Stormwater Manual

The following information is from www.stormwatercenter.net Manual Builder.

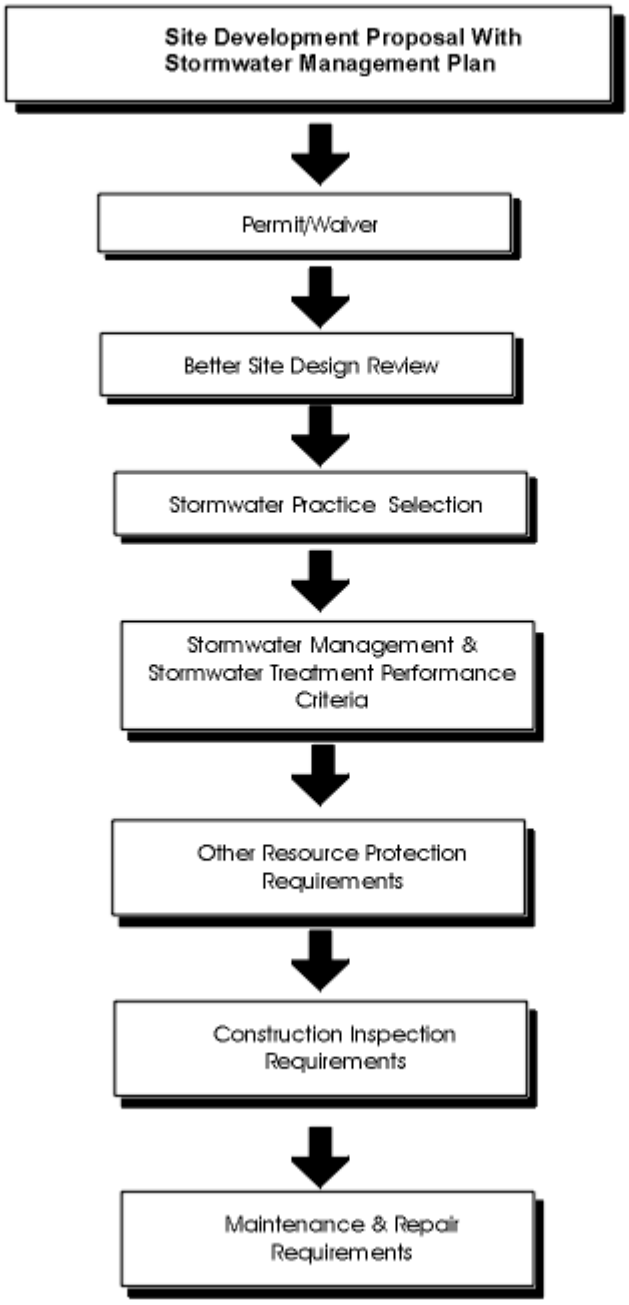
Basic Stormwater Requirements
The manual will typically detail the legal requirements as set forth in the Stormwater Ordinance.

- **Procedures for Reviewing Stormwater Plans**
  In this section, the manual outlines the process a community should go through to review the stormwater plan.
- **Basic Sizing Criteria**
  The manual needs to identify minimum sizing criteria for practices to meet groundwater recharge, water quality, channel protection, and flood control requirements.
- **List of Acceptable Practices**
  The manual should include a list of practices that can meet water quality requirements. An engineer can choose from this menu of practices to treat stormwater runoff from a new development.
- **Performance Criteria**
  The performance criteria provide required minimum elements and guidance to ensure that practices are designed and maintained to ensure practice longevity and performance.
- **Guidance on STP Selection**
  This section presents criteria to guide the design engineer to select the best practice for the site, based on characteristics such as soil type, site slope, and the local watershed conditions.
- **Stormwater Credits**
  Stormwater credits are reductions in stormwater volume requirements given in exchange for incorporating site design techniques that minimize the need for STPs on the site. Many manuals do not incorporate credits, because they can increase the burden of review on local governments substantially.
- **Design Examples**
  Design examples step the engineer through designs for a representative group of STP design variations. They should illustrate how to select, size, and locate the practice on the site.
- **Construction Specifications**
  Construction specifications detail specific materials and construction standards that ensure that the practice will function as designed.
- **Checklists for Construction Inspection**
  These checklists outline what minimum elements are needed for each practice group during construction.

A Model for the Local Stormwater Management Plan Review Process

At any new development site, the design engineer will need to prepare both a "concept" plan, and a final stormwater management plan. The *concept plan* identifies basic site information, locations of proposed development features, and preliminary locations and sizing of stormwater treatment practices. The *final plan* provides more detailed design information for stormwater practices, and includes much greater detail in terms of hydrologic conditions and site features. The concept plan is also typically reviewed in conjunction with a preliminary plan or site plan. The accompanying figure summarizes important stormwater management plan review elements. The process can be explained in seven steps.

Model for a Local Stormwater Management Concept and Plan Review Process



Step 1. What permits are required for this site, and what waivers are applicable?

In this step, the design engineer applies for appropriate permits, and requests waivers where available. A more detailed description of this process is described in the Post-Construction Stormwater Ordinance.

Step 2. Does the plan incorporate Better Site Design techniques to meet stormwater objectives?

In this step, a reviewer evaluates the site plan to see if the designer used available opportunities to meet stormwater objectives using innovative site design techniques. Click here for a Better Site Design Checklist that an applicant can use to demonstrate how these techniques were incorporated into the site plan. In some communities, these techniques are directly integrated into the stormwater planning process through the use of Stormwater Credits.

Step 3. Are the selected practices appropriate for this site?

In both the concept plan and final plan stages, the reviewer needs to ensure that the practices selected for the site are appropriate. The Stormwater Selection Criteria section of the Manual Builder provides guidance on how to locate practices on a site.

Step 4. Are the practices designed to meet minimum performance criteria?

In this step, the reviewer also needs to evaluate if the site plan as presented captures the necessary volume to meet the groundwater recharge, water quality, stream channel protection, and flood control goals outlined in the manual. The Sizing Criteria section outlines options for sizing stormwater treatment practices. The reviewer also evaluates design information in the plan to determine if the practices meet minimum design standards. The Performance Criteria section details specific requirements for each STP option.

Step 5. Does the Plan meet other resource protection requirements?

New developments are often subject to local, state, or federal regulations in addition to stormwater management, and the stormwater review process needs to be integrated with review for these other resource protection measures. Click here for example ordinances for:

- Open Space
- Forest Protection
- Wetlands and Waterways
- Erosion and Sediment Control
- Stream Buffers

Step 6. Do the As-Built Designs satisfy construction specifications?

During and after construction, inspectors determine if the plan is implemented adequately. The Manual Builder includes Construction Specifications, which provide detailed information on construction materials and techniques, as well as a Construction Inspection Checklist for each practice type.

Step 7. Are provisions for long-term maintenance adequate?

The stormwater management plan should provide for long-term maintenance of stormwater treatment practices. These maintenance measures may be assumed by the local government, or by the land owner. Click here to obtain Maintenance Inspection Checklists for each STP group.

Stormwater Practice Sizing Options

This section presents options for sizing stormwater treatment practices to prevent flooding, reduce channel erosion, meet pollutant removal goals, and maintain groundwater recharge. The remaining sections describe the four sizing criteria in detail and present guidance on how to properly compute and apply the required storage volumes. It is important to note that often multiple criteria can be met with one treatment practice.

In the remaining sections we describe in detail options to meet these goals. A default sizing criteria is selected for each practice. In each section we also present "Tools of Analysis" for each criterion.

Click below for detailed guidance on:

- Flood Control
- Channel Protection
- Water Quality
- Groundwater Recharge

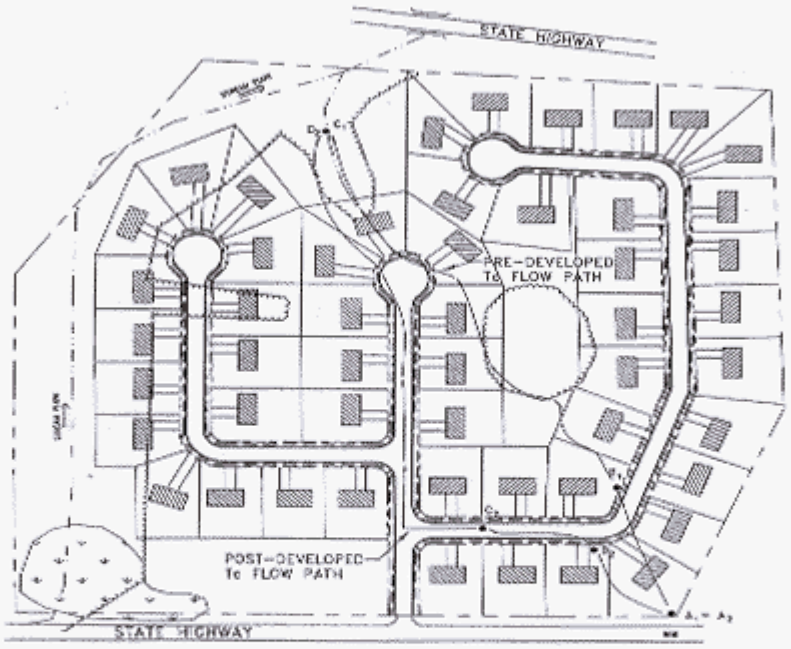
Finally, two design examples illustrate the application of these criteria to both a residential and a commercial site.

Design Example 1: Residential Development - Swann Center

This section presents a sizing example for a medium residential subdivision, Swann Center. The layout of the Swann Center subdivision is shown in Figure 1.

In this design example, the NRCS TR-55 method is used as the design methodology.

Figure 1. Swann Center



Base Data
Location: Anywhere, USA
Site Area = Total Drainage Area (A) = 38.0 ac
Measured Impervious Area = 13.8 ac; I =13.8/38 = 36.3%
Soils Types: 60% "B", 40% "C"
Stream Use Designation - I
Zoning: Residential (½ acre lots)

Hydrologic Data

	Pre	Post
CN	63	78
t_c	.35 hr	.19 hr

Step 1. Compute WQ_v Volume

Criteria:

- Size for the 90% rainfall event.
- Use a minimum runoff coefficient of 0.2.

Step 1a. Compute Runoff Coefficient

This runoff coefficient is derived from Schueler's Simple Method.

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

Where:
I = Impervious Cover (%)
$R_v = 0.05 + (36.3) (0.009) = 0.38$

Step 1b. Compute WQ_v

$WQ_v = (P_1) (R_v) (A)$

Where:
P_1 = 90% Rainfall Event (Inches). Assume 0.9" in this example
A = 38.0 acres

WQ_v = (0.9) (0.38) (38.0 ac) (1ft/12in)
= 1.08 ac-ft

Check Minimum: (0.2) (38.0 ac) (1ft/12) = 0.63 ac-ft [okay]

Step 2. Compute Recharge Volume (Re_v)

Step 2a. Determine Recharge Equation Based on Hydrologic Soil Group (Table 1).

Table 1. Recharge Based on Soil Group

HSG	Recharge Requirement
A	(0.38) (R_v) (A) / 12
B	(0.25) (R_v) (A) / 12
C	(0.13) (R_v) (A) / 12
D	(0.06) (R_v) (A) / 12

Assume imperviousness is located proportionally in B and C soils.

Step 2b. Compute Recharge Volume

For "B" soils =[(0.25 inches) (.38) (.38 ac)/12"/ft] (0.60) = 0.18 ac-ft
For "C" soils =[(0.13 inches) (.38) (.38 ac)/12"/ft] (0.40) = .06 ac-ft
Add recharge requirement for both soils

Re_v = (0.18 ac-ft) + (0.06 ac-ft)
= 0.24 ac-ft

This requirement can be met either with a structural practice, or with Stormwater Credits.

Step 3. Compute Stream Channel Protection Volume (Cp_v):

Requirement: Provide 24 hours of extended detention for the 1-year event.

Please note that the length of detention may be smaller based on the stream resource. In cold water trout streams, for example, the detention time may be as small as 6 to 12 hours.

Step 3a. Develop site hydrologic and TR-55 Input Parameters

Table 2 presents Input Parameters Per attached TR-55 calculations (see Figures 2 and 3).

Table 2. Input Parameters for STP Sizing

Condition	**CN**	**T_c**	**Qa, 1-yr**	**Q 1-yr**	**Q 2-yr**	**Q_{10-yr}**	**Q_{100-yr}**
		hours	*inches*	*cfs*	*cfs*	*cfs*	*cfs*
pre-developed	63	0.35	0.2	4.62	13.58	50.38	102.6
developed	78	0.19	0.8	35.02	54.94	129.96	216.3

Figure 2. Swann Center - Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:				TCS
DRAINAGE AREA NAME:	pre-developed conditions			
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (In acres)
meadow (good cond)		B	58	22.80 Ac.
meadow (good cond)		C	71	9.20 Ac.
woods (good cond)		C	70	6.00 Ac.
			AREA SUBTOTALS:	38.00 Ac.
Time of Concentration 2-Yr 24 Hr Rainfall = 5.1 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.	2.50% 0.31 Hrs
Shallow Flow	UNPAVED		500 Ft. 3.23 F.P.S.	4.00% 0.04 Hrs.
Channel Flow				
Total Area In Acres =	38.00 Ac.	Total Sheet Flow =	Total Shallow Flow =	Total Channel Flow =
Weighted CN =	63	0.31 Hrs.	0.04 Hrs.	0.00 Hrs.
Time Of Concentration =	0.36 Hrs.	RAINFALL TYPE II		
Pond Factor =	1			
STORM	Precipitation (P) Inches	Runoff (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.5 In.	0.2 In.	4.62 CFS	33,662 Cu. Ft.
2 Year	3.1 In.	0.5 In.	13.58 CFS	65,573 Cu. Ft.
5 Year	4.0 In.	0.9 In.	29.14 CFS	126,593 Cu. Ft.
10 Year	5.0 In.	1.5 In.	50.38 CFS	208,134 Cu. Ft.
25 Year	5.4 In.	1.8 In.	59.92 CFS	243,878 Cu. Ft.
50 Year	6.1 In.	2.2 In.	77.79 CFS	309,894 Cu. Ft.
100 Year	7.0 In.	2.9 In.	102.60 CFS	400,142 Cu. Ft.

Figure 3. Swann Center - Developed Conditions
(Source: TR-55 computer printouts using Microsoft Excel spreadsheet developed by Stormworks Software, Durham, NC)

PEAK DISCHARGE SUMMARY				
JOB:				TCS
DRAINAGE AREA NAME:	developed conditions			
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (in acres)
open space		B	61	13.00 Ac.
open space		C	74	5.20 Ac.
woods (good condition)		C	70	6.00 Ac.
impervious area		B	98	7.90 Ac.
impervious area		C	98	5.90 Ac.
			AREA SUBTOTALS:	38.00 Ac.
Time of Concentration	Surface Cover Cross Section	Manning 'n' Wetted Per	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	short grass	'n' = 0.15	100 Ft.	2.50% 0.15 Hrs
Shallow Flow	PAVED		300 Ft. 2.87 F.P.S.	2.00% 0.03 Hrs.
Channel Flow Hydraulic Radius = 1.00	X-S estimated	'n' = 0.013 WP estimated	600 Ft. 16.21 F.P.S.	2.00% 0.01 Hrs.
Total Area in Acres =	38.00 Ac.	Total Sheet Flow =	Total Shallow Flow =	Total Channel Flow =
Weighted CN =	78	0.15 Hrs.	0.03 Hrs.	0.01 Hrs.
Time Of Concentration =	0.19 Hrs.	RAINFALL TYPE II		
Pond Factor =	1			
STORM	Precipitation (P) Inches	Runoff (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.5 In.	0.8 In.	35.02 CFS	108,687 Cu. Ft.
2 Year	3.1 In.	1.2 In.	54.94 CFS	166,607 Cu. Ft.
5 Year	4.0 In.	1.9 In.	88.74 CFS	260,283 Cu. Ft.
10 Year	5.0 In.	2.7 In.	129.96 CFS	374,051 Cu. Ft.
25 Year	5.4 In.	3.1 In.	147.20 CFS	421,327 Cu. Ft.
50 Year	6.1 In.	3.7 In.	177.27 CFS	505,879 Cu. Ft.
100 Year	7.0 In.	4.5 In.	216.30 CFS	617,257 Cu. Ft.

Step 3b. Compute Storage Volume

The following technique is based on the method described in Design Procedures for Stormwater Management Extended Detention Structures, (Maryland Department of the Environment, 1987).

Initial abstraction (Ia) for CN of 78 is 0.564: (TR-55) [Ia = (200/CN - 2)]

Ia/P = (0.564)/ 2.5 inches = 0.226
Tc = 0.19 hours
From TR-55, Exhibit 4-11 (NRCS, 1986):
qu = 740 csm/in
Knowing qu and T (extended detention time), find Qo/Qi from Figure 4, "Detention Time Versus Discharge Ratios."

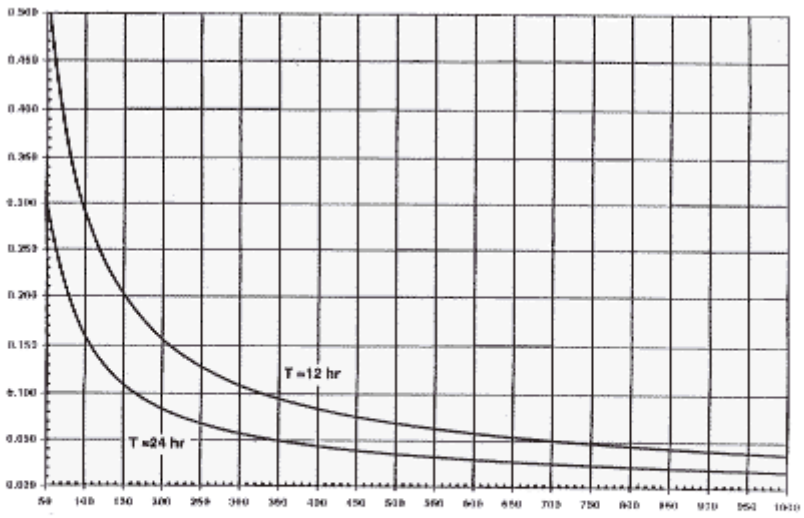


Figure 4. Detention Time vs. Discharge Ratios (q_o/q_i)
(Source: Maryland Stormwater Design Manual,
Maryland Department of the Environment, 2000)

Peak outflow discharge/peak inflow discharge (Qo/Qi) = 0.024

From TR-55, Volume of storage/Volume of runoff (Vs/Vr)

(Figure 6.1, page 6-2 of TR55)

For a Type II rainfall distribution,

$Vs/Vr = 0.683 - 1.43(Qo/Qi)+1.64(Qo/Qi)^2 - 0.804(Qo/Qi)^3$ (MDE, 1987)

Note that a different equation would be used to describe hydrographs of Type I or Type III distributions.

