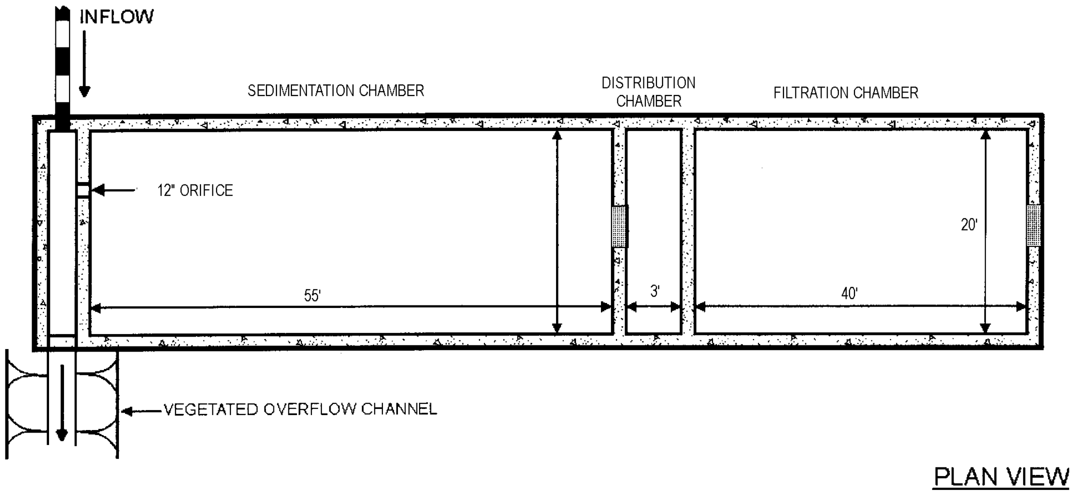
Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'