Vs/Vr = 0.65

Therefore, Vs = 0.65(0.8)(1/12)(38 ac) = 1.65 ac-ft

Step 3d. Define the average ED Release Rate

Qi is known (35.0 cfs), therefore,
Qo = (Qo/Qi) Qi = .024 (35.0) = .84 cfs

Step 4. Compute Peak Discharge Control Volume (Q_p):

Control the peak flow from 10-year storm event to pre-developed levels.

Step 4a. Model Site Using TR-55 for 10 year storm

Per TR-55, Figure 6-1 (Page 6-2 of TR-55) for a Q_{in} of 130.0 cfs, and an allowable Q_{out} of 50.4 cfs, the V_s necessary for control is 2.83 ac-ft, with a developed CN of 78. (See TR-55 Worksheet 6a, Page 6-5 of TR-55). Note that 5.0 inches of rain fall during this event, with 2.7 inches of runoff.

Step 5. Analyze Safe Passage of 100 Year Design Storm (Q_f)

Note that this design example doe not assume that control of the 100-year event is required.

At final design, provide safe passage for the 100 year event. Based on field observation, downstream conveyance will require analysis for passing the 100-year event through an existing culvert.

Table 3. Summary of General Storage Requirements for Swann Center

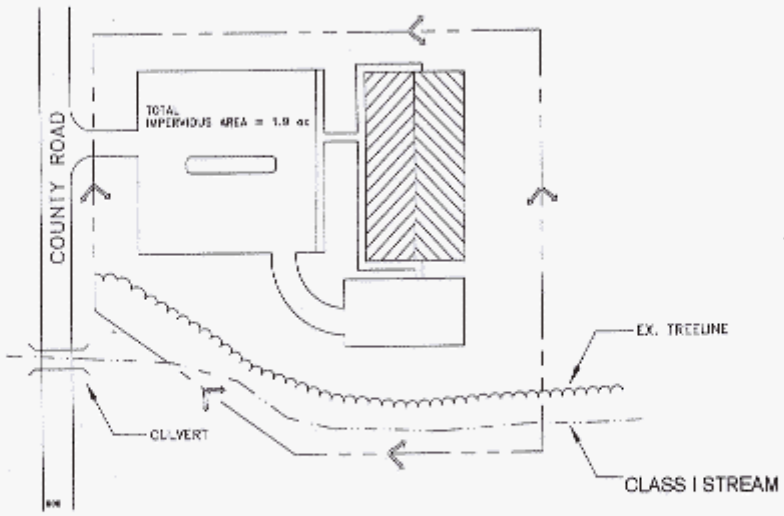
No.	Category	Volume Required (ac- ft)	Notes
WQ_v	Water Quality Volume	1.08	
Re_v	Recharge	.24	this volume is included within the WQ_v storage
Cp_v	Stream Protection	1.65	Average ED release rate is .84 cfs over 24 hours
Q_p	Peak Control	2.83	10-year, in this case
Q_f	Flood Safe Passage		provide safe passage for the 100-year event in final design

Design Example No. 2: Commercial Development - Brown Community Center

This is a sizing example for a commercial site, Brown Community Center. The layout of the Brown Community Center is shown in Figure 1.

In this design example, the NRCS TR-55 method is used as the design methodology.

Figure 1. Brown Community Center



Base Data	Hydrologic Data		
Location: Anywhere, USA			
Site Area = Total Drainage Area (A) = 3.0 ac		Pre	Post
Impervious Area = 1.9 ac; I = 1.9/3.0 =63.3%			
R_v = 0.05 + (63.3) (0.009) = 0.62	CN	57	87
Soils Type "B"	t_c	.42	.16
Stream Use Designation: (I)			

Step 1. Compute Water Quality Volume WQ_v

Criteria:

- Size for the 90% rainfall event.
- Use a minimum runoff coefficient of 0.2.

Step 1a. Compute Runoff Coefficient

This runoff coefficient is derived from Schueler's Simple Method.

$Rv = 0.05 + (I)\ (0.009)$

Where:

I = Impervious Cover (%)
Rv = 0.05+(63.3)(0.009) = 0.62

Step 1b. Compute WQ_v

$WQ_v = (P_1)\ (R_v)\ (A)$

Where:

P_1 = 90% Rainfall Event (Inches). Assume 1" in this example
A = 38.0 acres

WQ_v = (1.0") (R_v) (A) / 12
= (1.0") (0.62) (3.0ac) (43560ft^2/ac) (1ft/12in)
= 6752 ft^3

Check Minimum: (0.2) (3.0 ac) (3630) = 2178 ft^3 [okay]

Step 2. Compute Recharge Volume (Re_v)

Step 2a. Determine Recharge Equation Based on Hydrologic Soil Group (Table 1).

Table 1. Recharge Based on Soil Group

	Recharge Requirement
A	(0.38) (R_v) (A) / 12
B	(0.25) (R_v) (A) / 12
C	(0.13) (R_v) (A) / 12
D	(0.06) (R_v) (A) / 12

Step 2b. Compute Recharge Volume

For "B" soils, (0.25 inches) (.62) (3.0 ac) (1/12"/ft) (43,560 ft^2/ac) = 1,688 cubic feet; this represents the minimum target volume for recharge purposes.

Step 3. Compute Stream Channel Protection Volume (Cp_v):

In this design example, we assume that channel protection is not required at this site due to the small site size, and relatively flat slopes in the watershed.

Step 4. Compute Peak Discharge Control Volume (Q_p):

Table 2 presents Input Parameters Per attached TR-55 calculations (see Figures 2 and 3).

Figure 2. Brown Community Center - Pre-Developed Condition

PEAK DISCHARGE SUMMARY				
JOB:				TCS
DRAINAGE AREA NAME:	pre-developed condition			
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN From TABLE 2-2	AREA (In acres)
meadow (good conditio		B	58	2.40 Ac.
woods (good condition)		B	55	0.60 Ac.
			AREA SUBTOTALS:	3.00 Ac.
Time of Concentration 2-Yr 24 Hr Rainfall = 3.4 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.	1.50% 0.36 Hrs
Shallow Flow	UNPAVED		500 Ft. 2.28 F.P.S.	2.00% 0.06 Hrs.
Channel Flow				
Total Area in Acres =	3.00 Ac.	Total Sheet Flow =	Total Shallow Flow =	Total Channel Flow =
Weighted CN =	57	0.36 Hrs.	0.06 Hrs.	0.00 Hrs.
Time Of Concentration =	0.42 Hrs.	RAINFALL TYPE II		
Pond Factor =	1			
STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.8 In.	0.2 In.	0.22 CFS	2,055 Cu. Ft.
2 Year	3.4 In.	0.4 In.	0.58 CFS	4,128 Cu. Ft.
5 Year	4.5 In.	0.8 In.	1.79 CFS	9,249 Cu. Ft.
10 Year	5.5 In.	1.3 In.	2.91 CFS	13,809 Cu. Ft.
25 Year	6.0 In.	1.7 In.	3.98 CFS	18,252 Cu. Ft.
50 Year	6.8 In.	2.2 In.	5.50 CFS	23,754 Cu. Ft.
100 Year	7.6 In.	2.7 In.	6.75 CFS	29,633 Cu. Ft.

Figure 3. Brown Community Center - Developed Condition

PEAK DISCHARGE SUMMARY				
JOB:				TCS
DRAINAGE AREA NAME:	post-developed condition			
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (in acres)
open space (good cond.		B	61	0.50 Ac.
woods (good condition)		B	55	0.60 Ac.
Impervious		B	98	1.90 Ac.
			AREA SUBTOTALS:	3.00 Ac.
Time of Concentration 2-Yr 24 Hr Rainfall = 3.4 in	Surface Cover Cross Section	Manning 'n' Wetted Per	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	short grass	'n' = 0.15	50 Ft.	1.60% 0.10 Hrs
Shallow Flow	PAVED		600 Ft. 2.87 F.P.S.	2.00% 0.06 Hrs.
Channel Flow Hydraulic Radius = 0.75	X-S estimated	'n' = 0.024 WP estimated	50 Ft. 7.25 F.P.S.	2.00% 0.00 Hrs.
Total Area in Acres =	3.00 Ac.	Total Sheet Flow =	Total Shallow Flow =	Total Channel Flow =
Weighted CN =	83	0.10 Hrs.	0.06 Hrs.	0.00 Hrs.
Time Of Concentration =	0.16 Hrs.	RAINFALL TYPE II		
Pond Factor =	1			
STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.8 in.	1.3 in.	5.08 CFS	14,019 Cu. Ft.
2 Year	3.4 in.	1.8 in.	7.11 CFS	19,327 Cu. Ft.
5 Year	4.5 in.	2.7 in.	11.05 CFS	29,681 Cu. Ft.
10 Year	5.3 in.	3.4 in.	13.97 CFS	37,535 Cu. Ft.
25 Year	6.0 in.	4.1 in.	16.59 CFS	44,555 Cu. Ft.
50 Year	6.8 in.	4.8 in.	19.62 CFS	52,700 Cu. Ft.
100 Year	7.6 in.	5.6 in.	22.69 CFS	60,943 Cu. Ft.

Step 2b. Compute Recharge Volume

For "B" soils, (0.25 inches) (.62) (3.0 ac) (1/12"/ft) (43,560 ft^2/ac) = 1,688 cubic feet; this represents the minimum target volume for recharge purposes.

Step 3. Compute Stream Channel Protection Volume (Cp_v):

In this design example, we assume that channel protection is not required at this site due to the small site size, and relatively flat slopes in the watershed.

Step 4. Compute Peak Discharge Control Volume (Q_p):

Table 2 presents Input Parameters Per attached TR-55 calculations (see Figures 2 and 3).

Table 2. Input Parameters for STP Sizing					
Condition	**RCN**	**$Q_{1\text{-year}}$**	**$Q_{2\text{-year}}$**	**$Q_{10\text{-year}}$**	**$Q_{100\text{-year}}$**
		cfs	*cfs*	*cfs*	*cfs*
pre-developed	57	0.22	0.58	2.91	6.75
developed	83	5.08	7.11	13.97	22.69

Assume that 10-year quantity peak control is required (Q_{p10}).

Per TR-55, Figure 6-1 (Page 6-2 in TR-55), for a Q_{in} of 13.97 cfs, and an allowable Q_{out} of 2.91 cfs, the Vs necessary for 10-year control is 0.24 ac-ft or 16,890 ft^3, under a developed CN of 83.

Step 5. Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100 year event and discharge to receiving waters.

Table 3. Summary of Design Information for Brown Community Center			
No.	**Category**	**Volume Required** (cubic feet)	**Notes**
1	Water Quality (WQ_v)	6,752	
2	Recharge (Re_v)	1,688	This volume can be included within the WQ_v storage
3	Stream Protection (Cp_v)	n/a	Not required on this site.
4	Peak Control (Q_p)	16.890	10-year control
5	Flood Safe Passage (Q_f)		Provide safe passage for the 100-year event in final design

Acceptable Stormwater Treatment Practices

A stormwater manual needs to identify a set of stormwater treatment practices (STPs) that meet water quality goals. The Manual Builder defines a set of five major STP groups, and then identifies specific practices within these groups. The manual should typically be accompanied by a technical support document to justify selection or elimination of a particular practice. The Manual Builder focuses on practices that have demonstrated ability to remove 80% of Total Suspended Solids (TSS).

- *A manual may also identify when alternative practices can be used, and the specific criteria these practices should meet. In addition, a manual may include monitoring requirements needed to add a new practice to the manual.*

- *An alternative approach may be used in jurisdictions that require a specific load reduction at a site. In these areas, the manual may include a broader range of practices, and assign a specific pollutant removal to each design option.*

The following practices are included in the manual builder. The *Performance Criteria* section describes these practices in greater detail.

Group 1: Ponds

Stormwater ponds are practices that have a combination of a permanent pool, extended detention or shallow marsh equivalent to the entire WQ_v. Design variants include:

- Micropool Extended Detention Pond *
- Wet Pond
- Wet Extended Detention Pond
- Multiple Pond System
- "Pocket" Pond

Note that the Dry Extended Detention Pond is not included in this group. Available data suggest poor pollutant removal for this practice.

** Note that very little monitoring data are available to assess the pollutant removal performance of the micropool extended detention pond. This practice is a modification of the Dry Extended Detention pond. The major difference is that the micropool extended detention pond incorporates a small micropool at the outlet, and a wet forebay. The practice is ideal for cold water stream systems, where a practice with a large permanent pool may cause stream warming. It is assumed that this practice performs similarly to wet ponds.*

Group 2: Wetlands

Stormwater wetlands are practices that create shallow marsh areas to treat urban stormwater and often incorporate small permanent pools and/or extended detention storage to achieve the full WQ_v. Design variants include:

- Shallow Wetland
- ED Shallow Wetland
- Pond/Wetland System
- "Pocket" Wetland

The Gravel-Based Wetland, a design variant of the stormwater wetland has not been included in the Manual Builder. While this design has not been tested extensively, available monitoring data suggest good pollutant removal. For more information on this practice, consult Article 96 or Article 97 from the Practice of Watershed Protection.

Group 3: Infiltration Systems

Stormwater infiltration practices capture and temporarily store the WQ_v before allowing it to infiltrate into the soil. Design variants include:

- Infiltration Trench
- Infiltration Basin

Porous Pavement systems are not included in the Manual Builder, because field application has shown a very high failure rate. Porous Pavement may have some application in very low traffic areas, however. Article 103 from the Practice of Watershed Protection provides some information about porous pavement.

Group 4: Filtering Systems

Stormwater filtering system capture and temporarily store the WQ_v and pass it through a filter bed of sand, organic matter, soil or other media. Filtered runoff may be collected and returned to the conveyance system, or allowed to partially exfiltrate into the soil. Design variants include:

- Surface Sand Filter
- Underground Sand Filter
- Perimeter Sand Filter
- Organic Filter
- Bioretention*

** Little data is available to assess the performance of bioretention systems. It is assumed that they perform similarly to other filtering systems, or to the dry swale, which has a similar design.*

Group 5: Open Channel Systems

Open channel systems are vegetated open channels that are explicitly designed to capture and treat the full WQ_v within dry or wet cells formed by checkdams or other means. Design variants include:

- Dry Swale
- Wet Swale
- Grass Channels

Grass channels are included in the list of practices for the Manual Builder, although monitoring data suggest slightly less than 80% sediment removal. They are best applied to residential developments. Alternatively, this practice can be included in a manual as a Stormwater Credit.

STP Groups Not Included in the Manual Builder

Some groups of STPs that are not typically included as stand-alone water quality practices include the following:

- *Detention Practices*
  These practices include both dry detention ponds and underground detention vaults. While these practices reduce flood peaks, and may be able to protect stream channels from erosion, they are unable to improve water quality.

- *Oil/Grit Separators*
  These practices are a series of vaults designed to remove water and sediment from stormwater runoff. Available data suggest very poor pollutant removal. These practices are best applied as pretreatment to another practice, or in retrofit situations. See Article 119 of the Practice of Watershed Protection for a discussion of their pollutant removal capability.

- *Hydrodynamic Devices*
  Hydrodynamic devices include a series of commercially available practices that operate on a similar principle to oil/grit separators. In general, they are designed to accommodate the peak flow from a specific water quality event. Limited independent data are available to assess these practices, and the few independent studies suggest moderate to poor pollutant removal. See Article 120 of The Practice of Watershed Protection for studies on one hydrodynamic device, the Stormceptor®. The best application for these practices is as pretreatment, or as an urban retrofit, where little space is available on site.

- *Pollution Prevention*
  While not an actual treatment practice, pollution prevention practices can act to reduce the pollutant load from urban watersheds. These practices are not included in the Manual Builder, which focuses on treatment practices for new development. For more detail on pollution prevention options, look at the Pollution Prevention Fact Sheets.

- *Site Design*
  Practices such as stream buffers, rooftop disconnection, and reforestation may are not typically considered as stand-alone practices. They can, however, be incorporated into a stormwater manual as *stormwater credits*. In addition, site design elements that reduce impervious cover can reduce the volume required on a site, as most storage requirements are based on site impervious cover.

Performance Criteria for Urban STP Design

This section outlines performance criteria for five groups of structural water quality stormwater STPs that include ponds, wetlands, infiltration practices, filtering systems and open channels. These criteria represent a series of design features that ensure effective, long-lived practice.

Communities need to decide which performance criteria are voluntary, and which are required. As a starting point for this distinction, this section displays ***elements that are probably required in bold and italics***. In addition, required and voluntary elemants are summarzed in a table that preceeds the performance criteria for each practice.

Each set of STP performance criteria is based on six factors:

- General Feasibility
- Conveyance
- Pretreatment
- Treatment/Geometry
- Environmental/Landscaping
- Maintenance

Please use these links to performance criteria for:

- Ponds
- Wetlands
- Infiltration Systems
- Filtering Systems
- Open Channels

Please note that the criteria presented here were largely adapted from standards for the Mid-Atlantic. Notes are provided to allow the user to adapt them for a local region. For more information on cold climate design, please consult the Center's 1997 document, "BMP Design Supplement for Cold Climates."

STP Screening Matrices

This section presents a series of matrices that can be used as a screening process for selecting the best STP or group of STPs for a development site. It also provides guidance for locating practices on the site. The matrices presented can be used to screen practices in a step-wise fashion. Screening factors include:

- Land Use
- Physical Feasibility
- Climate/Regional Factors
- Watershed Factors
- Stormwater Management Capability
- Pollutant Removal
- Community and Environmental Factors

The matrices presented here are not exhaustive. Specific additional criteria may be incorporated depending on local design knowledge and resource protection goals. Furthermore, many communities may wish to eliminate some of the selection factors presented in this section. Caveats for the application of each matrix are included in the detailed description of each.

In the matrices presented below, several specific numerical requirements are included, and may vary between communities.

Most of the material in the screening matrices provides technical guidance, and is not necessarily a regulatory mandate. In the few cases where a screening element would typically be a regulatory requirement, cells are shaded.

More detail on the proposed step-wise screening process is provided below:

Step 1. Land Use

Which practices are best suited for the proposed land use at this site? In this step, the designer makes an initial screen to select practices that are best suited to a particular land use.

Step 2. Physical Feasibility Factors

Are there any physical constraints at the project site that may restrict or preclude the use of a particular STP? In this step, the designer screens the STP list using Matrix No. 2 to determine if the soils, water table, drainage area, slope or head conditions present at a particular development site might limit the use of a STP. In addition, the matrix indicates which STP options work well in highly urban areas.

Step 3. Climate/Regional Factors

Are there any regional characteristics that restrict or modify the use of certain STPs? Matrix No. 3 details potential modifications to STP selection based on climate and geology.