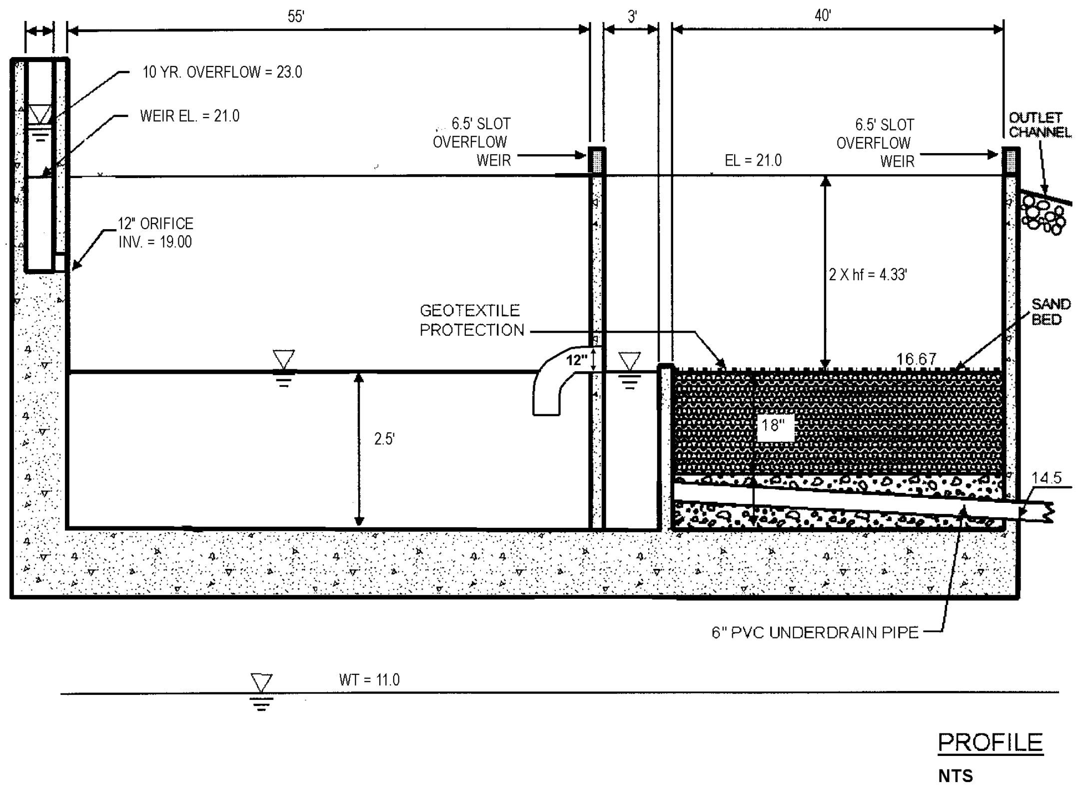
$7.1 = 3.1 * L * (0.5\ ft)^{3/2}$

<u>L = 6.5 ft.</u>

Adequate outlet protection and energy dissipation (e.g., riprap, plunge pool, etc.) should be provided for the downstream overflow channel.

Figure 8.17 Plan and Profile of Surface Sand Filter

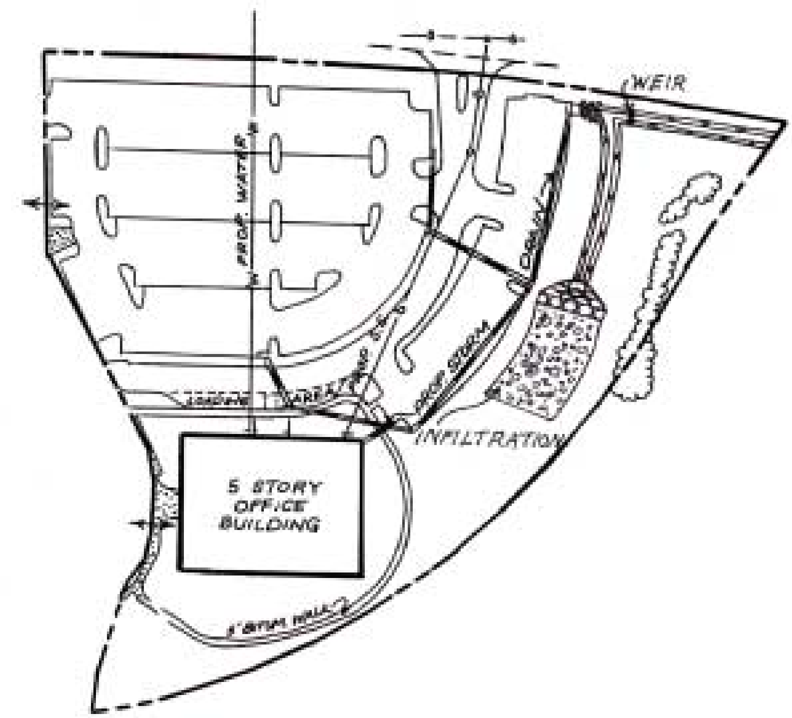




Section 8.4 Infiltration Trench Design Example

This design example focuses on the design of an infiltration trench for a 4.5-acre catchment of the Lake Center, a hypothetical commercial site located in Albany, NY. A five-story office building and associated parking are proposed within this catchment. The layout is shown in Figure 8.18. The catchment has 3.05 acres of impervious cover, resulting in a site impervious cover of 68%. The pre-developed site is a mixture of forest and meadow. On-site soils are predominantly HSG "B" soils.

Figure 8.18 Lake Center Site Plan



Base Data	Hydrologic Data		
Location: Albany, NY		Pre	Post
Site Area = Total Drainage Area (A) = 4.5 ac	CN	58	83
Impervious Area = 3.05 ac; or I =3.05/4.50 = 68%	t_c (hrs)	.44	.10
Soils Type "B"			

This step-by-step example will focus on meeting the water quality requirements. Channel protection control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. For an example of detailed sizing calculations, consult section 8.1. In general, the primary function of infiltration practices is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 10-yr peak discharge is provided to appropriately size the necessary by-pass flow splitter. Where quantity control is required, bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

Step 1. Compute design volumes and flows using the Unified Stormwater Sizing Criteria.