Step 4. Watershed Factors

What watershed protection goals need to be met in the resource my site drains to? Matrix No.4 outlines STP goals and restrictions based on the resource being protected.

Step 5. Stormwater Management Capability

Can one STP meet all design criteria, or is a combination of practices needed? In this step, designers can screen the STP list using Matrix No. 5 to determine if a particular STP can meet recharge, water quality, channel protection, and flood control storage requirements. At the end of this step, the designer can screen the STP options down to a manageable number and determine if a single STP or a group of STPs are needed to meet stormwater sizing criteria at the site.

Step 6. Pollutant Removal

How do each of the STP options compare in terms of pollutant removal? In this step, the designer views removal of select pollutants to determine the best STP options for water quality.

Step 7. Community and Environmental Factors

Do the remaining STPs have any important community or environmental benefits or drawbacks that might influence the selection process? In this step, a matrix is used to compare the twenty STP options with regard to maintenance, habitat, community acceptance, cost and other environmental factors.

Stormwater Credits

The purpose of establishing a stormwater credit system is to provide incentive to developers, designers, builders and municipal officials to implement better site design and locate new development in a manner that causes less impact to aquatic resources. By taking advantage of the credit system, developers and builders can reduce the stormwater management quantity and/or quality requirements. The credit system directly translates into cost savings to the developer by reducing the size of stormwater storage and conveyance systems.

Stormwater credits are tied directly to the *Recharge*, *Water Quality*, *Channel Protection*, and *Flood Control* volume requirements. For example, if there is a water quality volume requirement, then credits can be given for site design practices that reduce the volume of runoff generated by a site (e.g., establishing naturally vegetated conservation areas). Site conditions will also determine which credits can be applied. For example, a stream buffer credit may not be implemented on upland sites with no perennial or intermittent streams.

This section presents nine options for stormwater credits that can be applied within a community. Each community needs to select both stormwater sizing criteria and stormwater credits that best meet its economic, social, and resource protection needs. In addition the specific numerical calculation of credits can vary. For each credit, we present example calculations or applications, and provide guidance on the best application for each credit.

This section presents two broad types of credits: **Site Design Credits,** and **Watershed Credits**. Site design credits act as incentives to encourage *Better Site Design* techniques by reducing required quality or quantity storage volumes on site. Watershed credits are reductions or exemptions from stormwater management requirements to support watershed goals such as redevelopment or watershed zoning.

Please use these links to connect to descriptions of stormwater credits:

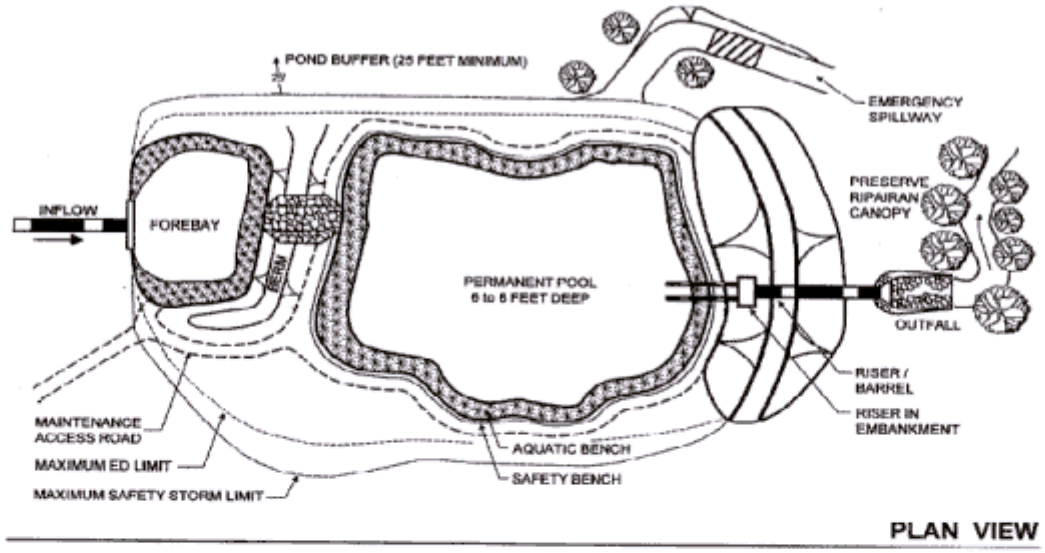
- Site Design Credits
- Watershed Credits

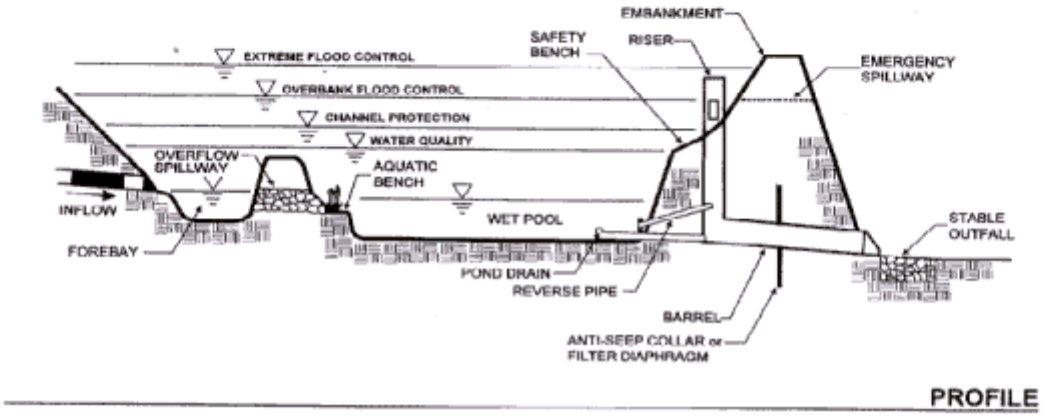
Stormwater Design Examples

This section provides detailed design examples for four practices, representing four different STP groups, annotated to discuss where specific departures may be made from the designs as presented. Please click here to link to design examples for:

- Ponds
- Infiltration
- Filtering Systems
- Open Channels

Stormwater Design Example: Pond





POND DESIGN EXAMPLE

The following design example is for the "wet extended detention (ED) pond," from STP Group 1 (stormwater ponds).

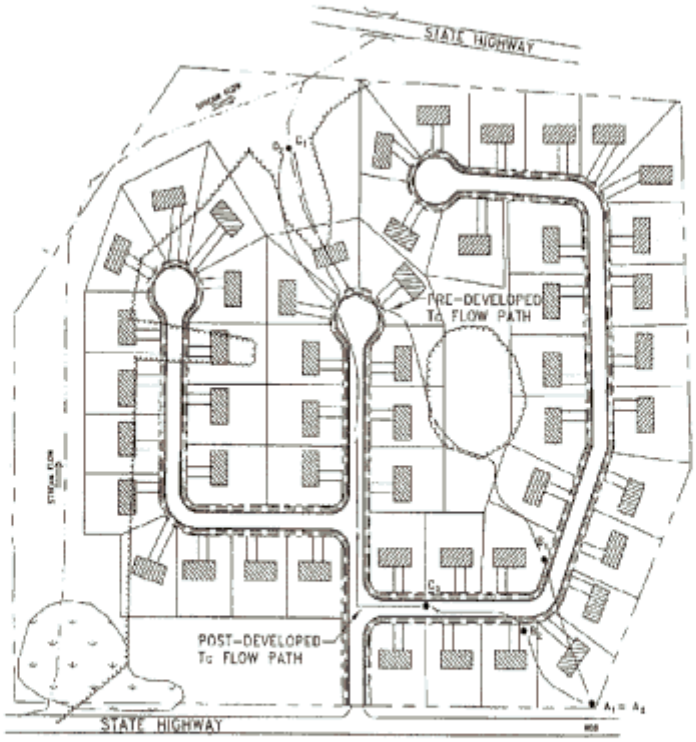
On this site the developer is required to provide control for the following:

1. Recharge (Based on soil type)
2. Water Quality (90% event)
3. Channel Protection

4. 10-year Flood Control
5. Safe Passage of the 100-year flood with 1' of freeboard.

Based on the specific criteria in a region. These criteria may vary. See Sizing Options.

Figure 1. Swan Center Site Plan



Base Data
Location: Anywhere, USA
Site Area = Total Drainage Area (a) = 38.0 ac
Measured Impervious Area=13.8 ac; or I=13.8/38=36.3%
Soils Types: 60% "B", 40% "C"

Site Data:

The site area and drainage area to the pond is 38.0 acres. Existing ground at the pond outlet is 320'. Soil boring observations reveal that the seasonally high water table is at elevation 318'. 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 316'. TR-55 analyses results are in Table 1.

Table 1. Hydrology Summary

Condition	CN	t_c	$Q_{1\text{-year}}$	$Q_{2\text{-year}}$	$Q_{10\text{-year}}$	$Q_{100\text{-year}}$
		hours	cfs	cfs	cfs	cfs
Pre-developed	63	0.35	4.62	13.58	50.38	102.6
Post-developed	78	0.19	35.0	55.0	131.0	216.3

Step 1. Compute Design Volumes

Required volumes for 1-4 were computed under Example 1 of the sizing options section and are presented in Table 2.

Table 2. Summary of General Design Information

Symbol	Category	Volume Required	Standard
WQ_v	Water Quality Volume	1.08 ac-ft	Provide as 24-Hour ED + Permanent Pool
Re_v	Recharge Volume	0.24 ac-ft	Infiltrate
Cp_v	Channel Protection Volume	1.65 ac-ft	Provide 24-Hour ED > 0.84 OFS
$Q_{p\text{-}10}$	Overbank Protection	2.83 ac-ft	Reduce Peak Flow to Pre-developed Levels

Step 2. Investigate potential pond hazard classification

Review downstream conditions and compute a preliminary Breach Peak Discharge (Q_{max}) per state requirements. Breach discharge and actual classification to be confirmed once embankment design has been finalized.

$Q_{max} = (3.2)\ (H_w^{5/2})$ where,

H_w = depth of water at the dam at the time of failure, in feet. (measured from the emergency spillway (or design high water, if no emergency spillway exists) to the lowest point in the original cross section at the centerline of the dam.

Assume H_w = 12 feet: $Q_{max} = (3.2)\ (12)^{5/2}$ = 1596 cfs

Check downstream culvert/road for passage of this flowrate, by separate analysis (FHWA culvert charts and HY-8 computer program) existing culvert and road section will pass 1596 cfs with only 1.0' of flow over road. No other structures or road crossings for 1.8 miles downstream

Preliminary classification: Class "a": Low Hazard

State requirements vary. For detailed information consult the appropriate state agency.

Step 3. Compute stream channel protection (Cp_v) and overbank protection (Q_{p-10}) volume requirements

Cp_v was previously computed to require 1.65 ac-ft of storage for 24 hour detention of runoff from the post-developed 1 year storm (see Table 2).

The preliminary Q_{p-10} was previously computed to require 2.83 ac-ft of storage for attenuation of the post-developed 10 year storm to the pre-developed peak discharge (see Table 2).

Note that at this step the volume for Q_{p-10} is computed only on a preliminary basis using TR-55 "short-cut" routing method. 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-10} = 2.83 * 1.15 = 3.25 ac-ft. The final volume is verified using SCS TR-20 "Project Formulation-Hydrology" computer program.

To complete this step the designer will need to develop the site hydrologic analysis using TR-55 or TR-20 to compute the volume of runoff for the 1 year post-developed storm and peak discharges for the 10 year pre-and post-developed storms (see Example 1, in sizing options section).

Step 4. Water Quality Allocations

The recharge volume must be provide in a separate facility. Assume that 100% of recharge volume (0.24 ac-ft) is provided in an upland bioretention facility. The required water quality volume for this facility is therefore reduced by this amount:

New WQ_v required in pond: (1.08 ac-ft) - (0.24 ac-ft) = 0.84 ac-ft

Step 5. Pretreatment Volume

Size wet forebay to treat 0.1"/impervious acre. (13.8 ac) (0.1") (1'/12") = **0.12 ac-ft**

(volume is included in WQ_v as part of permanent pool volume)

Specific treatment volumes may vary, but 0-1"/impervious acre is typical. See Design Criteria - Ponds

Step 6. Allocation of permanent pool volume and ED volume

Size permanent pool volume to contain 50% of new WQ_v (0.84 ac-ft, see step 4):

½ (0.84 ac-ft) = **0.42 ac-ft**. (include 0.12 ac-ft as a forebay within this volume)

Size extended detention volume to contain 50% of WQ_v: (½) (0.84 ac-ft) = **0.42 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 7. Conduct pond grading and determine storage available for permanent pool and ED, and compute ED orifice size

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 forebays), extended detention (WQ_v-ED), Cp_v-ED, and 10 year storm, plus sufficient additional storage to pass the 100 year storm with minimum freeboard. An elevation-storage table and curve is prepared using the average area method for computing volumes. See Figure 2 for pond grading and Figure 3 for Elevation-Storage Data.

Figure2. Plan View of Pond Grading

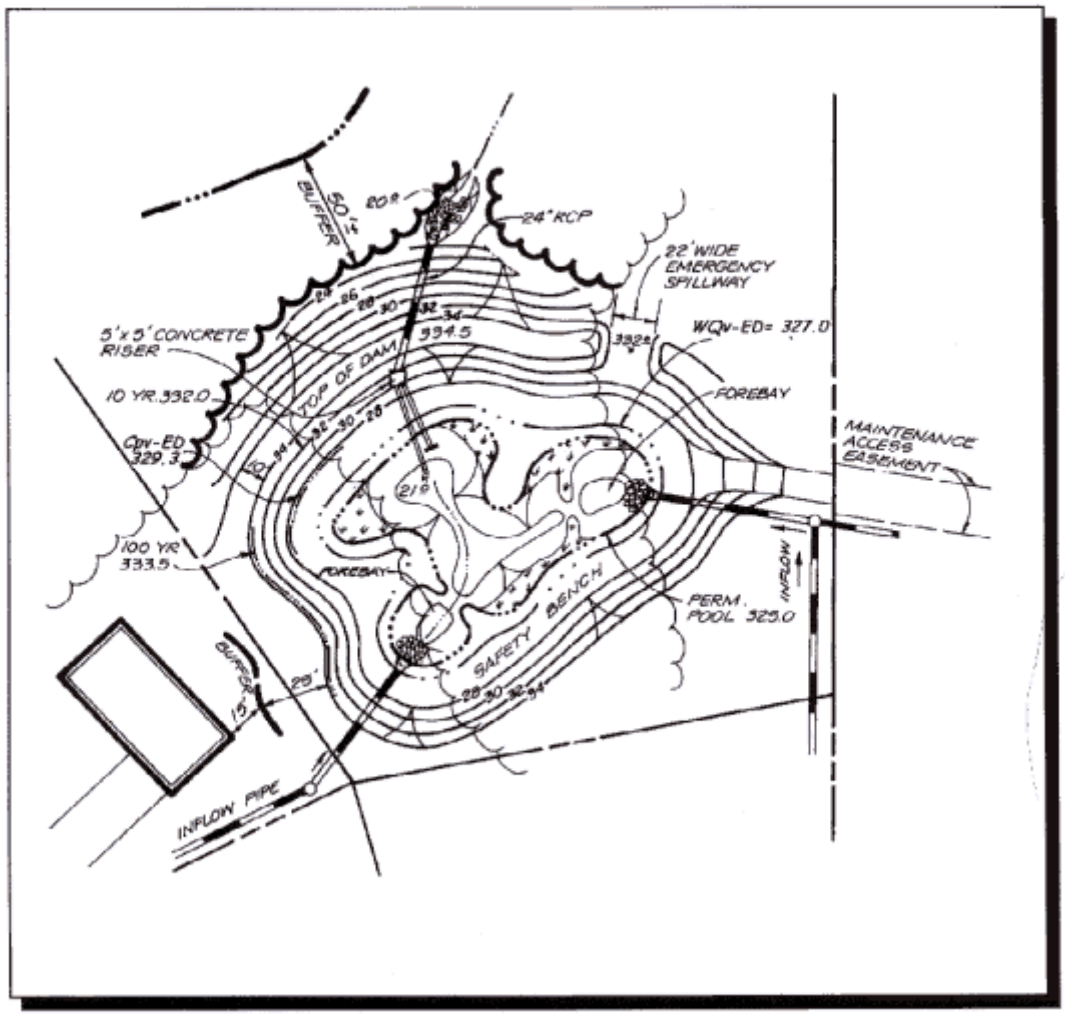
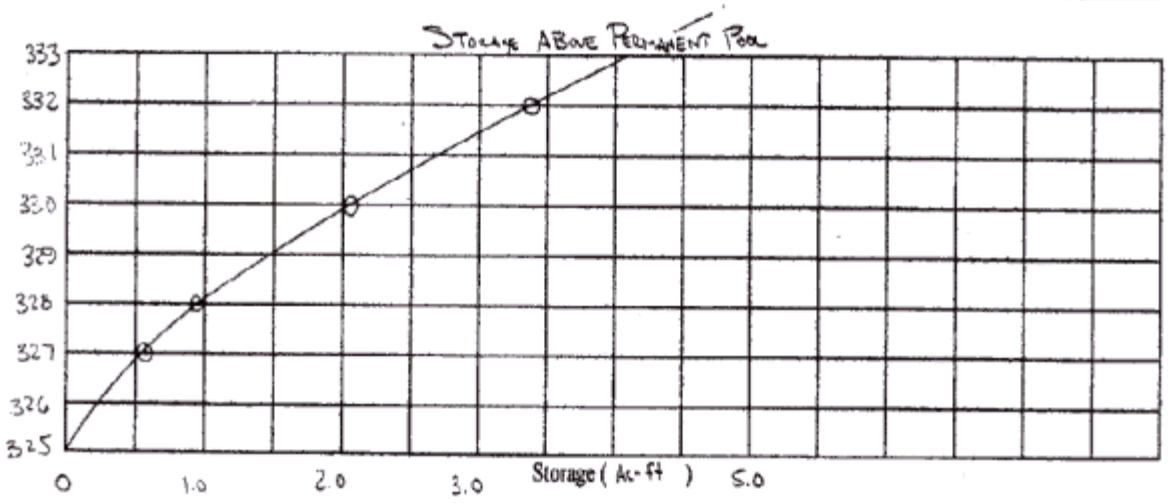


Figure 3. Storage-Elevation Table/ Curve

Elevation (MSL)	Area (in²)	Area (ft²) (50%+UE)	Average Area (ft²)	Depth (ft)	Volume (ft³)	Σ Volume (ft³)	Σ Volume (ac-ft)	Σ Volume above permanent pool (ac-ft)
321	1.26	3150						
324	3.33	8325	5738	3	17,214	17,214	0.40	
325	4.16	10,900	9363	1	9363	26,577	0.61	
327	5.54	13 850	12,125	2	24,250	50,827	1.17	0.56
328	8.79	21,650	17,850	1	17,850	68,677	1.58	0.97
330	10.54	26,350	24,100	2	48,200	116,877	2.68	2.07
332	12.13	30,975	28,113	2	56,825	173,702	3.98	3.36
334	14.21	35,525	33,000	2	66,000	239,702	5.50	4.85



Pond bottom set at elevation 321.0. Provide gravity flow to allow for pond drain, set riser invert at 320.8 and barrel outlet elevation at 320.5.
Required permanent pool volume = 0.42 ac-ft. From the elevation-storage table, read elevation 325.0 (0.61 ac-ft > 0.42 ac-ft) OK
Set permanent pool water surface elevation (wse) = 325.0

Forebay volume provided in two pools with avg. vol. = 0.08 ac-ft each (0.16 ac-ft > 0.12 ac-ft) OK

Required extended detention volume (WQ_v-ED)= 0.42 ac-ft. From the elevation-storage table, read elevation 327.0 (0.56 ac-ft > 0.42 ac-ft) OK.
Set ED wse = 327.0

Note: Total storage at elevation 327.0 = 1.17 ac-ft (greater than required WQ_v of 0.84 ac-ft)

Compute the required WQ_v-ED orifice diameter to release 0.56 ac-ft over 24 hours
Average ED release rate = 0.56 ac-ft (43,560 ft^2/ac) / (24 hr) (3600 sec/hr) = 0.28 cfs
Average head = (327 - 325.0)/ 2 = 1.0'
Use orifice equation to compute cross-sectional area and diameter
$Q = CA(2gh)^{½}$, for h = 1.0
$A = 0.28 \text{ cfs} / [(0.6)((2)32.2'/s^2)(1.0'))^{½}]$
A = 0.06 ft^2, A =d^2 / 4; dia. = 0.27 ft = 3.3" Use 4" pipe with 4" gate valve
$Q_{WQv\text{-}ED} = (0.6)\ (0.06\ ft^2)\ [(2)(32.2'/s^2)^{½}]\ (h^{½})$, $Q = (0.29)\ h^{½}$, where: h = wse - 325.14

Step 8. Compute release rate for Cp_v-ED control and establish Cp_v elevation

Required Cp_v storage = 1.65 ac-ft (see Table 2). From the elevation-storage table, read elevation 329.3'.
Set Cp_v wse = 329.3
Size low flow orifice to release 0.84 cfs @ elev. 329.3 (see Table 2)
@ wse 329.9, WQ_v-ED orifice releases 0.59 cfs, Cp_v-ED orifice release = 0.84 -0.59 = 0.25 cfs
Head = (329.3 - 327.0) = 2.3'

Use orifice equation to compute cross-sectional area and diameter
$Q = CA(2gh)^{½}$, for h = 2.3'
$A = 0.25 \text{ cfs} / [(0.6)((2)(32.2'/s^2)(2.3'))^{½}]$
A = 0.034 ft^2, A =d^2 / 4; dia. = 0.21 ft = 2.5" Use 4" pipe with 4" gate valve
$Q_{Cpv\text{-}ED} = (0.6)\ (0.034\ ft^2)\ [(2)\ (32.2'/s^2)^{½}]\ (h^{½})$, $Q_{Cpv\text{-}ED} = (0.16)\ (h^{½})$, where: h = wse - 327.10

Step 9. Calculate Q_p (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.

Size to capture 3.25 ac-ft (Table 2). From elevation-storage table (Figure 3), read elevation 331.8.

Size 10 year slot to release 50.4 cfs (Table 1) at elevation 331.8.
@ wse 331.8, WQ_v-ED orifice releases 0.75 cfs, Cp_v-ED orifice releases 0.35 cfs, therefore;
allowable $Q_{p\text{-}10}$ = 50.4 cfs - (.75 + .35) = 49.3 cfs
Max head = (331.8 - 329.3) = 2.5'
Use weir equation to compute slot length
$Q = CLH^{3/2}$
$L = 49.3 \text{ cfs} / (3.1)\ (2.5^{3/2}) = 4.0$ ft

Use 5' x 2.5' slot for 10 year release (slot should be slightly larger than needed so as to have the barrel control before slot goes from weir flow to orifice flow)

Check orifice equation cross-sectional area
$Q = CA(2gh)^{½}$, for h = 1.25'
A = (5) (2.5') = 12.5 ft^2
$Q = 0.6\ (12.5\ ft^2)\ [(64.4)\ (1.2)]^{½} = 67.3$ cfs, > 49.3 cfs use weir equation

$Q_{10} = (3.1)\ (5')\ H^{3/2}$, $Q_{10} = (15.5)\ H^{3/2}$, where H = wse - 329.3

Size barrel to release approximately 50.4 cfs at elevation 331.8

Check inlet condition: (use FHWA culvert charts)
H_w = 331.8-320.5 = 11.3 ft
Try 24" diameter RCP, Using FHWA Chart 2, entrance condition 1
H_w / D = 11.3/2 = 5.65, Discharge = 51 cfs

Check outlet condition: (use NRCS pipe flow equation--NRCS, 1984)

$Q = a\,[(2gH)/1+k_m+k_pL]^{½}$
where: Q = discharge in cfs
a = pipe cross sectional area in ft^2
g = acceleration of gravity in ft/sec^2
H = head differential (wse - downstream centerline of pipe or tailwater elev.)
k_m = coefficient of minor losses (use 1.0)
k_p = pipe friction loss coefficient (= $5087n^2/d^{4/3}$, d in ", n is Manning's n)
L = pipe length in ft

H = 331.8 - (320.0 + 1.0) = 10.8'
for 24" RCP, 70 feet long: $Q = 3.14\,[(64.4)\,(10.8)\,/\,1+1+(.0126)\,(70)]^{½}$ = 48.8 cfs
48.8 cfs < 51 cfs, use outlet equation
$Q = (14.85)\,H^{½}$, where H = wse - 321.0

Note: pipe will control flow before high stage slot goes from weir flow to orifice flow

Complete Stage-Storage Discharge Table (Figure 4) up to preliminary 10 Year wse (331.2) androute 10 year post-developed condition inflow using TR-20 (see Figures 5 and 6 for input and summary files from TR-20 run). Note TR-20 routing computes 10 year wse at 332.0 with discharge = 50.1 cfs.

Figure 4. Stage-Storage Discharge Characteristics Table

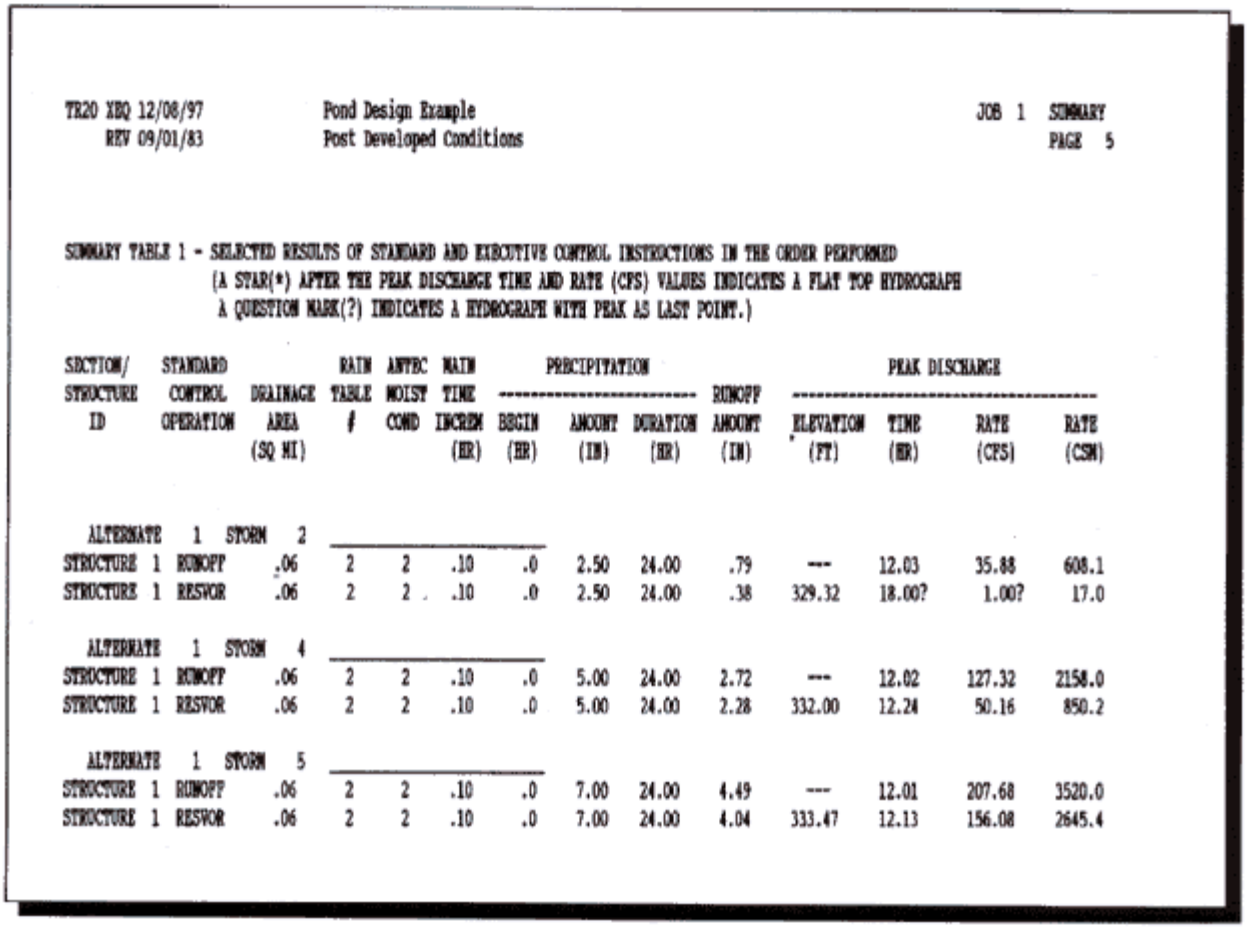
Storage - Elevation - Discharge Data																
Elevation (MSL)	Storage (ac-ft)	Low Flow WQ_v-ED 3.2" Equiv. Dia. Orifice		Riser						Barrel				Emergency Spillway		Total Discharge
				CPV-ED 2.9" Equiv. Dia. Orifice		High Stage Slot (6'x15')				Inlet		Pipe				
						Orifice		Weir								
		H (ft)	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)	(cfs)
325	0.0	0	0													0
325.5	0.14	.36	0.17													0.17
326.0	0.28	0.86	0.27													0.27
326.5	0.42	1.36	0.34													0.34
327.0	0.56	1.86	0.90	0	0											0.90
327.5	0.77	2.36	0.99	0.4	0.10											0.59
328.0	0.97	2.86	0.99	0.8	0.15											0.64
329.0	1.52	3.86	0.86	1.8	0.22											0.78
329.3	1.66	4.16	.59	2.2	0.24	N/A	-	0	0							0.83
329.5	1.80	4.36	0.61	2.4	0.25			0.2	1.39							2.25
330.0	2.07	4.86	0.69	2.9	0.27			0.7	9.08							10.0
331.0	2.73	5.86	0.70	3.9	0.32			1.7	34.4	10.5	45	10.0	47.0			35.4
331.8	3.25	6.66	0.75	4.5	0.35	1.25	61.3	2.5	61.3	11.3	51	10.8	48.8			98.8
332.0	3.38	-	-	-	-	-	-	-	-	11.5	52	11.0	99.3	0	0	43.3
333.	4.14	-	-	-	-	-	-	-	-	12.5	53	12.0	51.4	1.0	51	102.9
334	4.89	-	-	-	-	-	-	-	-	13.5	54	13.0	53.5	2.0	164	217.5

Project Name: Pond Design Example Date: Sept By: _____

Figure 5. TR-20 Computer Program Input File

```
JOB TR-20                                   FULLPRINT                    NOPLOTS
TITLE      Pond Design Example
TITLE      Post Developed Conditions
 3 STRUCT       1
 8                          325.0          0.0          0.0
 8                          325.5          0.17         0.14
 8                          326.0          0.27         0.28
 8                          326.5          0.34         0.42
 8                          327.0          0.40         0.56
 8                          327.5          0.54         0.68
 8                          328.0          0.64         0.97
 8                          329.0          0.78         1.52
 8                          329.3          0.83         1.66
 8                          329.5          2.25         1.80
 8                          330.0          10.0         2.07
 8                          331.0          35.4         2.73
 8                          331.8          48.8         3.25
 8                          332.0          49.9         3.38
 8                          333.0          102.4        4.14
 8                          334.0          217.5        4.89
 9 ENDTBL
 6 RUNOFF 1      1      2 0.059          -78.0          0.19        1 0    0 0 1
 6 RESVOR 2      1 2    3 325.0                                     1          1
   ENDATA
 7 INCREM 6              0.1
 7 COMPUT 7      1      1 0.0            2.5            1.0         2 2    1  02
   ENDCMP 1
 7 COMPUT 7      1      1 0.0            5.0            1.0         2 2    1  04
   ENDCMP 1
 7 COMPUT 7      1      1 0.0            7.0            1.0         2 2    1  05
   ENDCMP 1
   ENDJOB 2
```

Figure 6. TR-20 Computer Program Results - Summary Table



TR20 XEQ 12/06/97 Pond Design Example JOB 1 SUMMARY
REV 09/01/83 Post Developed Conditions PAGE 5

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)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 2													
STRUCTURE 1	RUNOFF	.06	2	2	.10	.0	2.50	24.00	.79	---	12.03	35.88	608.1
STRUCTURE 1	RESVOR	.06	2	2	.10	.0	2.50	24.00	.38	329.32	18.00?	1.00?	17.0
ALTERNATE 1 STORM 4													
STRUCTURE 1	RUNOFF	.06	2	2	.10	.0	5.00	24.00	2.72	---	12.02	127.32	2158.0
STRUCTURE 1	RESVOR	.06	2	2	.10	.0	5.00	24.00	2.28	332.00	12.24	50.16	850.2
ALTERNATE 1 STORM 5													
STRUCTURE 1	RUNOFF	.06	2	2	.10	.0	7.00	24.00	4.49	---	12.01	207.68	3520.0
STRUCTURE 1	RESVOR	.06	2	2	.10	.0	7.00	24.00	4.04	333.47	12.13	156.08	2645.4

Step 10. Size emergency spillway, calculate 100 year water surface elevation, and set top of embankment elevation

The 10 year wse is at 332.0. Set the emergency spillway at this elevation and use NRCS-Design Data for Earth Spillways (Engineering Field Manual, RTSC-NE-ENG 1110)

Q_{100} inflow = 216.3 cfs.
Try 22' wide vegetated emergency spillway with 2:1 side slopes.
@ elevation 334.0, H = 2.0', Q_{ES} = 164 cfs. Q_{pS} = 53.5 cfs

$Q_{ES} + Q_{PS}$ = 217.5, will be able to safely convey Q_f = 216.3. (see TR-20 routing for exact elevations and discharges).

100 year wse = 333.5, so set top of embankment with 1 foot of freeboard at elevation 334.5.

Check pond classification assumption: Height = 332-320 = 12', equals assumed embankment height, Pond will remain a Low Hazard.

See Figure 7 for profile through principal spillway of the facility.

See Figure 8 for a schematic of the riser.

Table 3. Summary of Controls Provided

Control Element	Type/Size of Control	Storage Provided	Elevation	Discharge	Remarks
Units		Acre-feet	MSL	cfs	
Permanent Pool	.	0.61	325.0	0	part of WQ_v
Forebay	submerged berm	0.16	325.0	0	included in permanent pool vol.
Extended Detention (WQ_v-ED)	4" pipe, sized to 3.3" equivalent diameter	0.56	327.0	0.28	part of WQ_v, vol. above perm. pool, discharge is average release rate over 24 hours
Channel Protection (Cp_v-ED)	4" pipe sized to 2.5" equivalent diameter	1.65	329.3	0.84	volume above perm. pool, discharge is max release rate @elev 329.3
Overbank Protection ($Q_{p\text{-}10}$)	5'x2.5' slot, 24"barrel	3.38	332.0	50.1	volume above perm. pool
Extreme Storm ($Q_{f\text{-}100}$)	22' wide earth spillway	4.51	333.5	155.4	volume above perm. pool

100 year wse = 333.5, so set top of embankment with 1 foot of freeboard at elevation 334.5.