Design values are presented in Table 8.4 below.

Table 8.4 Site Design Hydrology

Condition	CN	WQ_v	Q_1	Q_2	Q_{10}
		ft^3	*cfs*	*cfs*	*cfs*
Pre-Developed	58		0.2	0.4	3
Post-Developed	83	10,781	7	10	19

Step 2. Determine if the development site and conditions are appropriate for the use of an infiltration trench.

Site Specific Data:

Table 8.5 presents site-specific data, such as soil type, percolation rate, and slope, for consideration in the design of the infiltration trench. See Appendix D for infiltration testing requirements and Appendix C for infiltration practice construction specifications.

Table 8.5 Site Specific Data	
Criteria	**Value**
Soil	Silt Loam
Percolation Rate	0.5"/hour
Ground Elevation at BMP	219'
Seasonally High Water Table	211'
Local Ground Slope	<1%

Step 3. Confirm local design criteria and applicability.

Table 8.6, below, summarizes the requirements that need to be met to successfully implement infiltration practices. On this site, infiltration is feasible, with restrictions on the depth and width of the trench.

Table 8.6 Infiltration Feasibility	
Criteria	**Status**
Infiltration rate (f_c) greater than or equal to 0.5 inches/hour.	• Infiltration rate is 0.5 inches/hour. OK.
Soils have a clay content of less than 20% and a silt/clay content of less than 40%.	• Silt Loam meets both criteria.
Infiltration cannot be located on slopes greater than 6% or in fill soils.	• Slope is <1%; not fill soils. OK.
Hotspot runoff should not be infiltrated.	• Not a hotspot land use. OK.
The bottom of the infiltration facility must be separated by at least two feet vertically from the seasonally high water table.	• Elevation of seasonally high water table: 11' • Elevation of BMP location: 19'. • The difference is 8'. • Thus, the trench can be up to 5' deep. OK.
Infiltration facilities must be located 100 feet horizontally from any water supply well.	• No water supply wells nearby. OK.
Maximum contributing area generally less than 5 acres.	• Area draining to facility is approximately 4.5 acres.
Setback 25 feet down-gradient from structures.	• Trench edge is > 25' from all structures. OK.

Step 4. Size overflow channel.

Water flows from the edge of the parking lot to a 4' wide, flat bottom channel with 3:1 side slopes and a 2% slope. This channel also provides pretreatment (See Step 6). Use a weir to divert the water quality volume to the infiltration trench, while allowing the 10-year event to an adjacent drainage channel and the water quality storm to flow to the infiltration trench. The peak flow for the water quality storm is 4.6 cfs (see Section 8.3 for an example calculation).

Determine the depth of flow for the water quality storm using Manning's equation. (Several software packages can be used). The following assumptions are made:

Trapezoidal channel with 3:1 side slopes

4' bottom width.

$S = 1\%$

n varies between 0.03 at 1' depth to 0.15 at 4" depth (See Appendix L and Grass Channel Fact Sheet in Chapter 5).

Determine that the water quality storm passes at d = 0.64'.

Size a weir to pass the 10-year peak event, less the water quality peak flow, so that:

$Q = 19\text{cfs} – 4.6 \text{ cfs} = 14.4 \text{ cfs}.$

Use a weir length, L, of 4.0'.

By rearranging the weir equation:

$H= (Q/CL)^{2/3} = (14.4/3.1(4))^{2/3} = 1.1'$

Size the channel to pass the 10-year event with 6" of freeboard.

Step 5. Size the infiltration trench.

The area of the trench can be determined by the following equation:

$A = WQv/(nd)$

Where:

A	=	Surface Area
WQ_v	=	Water Quality volume (ft^3)
n	=	Porosity
d	=	Trench depth (feet)

Assume that:

n = 0.4

d = 4 feet

Therefore:

$A = 10{,}781\ ft^3 / (0.4 \times 4)ft$

$A = 6{,}738 ft^2$

The proposed location for the infiltration trench will accommodate a trench width of up to 65 feet. Therefore, the minimum length required would be:

$L = 6{,}738\ ft^2 / 65\ ft$

L = 104 feet, say 105 feet

Step 6. Size pretreatment.