Figure 7. Profile of Principle Spillway

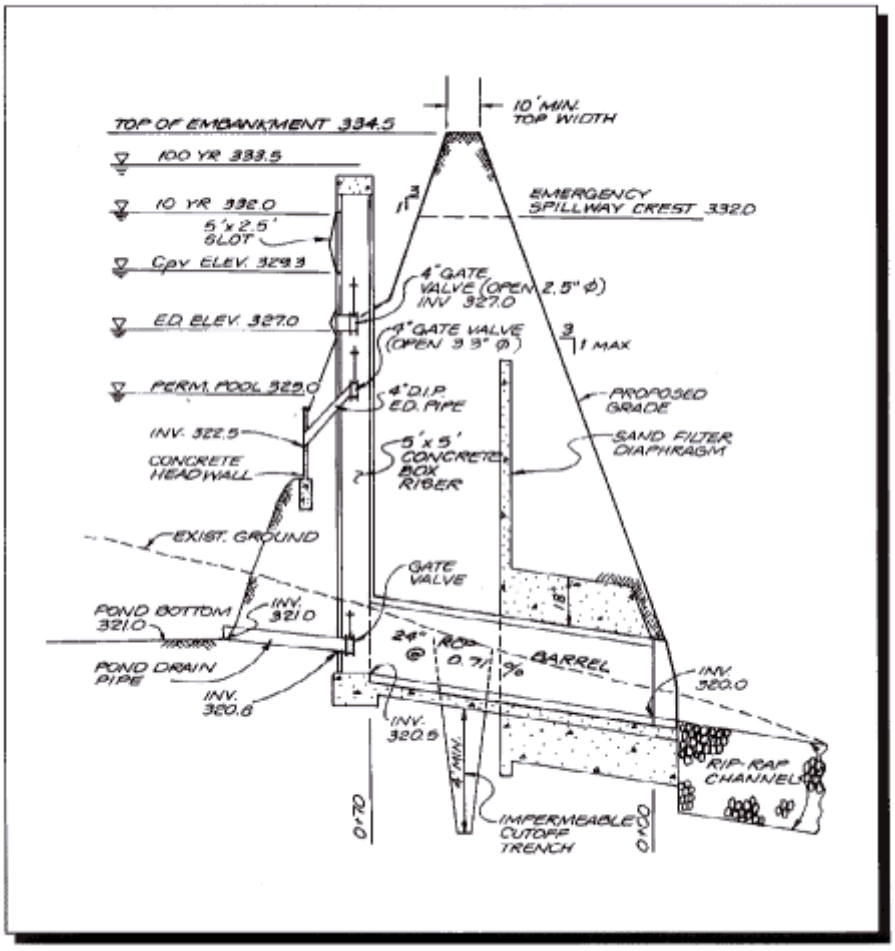
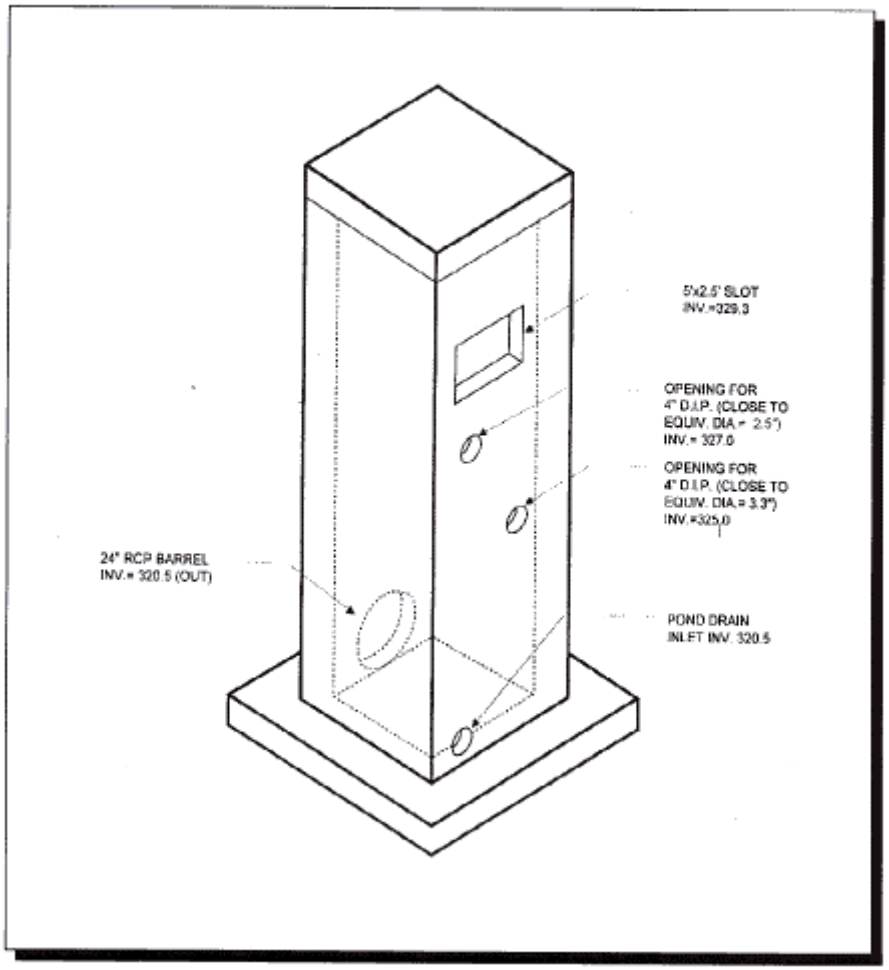
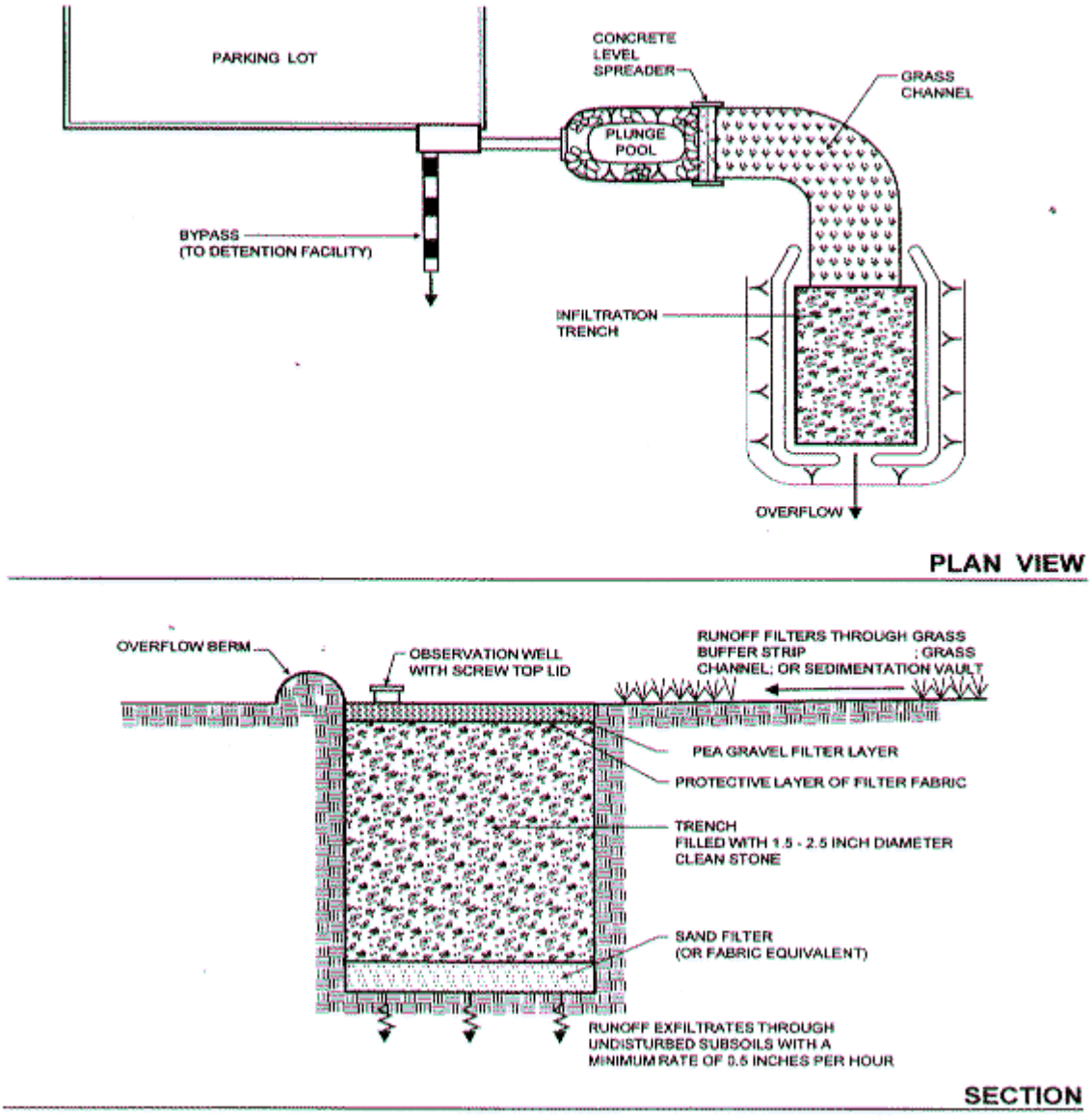


Figure 8. Schematic of Riser Detail



Stormwater Design Example: Infiltration Trench

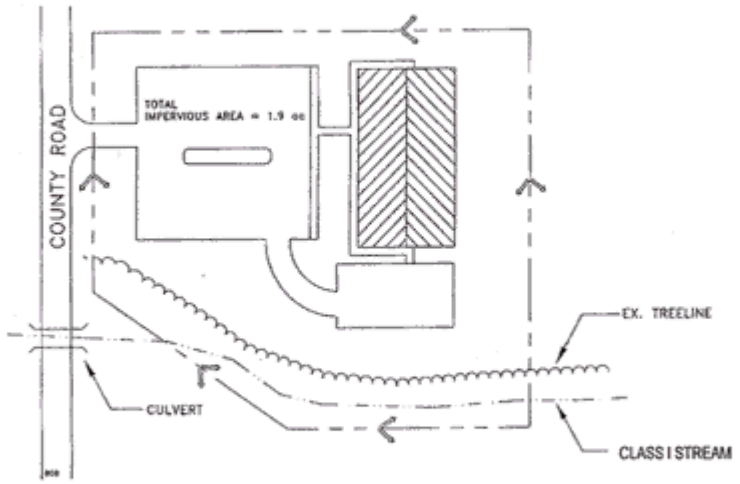


INFILTRATION TRENCH DESIGN EXAMPLE

This example describes in detail the design of an infiltration trench to treat stormwater runoff from the Brown Community Center (Figure 1). The trench must meet the following design criteria:

- Provide Recharge
- Treat the Water Quality Volume

Figure 1. Brown Community Center Site Plan



Base Data
Location: Anywhere, USA
Site Area = Total Drainage Area (a) = 3.0 ac
Impervious Area = 1.9 ac or I = 63.3%

Table 1. Hydrology Summary

Condition	$Q_{1\text{-year}}$	$Q_{2\text{-year}}$	$Q_{10\text{-year}}$	$Q_{100\text{-year}}$
	cfs	*cfs*	*cfs*	*cfs*
pre-developed	0.22	0.58	2.91	6.75
developed	5.08	7.11	13.97	22.69

Table 2. Treatment Volumes

Condition	Volume Required
	cubic feet
Water Quality Volume (WQ_v)	6,752
Recharge (Re_v)	1,688

Table 3. Site Specific Data

Criteria	Value
Soil	Sandy Loam
Percolation Rate	1"/hour
Ground Elevation at BMP	22'
Seasonally High Water Table	12'
Stream Invert	10'
Soil slopes	<1%

Step 1. Determine if Infiltration Is Feasible

The table 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 4. Infiltration Feasibility

Criteria	Status
Infiltration rate (f_c) greater than or equal to 0.52 inches/hour.	Infiltration rate is 1.0 inches/hour. OK.
Soils have a clay content of less than 20% and a silt/clay content of less than 40%.	Sandy 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 to bedrock layer.	Elevation of seasonally high water table: 12' Elevation of BMP location: 20'. The difference is 8'. Thus, the trench can be up to 6' 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.	3 acres. OK.
Setback 25 feet down-gradient from structures.	Fifty feet straight-line distance between the parking lot and the tree line. OK if the trench is 25' wide or narrower.

This table presents examples for infiltration feasibility criteria. Specific criteria may vary. See Selection Criteria for more details.

Step 2. Compute Water Quality Volume:

WQ_v previously determined to be 6,752 cubic feet.

Step 3. Compute Recharge Volume

The recharge volume was previously calculated as 1,688 cubic feet. The infiltration trench will automatically meet the recharge requirement because it infiltrates the entire water quality volume which is greater than the recharge volume.

Step 4. Compute WQ_v peak discharge (q_p)

Calculated as 2.6 cfs (see Sandfilter Design Example, Step 4).

Step 5. Size the infiltration trench.

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

$$A = \frac{WQ_v}{(nd + kT/12)}$$

This equation is borrowed from the Maryland Department of Natural Resources 1984 Publication "Maryland Standards and Specifications for Stormwater Management Infiltration Practices"

Where:

A = Surface Area
n = porosity
d = trench depth (feet)
k = percolation (inches/hour)
T=Fill Time (time for the practice to fill with water), in hours

Assume that:

n= 0.32 (see Chapter 3; porosity for a stone reservoir)
d= 5 feet (see above; feasibility criteria)
k= 1 inch/hour (see above; site data)
T= 2 hours

Therefore:

$$A = \frac{6{,}752 ft^3}{(0.32 \cdot 5 + 1 \cdot 2/12) ft}$$

$A = 3{,}800\ ft^2$

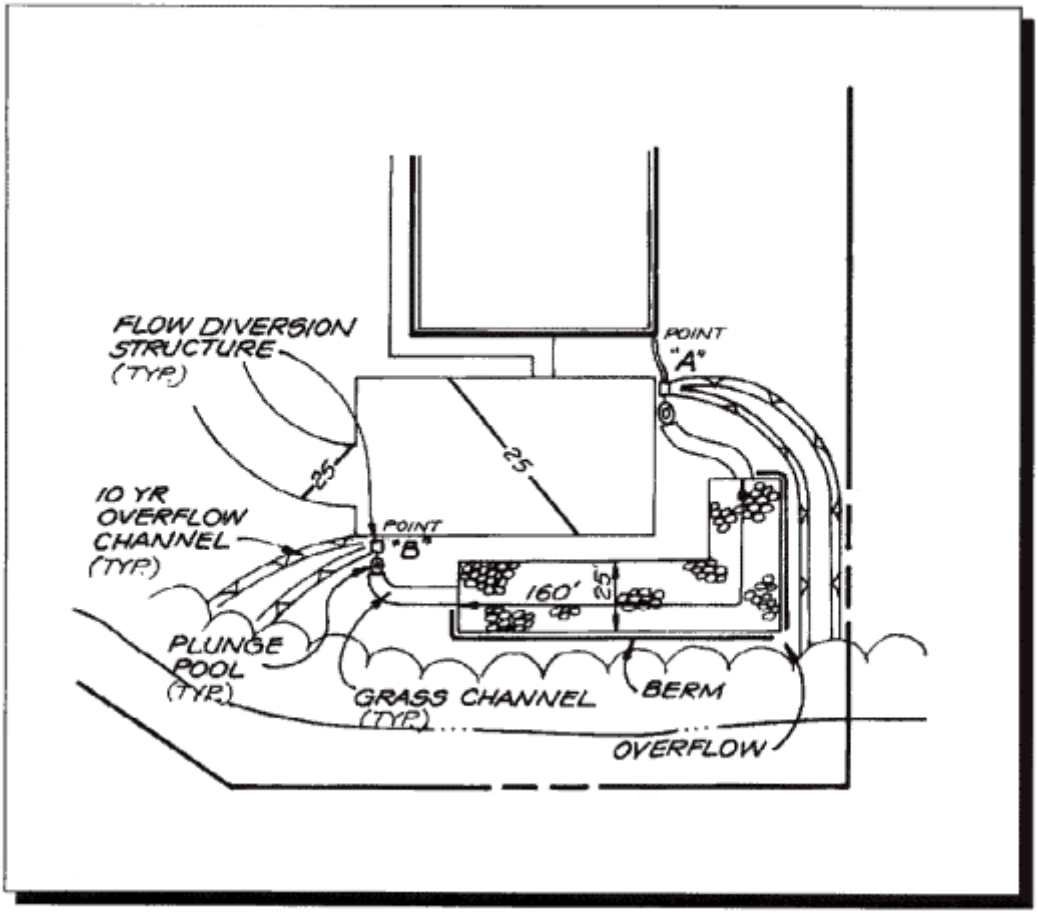
Since the width can be no greater than 25' (see above; feasibility), determine the length:

$$L = \frac{3{,}800 ft^2}{25 feet}$$

$L = 160\ feet$

Assume that of the runoff from the site drains to Point A and drains to Point B.
Use an L-shaped trench in the corner of the site (see Figure 2 for a site plan view).
The surface area of the trench is proportional to the amount of runoff it drains (e.g., the portion draining from Point A is half as large as the portion draining Point B).

Figure 2. Infiltration Trench Site Plan



Step 6. Size the flow diversion structures

Since two entrances are used, two flow diversions are needed.

For the entire site:

$Q_{10\text{-year}}$ = 13.97 cfs (See site data)
Peak flow for WQ_v = 2.6 cfs. (Step 4).
For the first diversion (Point A)

Assume peak flows equal of their value for the entire site.

Thus, $Q_{10\text{-year}}$ = 4.7 cfs
Peak flow for WQ_v = 0.9 cfs

Size the low flow orifice to pass 0.9 cfs with 1.5' of head using the Orifice equation.
$Q=CA(2gh)^{½}$; 0.9 cfs = $0.6A(232.2\ ft/s^2\ 1.5')^{½}$
A=0.15 sq. ft. = $d^2/4$; d = 5.3"; use 6"

Size the 10-year overflow at 22'. Use a concrete weir to pass the 10-year flow (4.7 cfs.) Assume 1 foot of head to pass this event. Size using the weir equation.

$Q = CLH^{1.5}$; $L = Q/(CH^{1.5})$
$L = 3.8 \text{ cfs} / (3.1)(1)^{1.5} = 1.2'$; use 1.5'

Size the second diversion (Point B) using the same techniques.

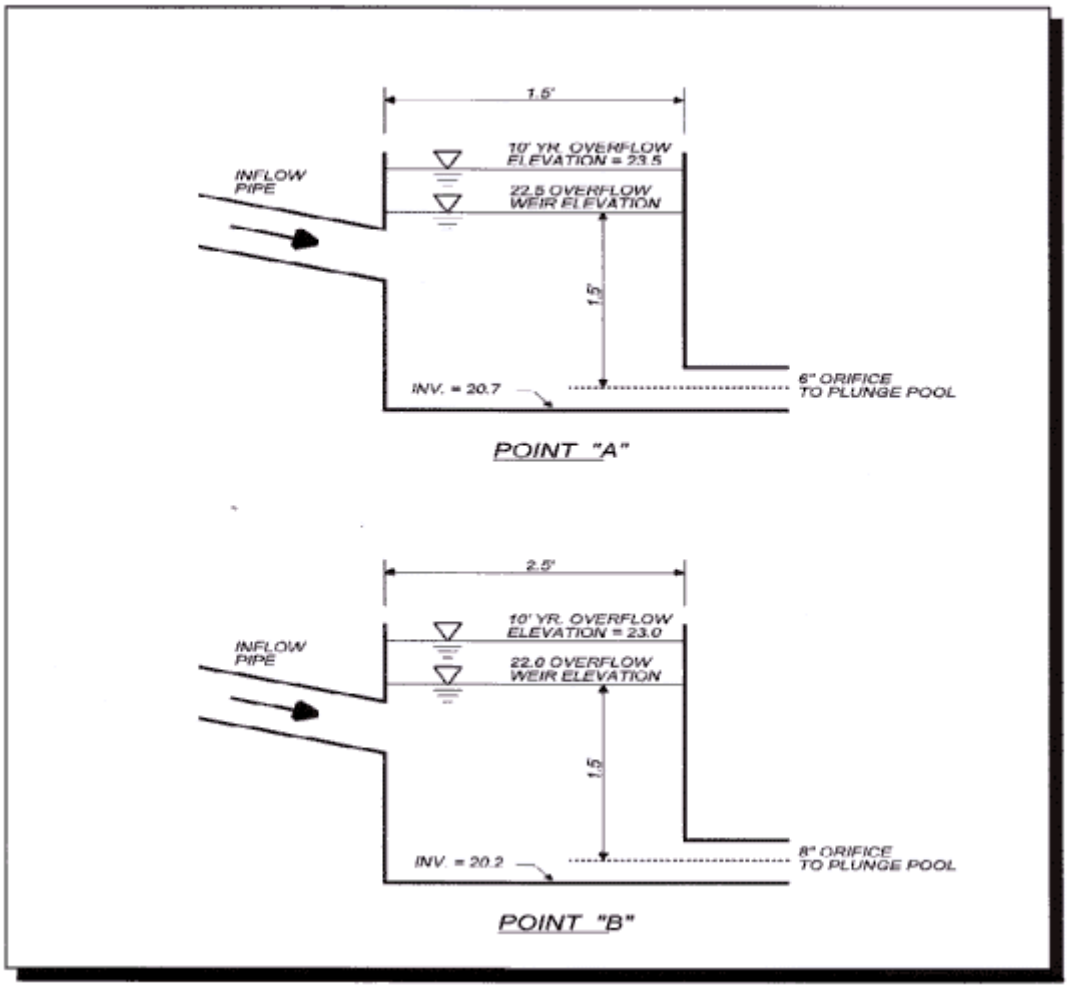
Peak flows equal of their volume for the entire site. Thus:

$Q_{10\text{-year}} = 9.3$ cfs
Peak flow for $WQ_v = 1.7$ cfs

Using the same calculations as above, and substituting the higher flows, d=7.3";
Use 8" for low flow orifice diameter.

For the 10-year overflow, L=2.4'. Use 2.5'.

Figure 3. Flow Diversion Structures



Step 7. Size pretreatment.

Must treat ¼ of the WQ_v. Therefore, treat 1,688 ft^3.

In general, infiltration practices should have significant pretreatment volumes to prevent clogging. (See Design Criteria - Infiltration)

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

Pea Gravel Filter
The 2" pea gravel filter layer covers the entire trench. Assuming a porosity of 0.32, the water quality treatment in the pea gravel filter layer is:
WQ_{filter}= (0.32)(2")(1 ft/12 inches)(4,000 ft^2) = 213 ft^3

*Plunge Pool*s
Use a 5'X10' plunge pool at Point A and a 10'X10' plunge pool at Point B with average depths of 2'.
WQ_{pool}= (10 ft)(10+5 ft)(2 ft^2) = 300 ft^3

Grass Channel
Thus, the grass channel needs to treat at least (1,688-213-300)ft^3 = 1,175 ft^3

Use a Manning's equation nomograph or software to size the swale.

The channel at point A should treat one third of 1,175 ft^3 or 392 ft^3

- Assume a 4' channel bottom and a Manning's n value of 0.15. Use a nomograph to size the swale; assume a 1% slope.
- Use a peak discharge of 0.9 cfs (Peak flow for one third of WQ_v)
- Compute velocity:

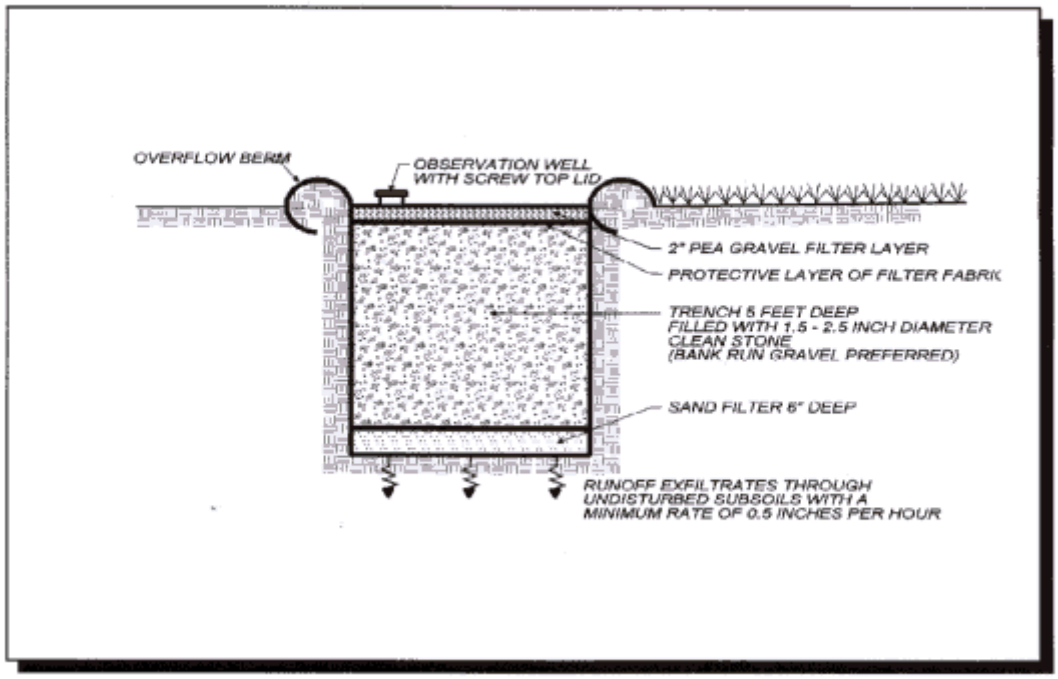
V=0.5 ft/s

- To retain the 1/3 of the WQ_v (2,250 ft^3) for 10 minutes, the length would be 300 feet.
- The swale only needs to treat 25% of the water quality volume minus the treatment provided by the plunge pool and the gravel layer, or:
  2,250 ft^3 (0.25) - (300 ft^3/3) - (213 ft^3/3) = 392 ft^3

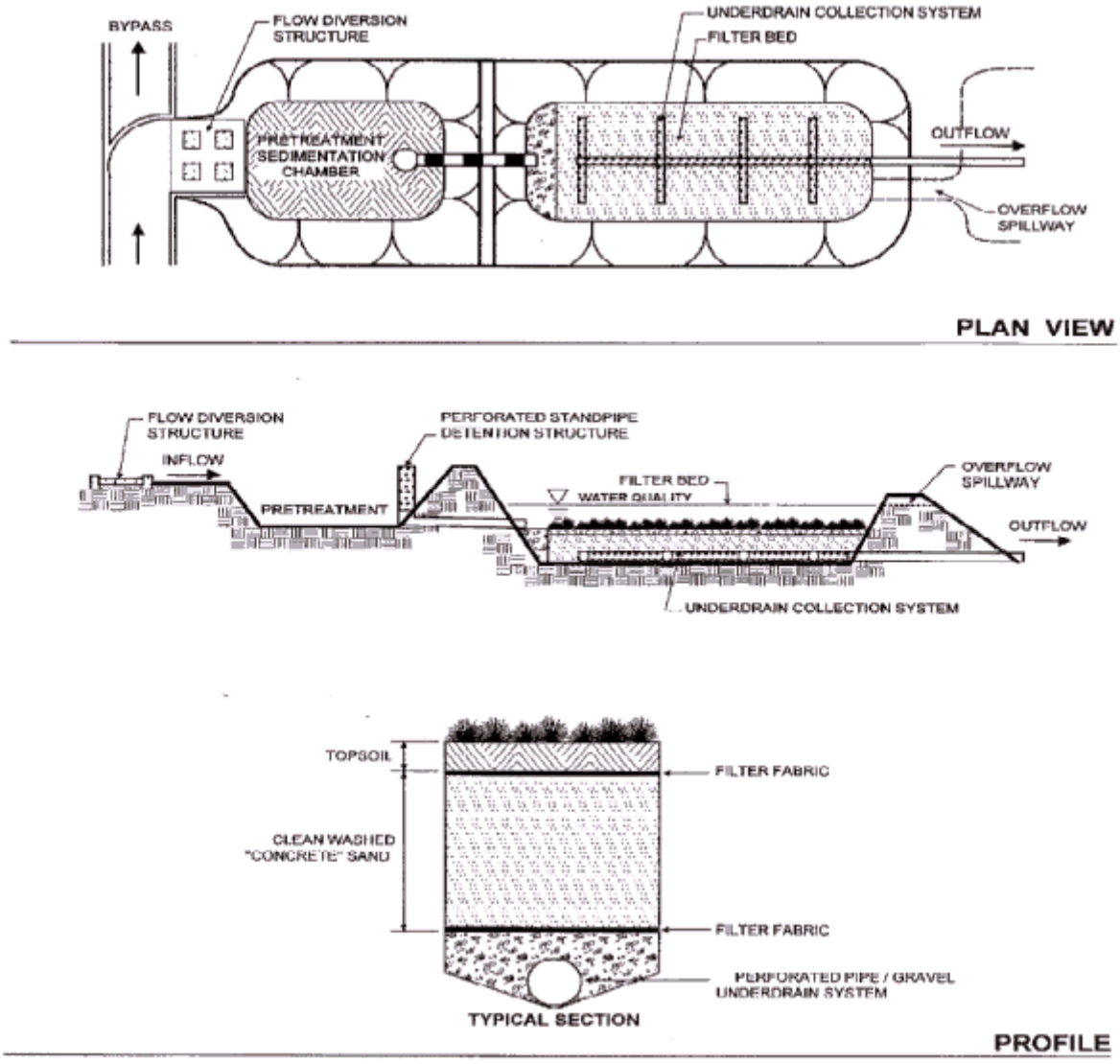
Therefore, adjust length:
L= (300 ft)(392 ft^3/2,250 ft^3) =52 feet. Use 55 feet.

Size the swale at point B using the same method. Q=1.7 cfs Slope = 0.5%
L=55 feet.

Figure 4. Infiltration Trench Cross Section



Stormwater Design Example: Sand Filter

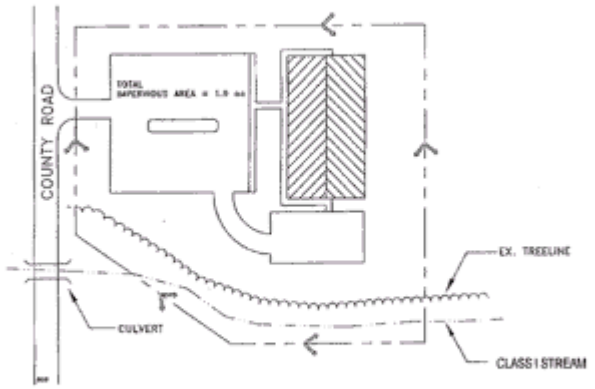


SANDFILTER DESIGN EXAMPLE

This example describes in detail the design of a sand filter to treat stormwater runoff from the Brown Civic Center (Figure 1). The filter must meet the following design criteria:

- Provide Recharge
- Treat the Water Quality Volume

Figure 1. Brown Community Center Site Plan



Base Data
Location: Anywhere, USA
Site Area = Total Drainage Area (a) = 3.0 ac
Impervious Area = 1.9 ac or I = 63.3%

Table 1. Hydrology Summary

Condition	CN	T_c	$Q_{1\text{-year}}$	$Q_{2\text{-year}}$	$Q_{10\text{-year}}$	$Q_{100\text{ year}}$
		hours	*cfs*	*cfs*	*cfs*	*cfs*
pre-developed	57	0.42	0.22	0.58	2.91	6.75
developed	83	0.16	5.08	7.11	13.97	22.69

Site Specific Data:

Existing ground elevation at BMP location is 22.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 13.0 feet. Adjacent creek invert is at 12.0'. Hydrology analyses results are in Table 1.

Step1. Compute Water Quality Volume:

WQ_v previously determined to be 6,752 cubic feet (Table 2).

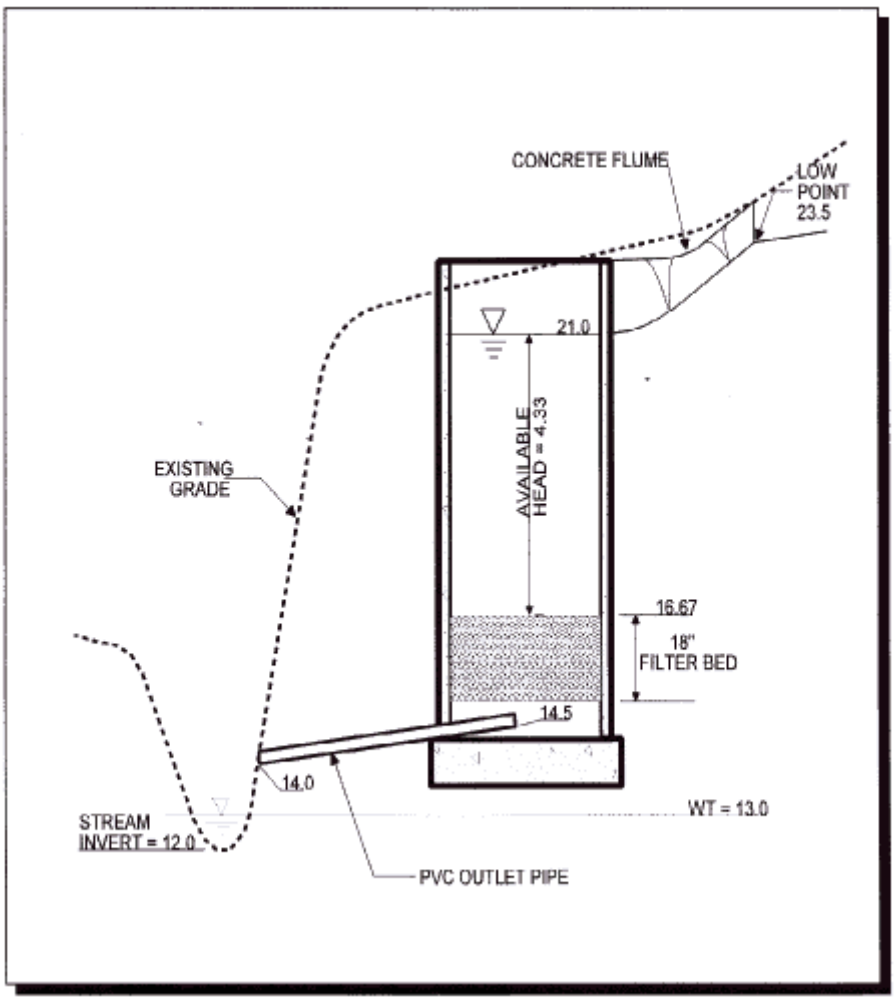
Table 2. Summary of General Design Information

Category	Volume Required
	cubic feet
Water Quality (WQ_v)	6,752
Recharge (Re_v)	1,688

Step 2. Determine available head (See Figure 2).

Low point at parking lot is 23.5'. Subtract 2' to pass Q_{10} discharge (21.5) and a half foot for channel to facility (21.0). Low point at stream invert is 12.0'. Set outfall underdrain pipe 2' above stream invert and add 0.5' to this value for drain (14.5). Add to this value 8" for the gravel blanket over the underdrains, and 18" for the sand bed (16.67). The total available head is 21.0 - 16.67 or 4.33 feet. Therefore, the allowable depth (2) (h_f) = 4.33', and h_f = 2.17'.

Figure 2. Available Head Diagram



Step 3. Compute Recharge Volume:

This surface sand filter design will have an "open bottom" sedimentation basin to allow ground water recharge. The exact dimensions will be sized later, but the volume stored within the sediment chamber must at least be 1,688 cubic feet.

Step 4. Compute WQ_v peak discharge (q_p):

Compute modified CN for 1" rainfall (Note: In this example the WQ_v corresponds to the runoff from a 1" storm.)

P = 1.0"

Q = 0.62"

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

= $1000/[10+5*1.0+10*0.62-10(0.62^2+1.25*0.62*1.0)^{½}]$

= 95.6

Use CN = 96

For CN = 96 and the T_c = 0.16 hours, compute the q_p for a 1" storm. With the CN = 96, a 1.0" storm will produce 0.62" of runoff. From TR-55, Chapter 4, I_a = 0.083, therefore I_a/P = 0.083/1.0 = 0.083. From Exhibit 4 - II (page 4-6 of TR-55), q_u = 900 csm/in, and therefore q_p = (900 csm/in) (0.62") (3.0 ac/640ac/sq mi.) = 2.6 cfs.

The modified curve number approach adjusts for the fact that TR-55 typically underpredicts the runoff from small storm events. The curve number is calculated based on the runoff volume calculated in the "Sizing Options Design Example".

Step 5. Size flow diversion structure (see Figure 3):

Size a low flow orifice to pass 2.6 cfs with 1.5' of head using the Orifice equation.

$Q = CA(2gh)^{1/2}$; 2.6 cfs = (0.6) (A) $[(2) (32.2 ft/s^2) (1.5')]^{1/2}$

A = 0.44 sq ft = $d^2/4$: d = 0.75' or 8.0"; use 8 inches

Size the 10-year overflow as follows: the 10-year WSE is set at 21.0. Use a concrete weir to pass the ten-year flow (14.0 cfs) into a grassed overflow channel using the Weir equation. Assume 2' of head to pass this event.

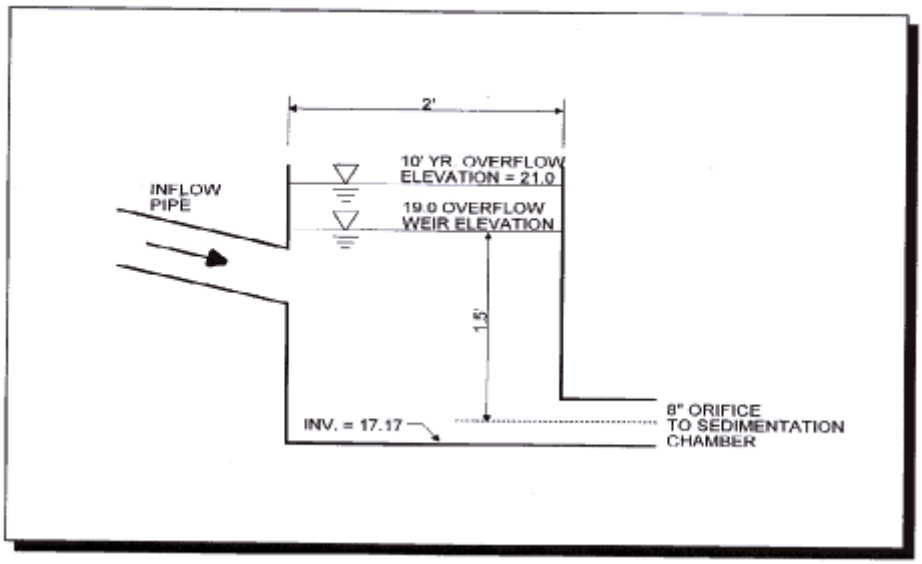
$Q = CLH^{3/2}$

* = 3.1 (L) $(2')^{1.5}$

L = 1.6'; use L = 2'-0" which sets flow diversion chamber dimension.

Weir wall elev. = 19.0. Set low flow invert at 19.0 - [1.5' + (0.5*8"*1ft/12")] = 17.17.