Pass the 10-year flow event through an overflow channel.

Size pretreatment to treat ¼ of the WQ_v. Therefore, treat $10{,}781 \times 0.25 = 2{,}695\ ft^3$.

For pretreatment, use a pea gravel filter layer with filter fabric, a plunge pool, and a grass channel.

Pea Gravel Filter

The pea gravel filter layer covers the entire trench with 2" (see Figure 8.19). Assuming a porosity of 0.32, the pretreatment volume (Pv) provided in the pea gravel filter layer is:

Pv_{filter} = (0.32)(2")(1 ft/12 inches)(125')(50') = 333 ft^3

Plunge Pools

Use a 65 'X20' triangular plunge pool with a two foot depth as flow is diverted to the infiltration trench.

$Pv_{pool} = (65 \text{ x } 20 \text{ ft})/2*(2 \text{ ft}) = 1{,}300 \text{ ft}^3$

Grass Channel

Accounting for the pretreatment volumes provided by the pea gravel filter and plunge pool, the grass channel then needs to treat at least $(2{,}695 - 333 – 1{,}300)\text{ft}^3 = 1{,}062 \text{ ft}^3$

Currently stormwater flows through a 150’ long channel, with parameters described under step 4. For this channel, the flow velocity of the peak flow from the water quality storm (4.6 cfs) is approximately 1.2 fps.

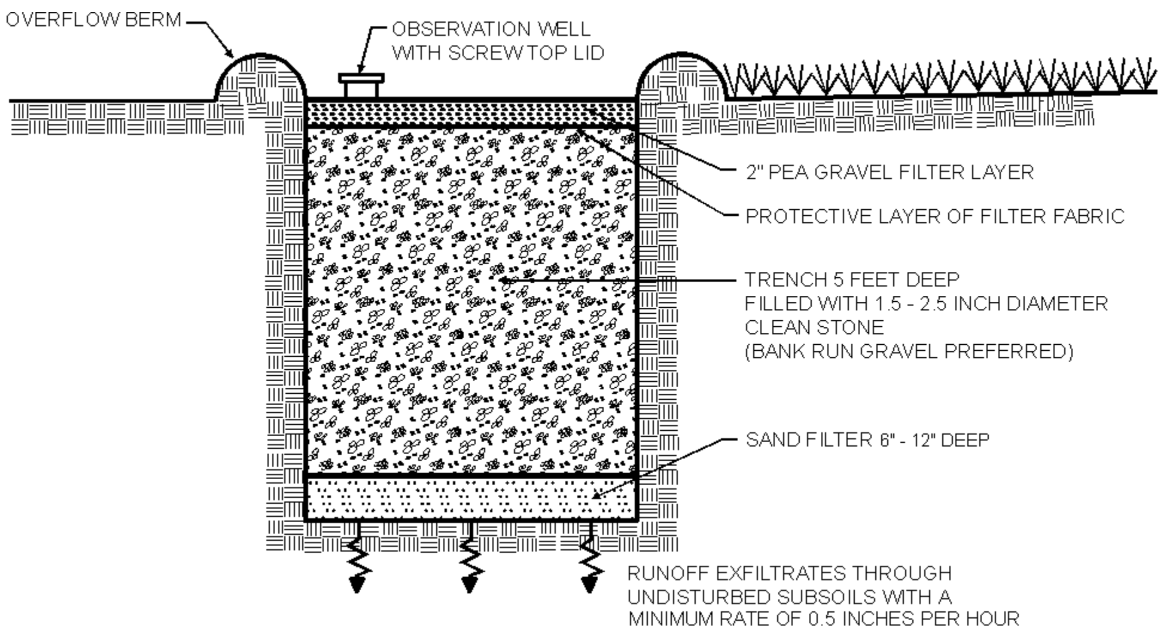
Using a required residence time of 10 minutes (600 seconds), the required length of channel for 100% of the WQ_v (10,781 ft^3) would be 1.2 fps x 600 sec = 720ft.