Figure 3. Flow Diversion Structure



Step 6. Size filtration bed chamber (see Figure 4):

From the City of Austin's 1988 Environmental Criteria Manual (Darcy's Law): +

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

where d_f = 18" (Depth of the filter)

k = 3.5 ft/day (Permeability of the sand)

h_f = 2.17' (Average head above the filter)

t_f = 40 hours (Drawdown time)

A_f = (6,752 cubic feet) (1.5') / [3.5 (2.17' + 1.5') (40hr/24hr/day)]

A_f = 473.1 sq ft; using a 2:1 ratio, say filter is 15.5' by 30.5' (= 473 sq ft)

Step 7. Size sedimentation chamber

From Design Criteria - Filters, (Camp-Hazen), for I < 75%: $A_s = 0.066 (WQ_v)$

A_s = 0.066 (6,752 cubic ft) or 445.6 sq ft

given a width of 15.5 feet, the length will be 445.6'/15.5' or 28.75 feet (use 15.5'x28'-9")

Step 8. Compute V_{min}

$V_{min} = ¾(WQ_v)$ or 0.75 (6,752 cubic feet) = 5,064 cubic feet

See Design Criteria - Filters. Typically, the volume within the practice can be less than the total water quality volume for filter designs.

Step 9. Compute volume within practice:

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

V_f = (473 sq ft) (1.5') (0.4) = 284 cubic feet

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

$V_{f\text{-}temp}$ = 2 (2.17') (473 sq ft) = 2053 cubic feet

Compute remaining volume for sedimentation chamber (V_s):

$V_s = V_{min}$ - [V_f + $V_{f\text{-}temp}$] or 5,064 - [284 + 2053] = 2,727 cubic feet

compute height in sedimentation chamber (h_s): $h_s = V_s/A_s$

(2,727 cubic ft)/(15.5' x 28.75') = 6.12' which is larger than the head available (4.33'); increase the size of the settling chamber, using 4.33' as the design height;

(2727 cubic ft)/4.33' = 630 sq ft; 630/15.5' yields a length of 40.6 feet (say 40')

new sedimentation chamber dimensions are 15.5' by 40'

check recharge requirements: with adequate preparation of the bottom of the settling chamber (rototill earth, place gravel, then surge stone), the bottom will infiltrate water into the substrate. Note that runoff will enter the groundwater directly without treatment. This stone will eventually clog without protection from settling solids so use a removable geotextile to facilitate maintenance. Note that V_s = 2,727 cubic feet which is greater than the recharge requirement of 1,688 cubic feet. Also note that there is 2.17' of freeboard between bottom of recharge filter and water table.

Often, a depth of at least 2' is required between the practice bottom and the water table. See Design Criteria - Filters

Step 10. Compute overflow weir sizes

From sediment chamber (size to pass 2/3 of WQ_v peak discharge)

0.67 (2.6 cfs) = 1.7 cfs

From filtration chamber (size to pass 1/3 of WQ_v peak discharge)

0.33 (2.6 cfs) = 0.9 cfs

See Figure 5 for Site Plan View.

Figure 4. Plan and Profile of Surface Sand Filter

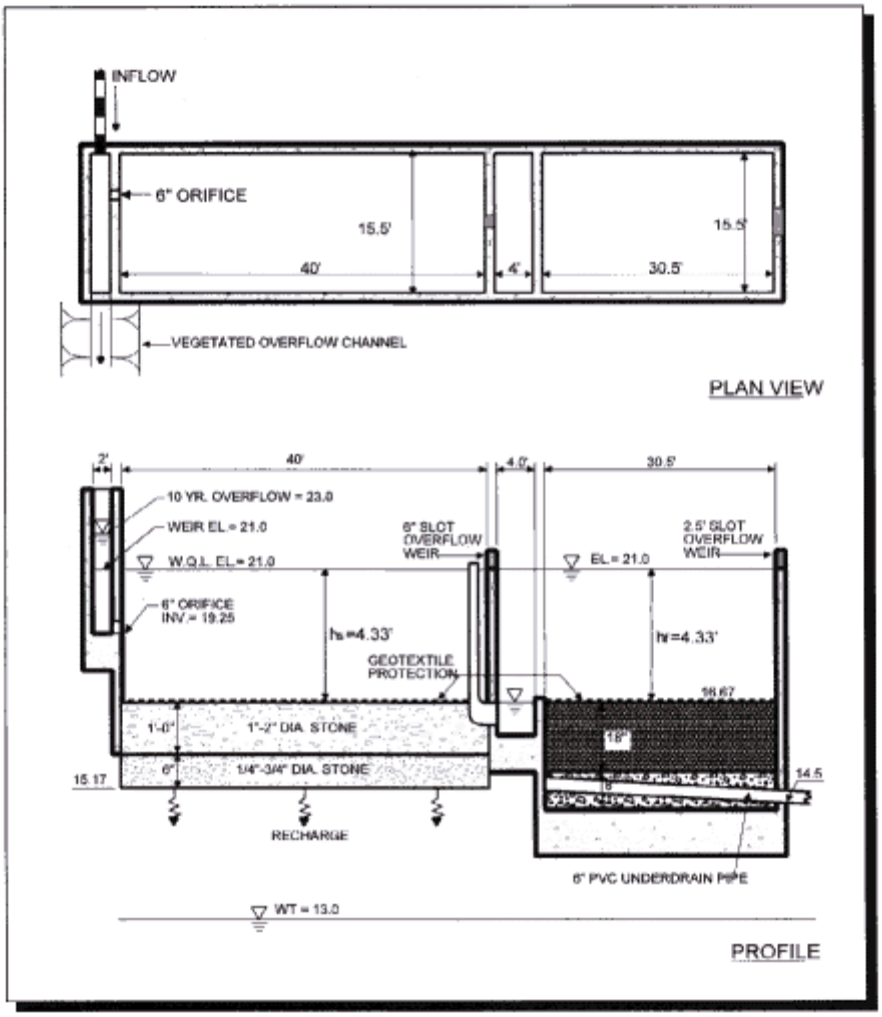
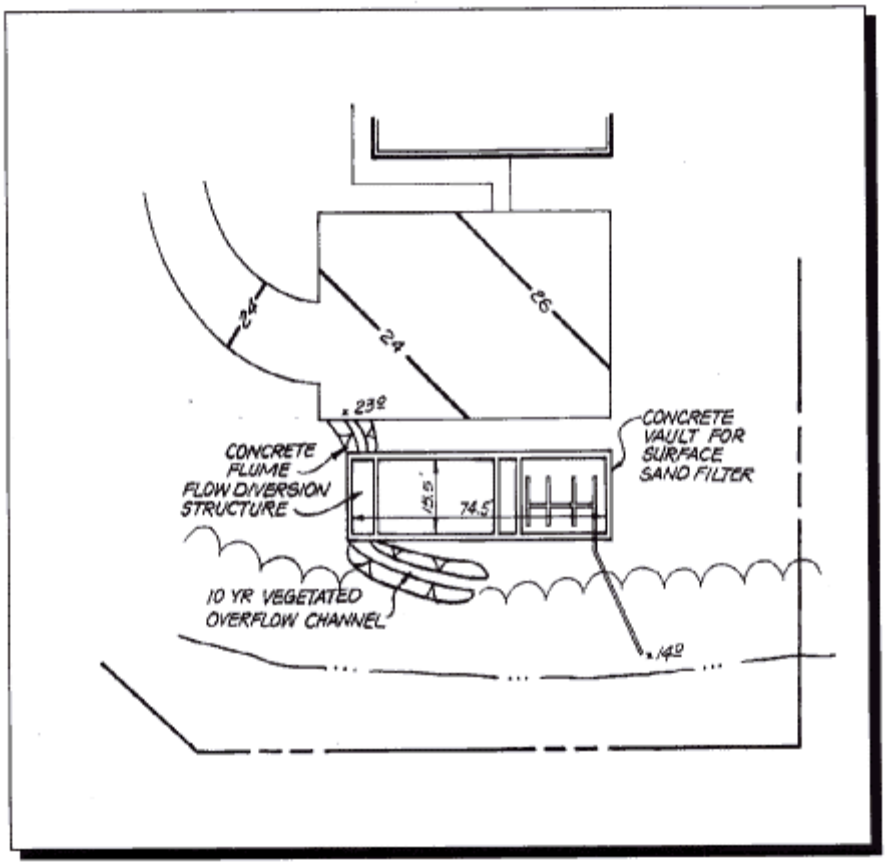
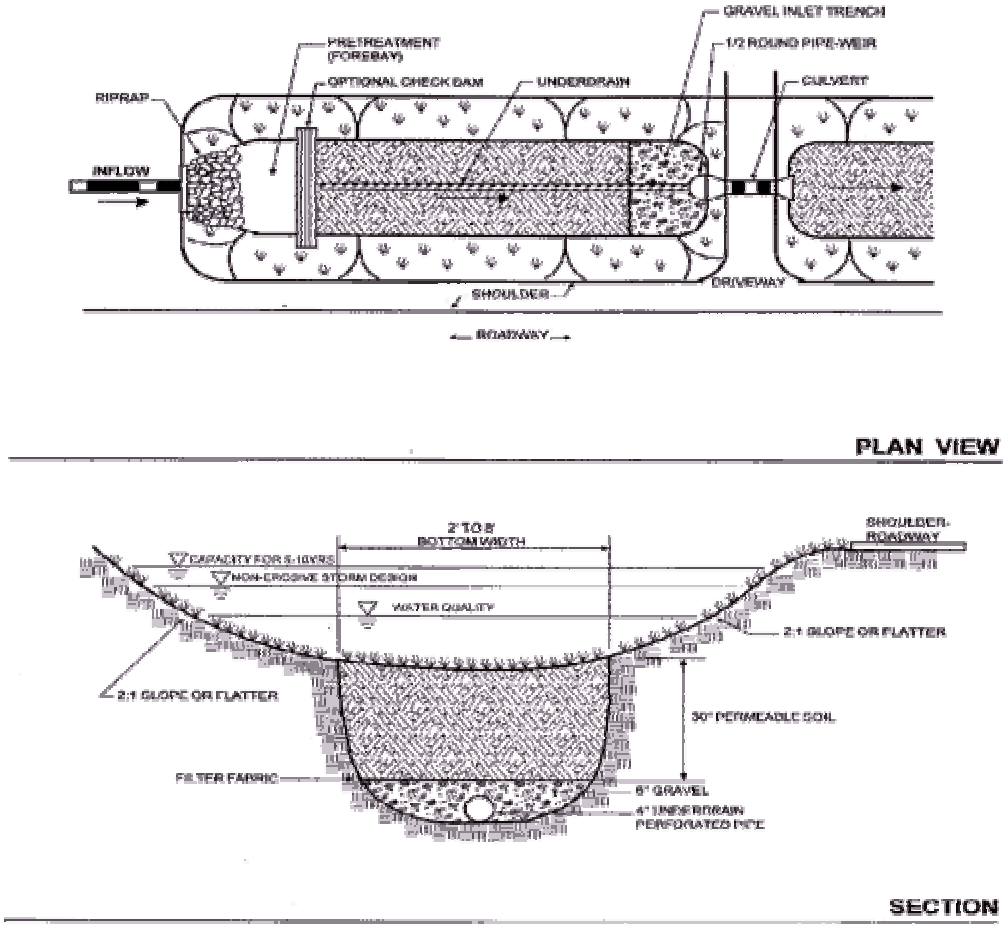


Figure 5. Surface Sand Filter Site Plan



Stormwater Design Example: Dry Swale



DRY SWALE DESIGN EXAMPLE

This example describes in detail the design of a dry swale to treat stormwater runoff from the Brown Community Center (Figure 1). The swale must meet the following design criteria:

- Provide Recharge
- Treat the Water Quality Volume
- Pass 2-Year + 10-Year storm events without eroding their channels

Design volumes and hydrology are provided in Tables 1 and 2.

Specific sizing criteria may vary between jurisdictions. For more information please consult the Sizing Options and Performance Criteria - Open Channels sections.

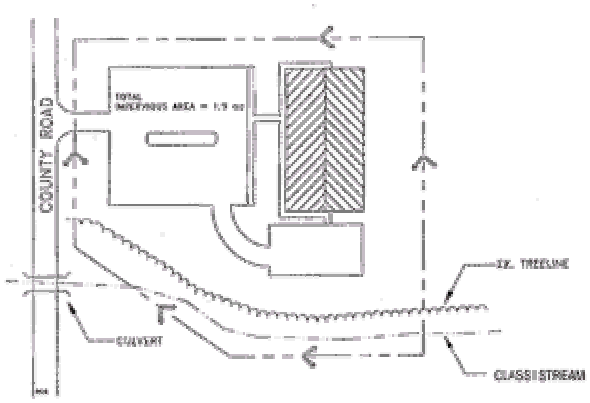
Table 1. Hydrology Summary

Condition	CN	$Q_{2\text{-year}}$	$Q_{10\text{-year}}$
		cfs	*cfs*
pre-developed	57	0.58	2.91
developed	83	7.11	13.97

Table 2. Required Treatment Volumes

Category	Volume Required
	cubic feet
Water Quality (WQ_v)	6,752
Recharge (Re_v)	1,688

Figure 1. Brown Community Center Site Plan



Base Data
Location: Anywhere, USA
Site Area = Total Drainage Area (a) = 3.0 ac
Impervious Area = 1.9 ac or I = 63.3%

Site Specific Data:

Existing ground elevation at STP location is 22.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 13.0 feet and underlying soils are silt loams (ML). Adjacent creek invert is at 12.0.

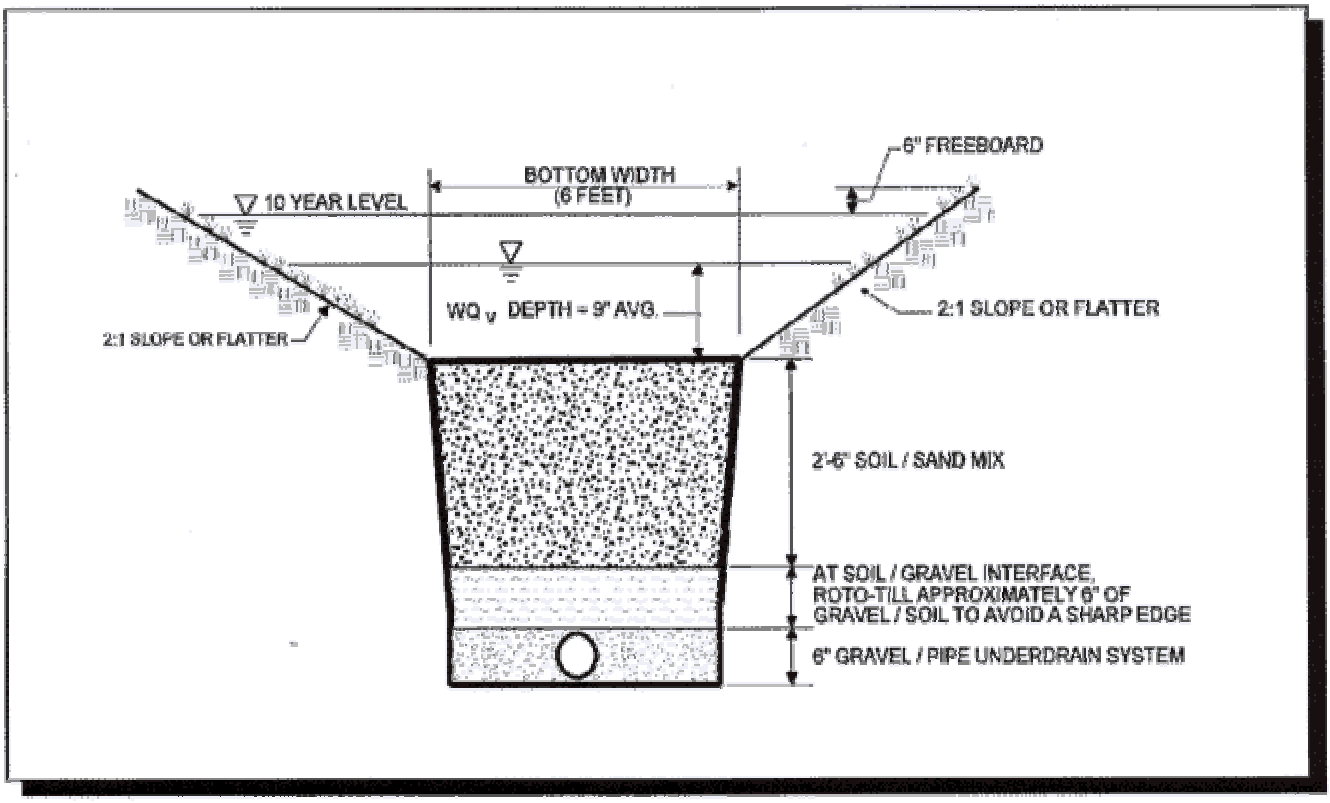
Step 1. Compute WQ_v

WQ_v previously determined to be 6,752 cubic feet. See Table 2.

Step 2. Compute RE_v

This dry swale design will have an "open bottom" (i.e., no impermeable liner (Figure 2)) to allow ground water recharge. 100% of the recharge volume (1,688 cubic feet) is contained within the WQ_v.

Figure 2. Trapezoidal Dry Swale Section



Step 3. Pretreatment:

Provide pretreatment in two shallow forebays at the head of the swales. As a target, provide 0.1"/impervious acre on the site. Divide this volume equally between the two forebays so that each provides 0.05"/ impervious acre of drainage.

(1.9 ac) (0.05) (1ft/12) (43560 sq ft/ac) = 344.9 cubic ft

This is a typical pretreatment requirement. See the Performance Criteria - Open Channels section for more information.

In addition, use a 2-0 pea gravel drain at the head of the swale.

This feature provides supplemental pretreatment.

Step 4. Identify swale dimensions:

Provide bottom width, depth, length, and slope necessary to store WQ_v with less than 18 of ponding.

Assume a trapezoidal channel with a maximum WQ_v depth of 18. Control for this swale will be a shallow concrete wall with a low flow orifice trash rack (Figure 4). Per the site plan, we have about 1225 of swale available if the swale is put in with two tails (Figure 3). The outlet control will be set at the existing invert minus three feet (22.0 - 3.0 = 19.0). The existing uphill invert for the northwest fork is 24.0 (length of 400). The invert for the northeast fork is 28.0 (length of 825).

Slope of northwest fork is (24 - 19)/400 = 0.0125 or 1.25%
Slope of northeast fork is (28 - 19)/825 = 0.0109 or 1.10%
minimum slope is 1.0 % [okay]

Minimum slope criteria are specified in the Performance Criteria - Open Channels section.

For a trapezoidal section with a bottom width of 6, a WQ_v average depth of 9, 2:1 side slopes, compute a cross sectional area of (6) (0.75) + (0.75) (1.5) = 5.63 ft^2 (see Figure 2).