Adjust the length to account for the volume that must be provided, or:

$(720\text{ft}) (1{,}062 \text{ ft}^3)/(10{,}781 \text{ ft}^3) = 71 \text{ ft}$

Therefore, for this example, a grass channel length of at least 71 feet is required. 150’ is OK.

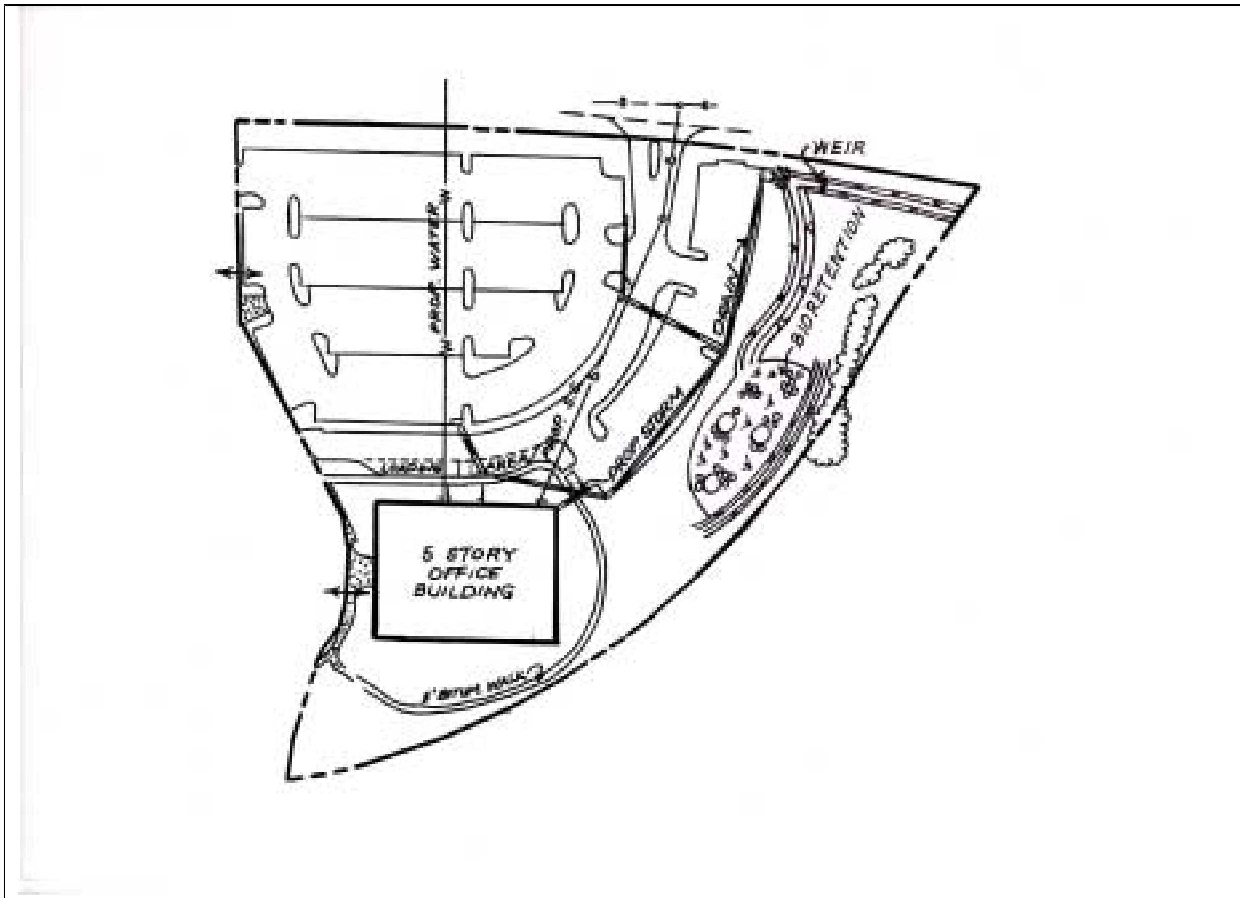
Figure 8.19 Schematic Infiltration Trench Cross Section



Section 8.5 Bioretention Design Example

This design example focuses on the design of a Bioretention area for a 4.5-acre catchment of Lake Center, a hypothetical commercial site located in Albany, NY. A five-story office building and associated parking are proposed within this catchment. The layout is shown in Figure 8.20. The catchment has 3.05 acres of impervious cover, resulting in 68% impervious cover. The pre-developed site is a mixture of forest and meadow. On-site soils are predominantly HSG "B" soils.

Figure 8.20 Lake Center Site Plan



Base Data
Location: Albany, NY
Site Area = Total Drainage Area (A) = 4.5 ac
Impervious Area = 3.05 ac; or I =3.05/4.50 = 68%
Soils Type "B"

Hydrologic Data

	Pre	**Post**
CN	58	83
t_c	.44	.10

This step-by-step example will focus on meeting the water quality requirements. Channel protection control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. For an example of detailed sizing calculations, consult section 8.1. In general, the primary function of bioretention is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 2-year and 10-year peaks are used to appropriately size the grass channel leading to the facility.

Step 1. Compute design volumes using the Unified Stormwater Sizing Criteria.

Design volumes are presented in Table 8.7 below.

Table 8.7 Design Hydrology

Condition	CN	WQ_v	Q_1	Q_2	Q_{10}
		ft^3	*cfs*	*cfs*	*cfs*
Pre-developed	58		0.3	0.6	4
Post-Developed	83	10,781	9	13	26

Step 2. Determine if the development site and conditions are appropriate for the use of a bioretention area.

Site Specific Data:

Existing ground elevation at practice location is 222.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 211.0 feet and underlying soil is silt loam (ML). Adjacent channel invert is at 213 feet.

Step 3. Determine size of bioretention filter area.

$A_f = (WQ_v)\ (d_f)\ /\ [\ (k)\ (h_f + d_f)\ (t_f)]$

Where:

A_f = surface area of filter bed (ft^2)

d_f = filter bed depth (ft)

k = coefficient of permeability of filter media (ft/day)

h_f = average height of water above filter bed (ft)

t_f = design filter bed drain time (days) (2 days is recommended)

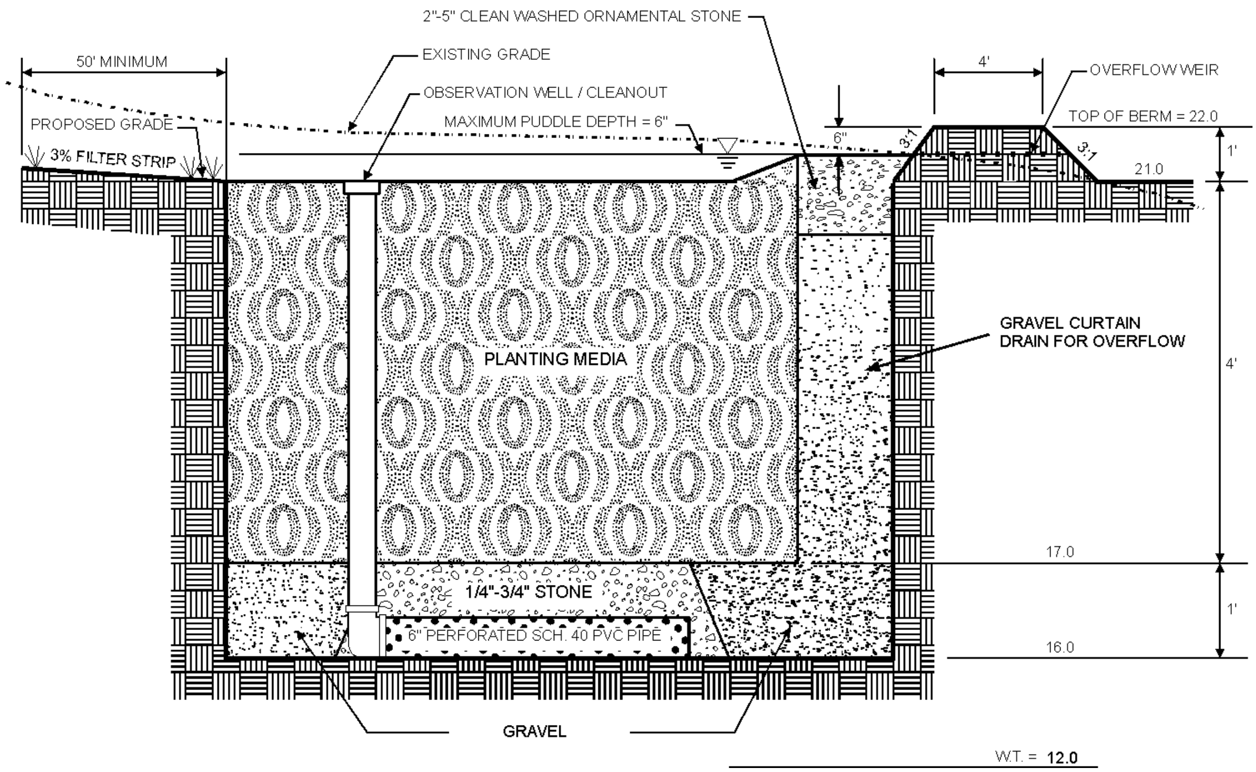
A_f = (10,781 ft^3)(5’) / [(0.5’/day) (0.25’ + 5’) (2 days)] (With k = 0.5'/day, h_f = 0.25’, t_f = 2 days)

A_f = 10,267 sq ft

Step 4. Set design elevations and dimensions.

Assume a roughly 2 to 1 rectangular shape. Given a filter area requirement of 10,267 sq ft, say facility is roughly 70’ by 150'. Set top of facility at 219.0 feet, with the berm at 220.0 feet. The facility is 5' deep, which will allow 3' of separation distance over the seasonally high water table. See Figure 8.21 for a typical section of the facility.

Figure 8.21 Typical Section of Bioretention Facility



Step 5. Size overflow channel.

Assuming the same channel configuration as in Section 8.3, use a 4’ weir set 0.63’ above the base of the overflow channel. The overflow channel will flow to the adjacent drainage channel, while the water quality storm will be diverted to the bioretention cell.

Step 6. Design Pretreatment

Size pretreatment to treat ¼ of the WQ_v. Therefore, treat $10,781 \times 0.25 = 2,695$ ft^3.

For pretreatment, a grass channel is used. This channel has a 4’ width and 3:1 side slopes.

Using the methodologies described in Section 6.3, determine that the length of channel required to treat the entire water quality volume is 720 ft. Adjust the length to correspond to the pretreatment volume, or L = (720 ft)(2,695/10,781) = 180ft.

Step 7. Size underdrain area.

As a rule of thumb, the length of underdrain should be based on 10% of the A_f or 1,027 sq ft and a three-foot wide zone of influence. Using 8" perforated plastic pipes surrounded by a three-foot wide gravel bed, 10' on center (o.c.), yields the following length of pipe:

(1,027 sq ft)/3' per foot of underdrain = <u>342' of perforated underdrain</u>

Step 8. Create overdrain design.

To ensure against the planting media clogging, design a small ornamental stone window of 2" to 5" stone connected directly to the gravel curtain drain. This area is based on 5% of the A_f or 514 sq ft. Say 15' by 35' (see Figure 8.23).

Step 9. Choose plants for planting area.

Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost, aesthetics, maintenance, etc. Select species locations (i.e., on center planting distances) so species will not "shade out" one another. Do not plant trees and shrubs with extensive root systems (e.g., willows) near pipe work. A potential plant list for this site is presented in Appendix H.