(5.63 sq ft) (1,225) = 6,897 cubic feet [WQ_v of 6,752 ft^3, OK]

Step 5. Compute number of check dams (or similar structure) required to detain WQ_v (see Figure 5)

For northwest fork, 400 ft @ 1.25% slope, and max. depth at 18, place checkdams at: 1.5/0.0125 = 120'
Place at 120, 4 required
For northeast fork, 825 ft @ 1.10% slope, and 18" depth, place checkdams at 1.5/.011 = 136.4
Place at 140, 6 required

Figure 3. Dry Swale Site Plan

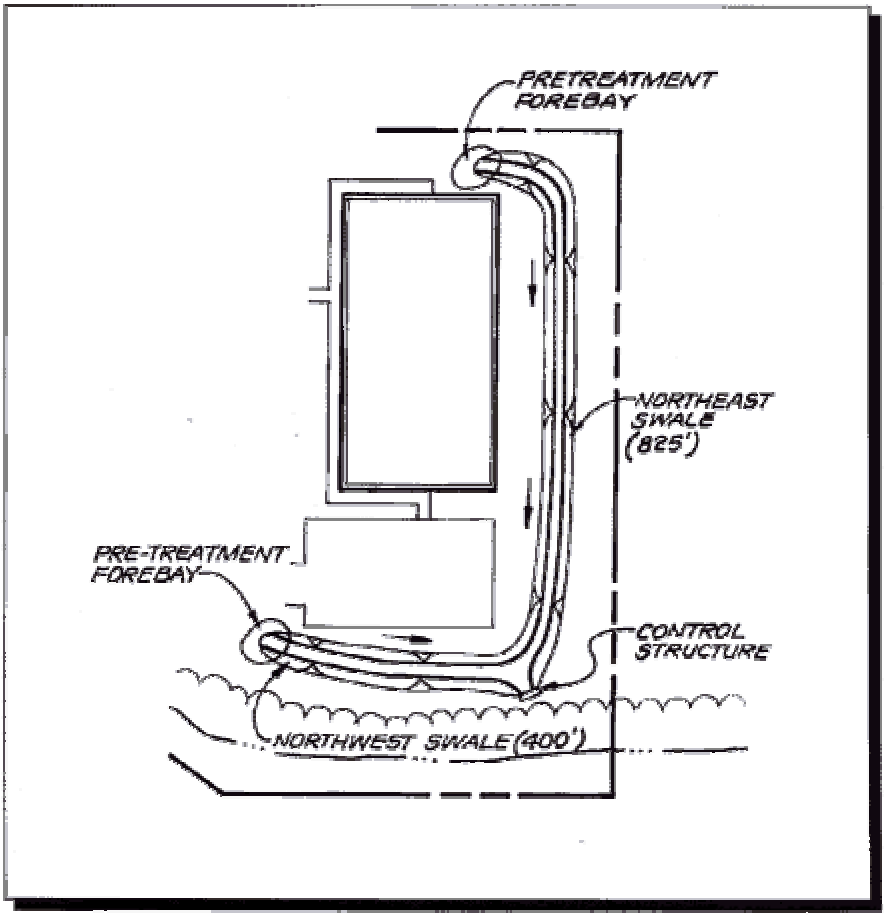
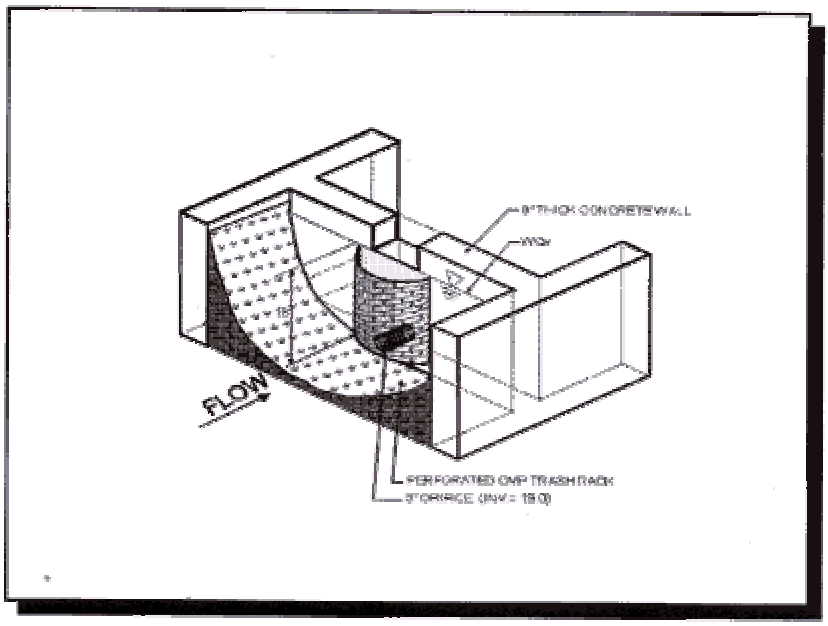


Figure 4. Control Structure at End of Swale



Step 6. Calculate draw-down time

In order to ensure that the swale will draw down within 24 hours, the planting soil will need to pass a max. rate of 1.5 in 24 hours (k = 1.5 per day). Provide 6 perforated underdrain pipe and gravel system below soil bed (see Figure 2).

Limiting drawdown time prevents risks of damp ground and potential mosquito infestation.

Step 7. Check 2-year velocity erosion potential:

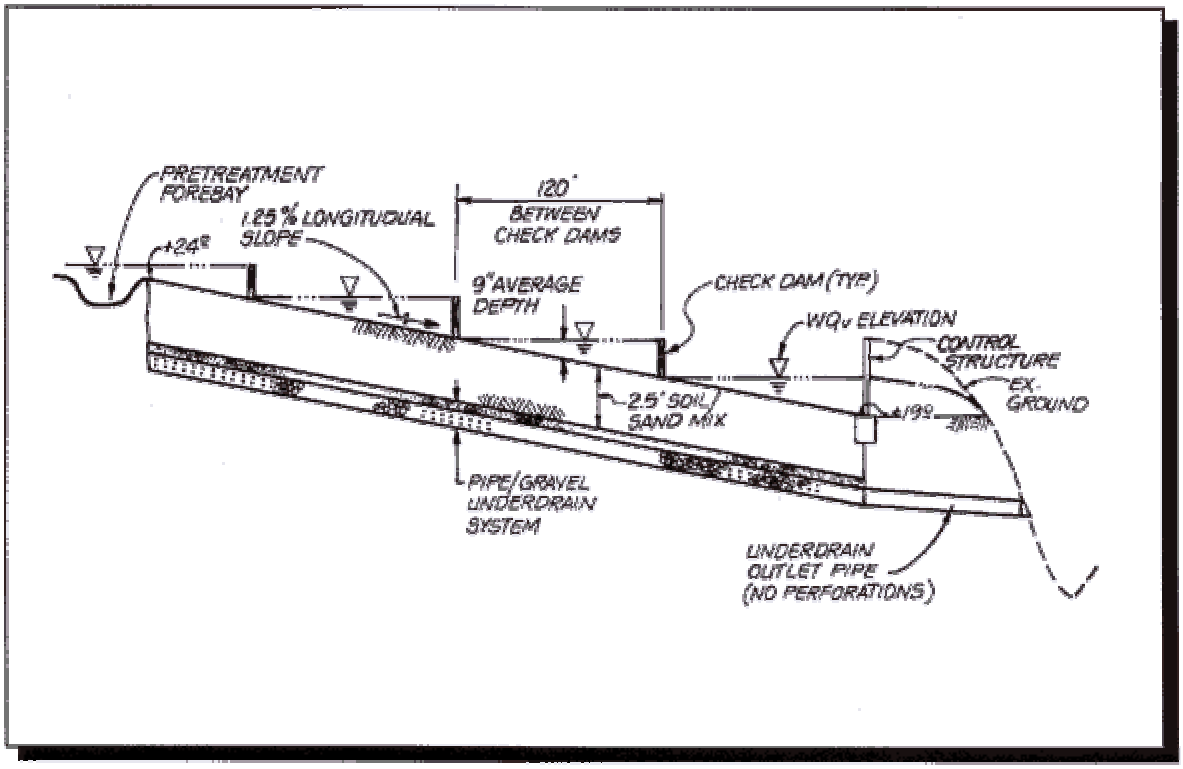
From hydrology information, the two-year flow is 7.11 cfs. Assume that 30% goes through northwestern swale (2.13 cfs) and 70% goes through the northeastern swale (4.98 cfs). Design for the larger amount (5.0 cfs). From separate computer analysis using a Manning's n value of 0.03 for tall grass and a slope of 1.1%, the two-year velocity will be 2.4 feet per second at a depth of 0.35 feet. [provide at least 0.35 above top of check dams, two year v less than 5 fps, okay]

Step 8. Check 10-year velocity erosion potential and freeboard:

From hydrology information, the ten-year flow is 14.0 cfs. Assume that 30% goes through northwestern swale (4.2 cfs) and 70% goes through the northeastern swale (9.8 cfs). Design for the larger amount (9.8 cfs). From separate computer analysis, with a slope of 1.1%, the ten-year velocity will be 2.9 feet-per-second at a depth of .45 feet, provide an additional .5 of freeboard above top of checkdams or 1.0' (total channel depth = 2.5).

Find 10 year overflow weir length required: (weir eq. $Q= CLH^{3/2)}$, where C = 3.1, Q_{10} = 9.8 cfs, H =1.0; L = 9.8 cfs/ $(3.1*1.0^{1.5})$ = 3.2' Use 4 ft

Figure 5. Profile of Northwest Fork Dry Swale



Step 9. Specify vegetation:

Use K-31 tall fescue in this case.

Step 10. Design low flow orifice at headwall (See Figure 4):

Design orifice to pass 6,897 cubic feet (calculated in Step 4) in 6 hours.
6,897 cubic feet/ (6 hours) (3600 sec/hour) = 0.319 cfs
use Orifice equation: $Q = CA(2gh)^{½}$
assume h = 1.5

$A = (0.319 \text{ cfs}) / [(0.6) ((2) (32.2 \text{ ft/s}^2) (1.5'))^{½}]$
A = 0.054 sq ft, dia = 0.206 feet or 2.47 use 3 minimum orifice size
Provide 3 v notch slot in each check dam

Construction Specifications for Stormwater Treatment Practices

The following section provides standards and specifications for the construction of stormwater treatment practices. Please click the links below to connect to specifications for:

- Ponds and Wetlands
- Infiltration Practices
- Filters and Open Channels

Construction Standards/Specifications for Ponds

These specifications are generally appropriate to all earthen ponds. Practitioners should consult their local dam safety division for the latest version of pond construction specifications. All references to ASTM and AASHTO specifications apply to the most recent version.

Site Preparation

Areas designated for borrow areas, embankment, and structural works shall be cleared, grubbed and stripped of topsoil. All trees, vegetation, roots and other objectionable material shall be removed. Channel banks and sharp breaks shall be sloped to no steeper than 1:1. All trees shall be cleared and grubbed within 15 feet of the toe of the embankment.

Areas to be covered by the reservoir will be cleared of all trees, brush, logs, fences, rubbish and other objectionable material unless otherwise designated on the plans. Trees, brush, and stumps shall be cut approximately level with the ground surface. For dry stormwater management ponds, a minimum of a 25-foot radius around the inlet structure shall be cleared.

All cleared and grubbed material shall be disposed of outside and below the limits of the dam and reservoir as directed by the owner or his representative. When specified, a sufficient quantity of topsoil will be stockpiled in a suitable location for use on the embankment and other designated areas.

Earth Fill

Material - The fill material shall be taken from approved designated borrow areas. It shall be free of roots, stumps, wood, rubbish, stones greater than 6", frozen or other objectionable materials. Fill material for the center of the embankment, and cut off trench shall conform to Unified Soil Classification GC, SC, CH, or CL and must have at least 30% passing the #200 sieve. Consideration may be given to the use of other materials in the embankment if designed by a geotechnical engineer. Such special designs must have construction supervised by a geotechnical engineer.

Materials used in the outer shell of the embankment must have the capability to support vegetation of the quality required to prevent erosion of the embankment.

Placement - Areas on which fill is to be placed shall be scarified prior to placement of fill. Fill materials shall be placed in maximum 8 inch thick (before compaction) layers which are to be continuous over the entire length of the fill. The most permeable borrow material shall be placed

in the downstream portions of the embankment. The principal spillway must be installed concurrently with fill placement and not excavated into the embankment.

Compaction - The movement of the hauling and spreading equipment over the fill shall be controlled so that the entire surface of each lift shall be traversed by not less than one tread track of heavy equipment or compaction shall be achieved by a minimum of four complete passes of a sheepsfoot, rubber tired or vibratory roller. Fill material shall contain sufficient moisture such that the required degree of compaction will be obtained with the equipment used. The fill material shall contain sufficient moisture so that if formed into a ball it will not crumble, yet not be so wet that water can be squeezed out.

When required by the reviewing agency the minimum required density shall not be less than 95% of maximum dry density with a moisture content within 2% of the optimum. Each layer of fill shall be compacted as necessary to obtain that density, and is to be certified by the Engineer at the time of construction. All compaction is to be determined by AASHTO Method T-99 (Standard Proctor).

Cut Off Trench - The cutoff trench shall be excavated into impervious material along or parallel to the centerline of the embankment as shown on the plans. The bottom width of the trench shall be governed by the equipment used for excavation, with the minimum width being four feet. The depth shall be at least four feet below existing grade or as shown on the plans. The side slopes of the trench shall be 1 to 1 or flatter. The backfill shall be compacted with construction equipment, rollers, or hand tampers to assure maximum density and minimum permeability.

Embankment Core - The core shall be parallel to the centerline of the embankment as shown on the plans. The top width of the core shall be a minimum of four feet. The height shall extend up to at least the 10 year water elevation or as shown on the plans. The side slopes shall be 1 to 1 or flatter. The core shall be compacted with construction equipment, rollers, or hand tampers to assure maximum density and minimum permeability. In addition, the core shall be placed concurrently with the outer shell of the embankment.

Structure Backfill

Backfill adjacent to pipes or structures shall be of the type and quality conforming to that specified for the adjoining fill material. The fill shall be placed in horizontal layers not to exceed four inches in thickness and compacted by hand tampers or other manually directed compaction equipment. The material needs to fill completely all spaces under and adjacent to the pipe. At no time during the backfilling operation shall driven equipment be allowed to operate closer than four feet, measured horizontally, to any part of a structure. Under no circumstances shall equipment be driven over any part of a concrete structure or pipe, unless there is a compacted fill of 24" or greater over the structure or pipe.

Structure backfill may be flowable fill meeting the requirements of the local Department of Transportation or State Materials Testing Agency. The mixture shall have a 100-200 psi; 28 day unconfined compressive strength. The flowable fill shall have a minimum pH of 4.0 and a minimum resistivity of 2,000 ohm-cm. Material shall be placed such that a minimum of 6" (measured perpendicular to the outside of the pipe) of flowable fill shall be under (bedding), over and, on the sides of the pipe. It only needs to extend up to the spring line for rigid conduits. Average slump of the fill shall be 7" to assure flowability of the material. Adequate measures shall be taken (sand bags, etc.) to prevent floating the pipe. When using flowable fill, all metal pipe shall be bituminous coated. Any adjoining soil fill shall be placed in horizontal layers not to exceed four inches in thickness and compacted by hand tampers or other manually directed compaction equipment. The material shall completely fill all voids adjacent to the flowable fill zone. At no time during the backfilling operation shall driven equipment be allowed to operate

closer than four feet, measured horizontally, to any part of a structure. Under no circumstances shall equipment be driven over any part of a structure or pipe unless there is a compacted fill of 24" or greater over the structure or pipe. Backfill material outside the structural backfill (flowable fill) zone shall be of the type and quality conforming to that specified for the core of the embankment or other embankment materials.

Pipe Conduits

All pipes shall be circular in cross section.

- ***Corrugated Metal Pipe*** - All of the following criteria shall apply for corrugated metal pipe:

Materials - (Polymer Coated steel pipe) - Steel pipes with polymeric coatings shall have a minimum coating thickness of 0.01 inch (10 mil) on both sides of the pipe. This pipe and its appurtenances shall conform to the requirements of AASHTO Specifications M-245 & M-246 with watertight coupling bands or flanges.

Materials - (Aluminum Coated Steel Pipe) - This pipe and its appurtenances shall conform to the requirements of AASHTO Specification M-274 with watertight coupling bands or flanges. Aluminum Coated Steel Pipe, when used with flowable fill or when soil and/or water conditions warrant the need for increased durability, shall be fully bituminous coated per requirements of AASHTO Specification M-190 Type A. Any aluminum coating damaged or otherwise removed shall be replaced with cold applied bituminous coating compound. Aluminum surfaces that are to be in contact with concrete shall be painted with one coat of zinc chromate primer or two coats of asphalt.

Materials - (Aluminum Pipe) - This pipe and its appurtenances shall conform to the requirements of AASHTO Specification M-196 or M-211 with watertight coupling bands or flanges. Aluminum Pipe, when used with flowable fill or when soil and/or water conditions warrant for increased durability, shall be fully bituminous coated per requirements of AASHTO Specification M-190 Type A. Aluminum surfaces that are to be in contact with concrete shall be painted with one coat of zinc chromate primer or two coats of asphalt. Hot dip galvanized bolts may be used for connections. The pH of the surrounding soils shall be between 4 and 9.

- Coupling bands, anti-seep collars, end sections, etc., must be composed of the same material and coatings as the pipe. Metals must be insulated from dissimilar materials with use of rubber or plastic insulating materials at least 24 mils in thickness.
- Connections - All connections with pipes must be completely watertight. The drain pipe or barrel connection to the riser shall be welded all around when the pipe and riser are metal. Anti-seep collars shall be connected to the pipe in such a manner as to be completely watertight. Dimple bands are not considered to be watertight.

  All connections shall use a rubber or neoprene gasket when joining pipe sections. The end of each pipe shall be re-rolled an adequate number of corrugations to accommodate the bandwidth. The following type connections are acceptable for pipes less than 24 inches in diameter: flanges on both ends of the pipe with a circular 3/8 inch closed cell neoprene gasket, pre-punched to the flange bolt circle, sandwiched between adjacent flanges; a 12-inch wide standard lap type band with 12-inch wide by 3/8-inch thick closed cell circular neoprene gasket; and a 12-inch wide hugger type band with o-ring gaskets having a minimum diameter of 1/2 inch greater than the corrugation depth. Pipes 24 inches in diameter and larger shall be connected by a 24 inch long annular corrugated band using a minimum of 4 (four) rods and lugs, 2 on each connecting pipe end. A 24-inch wide by 3/8-inch thick closed cell circular neoprene gasket will be installed with 12

inches on the end of each pipe. Flanged joints with 3/8 inch closed cell gaskets the full width of the flange is also acceptable.

Helically corrugated pipe shall have either continuously welded seams or have lock seams with internal caulking or a neoprene bead.

- Bedding - The pipe shall be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material shall be removed and replaced with suitable earth compacted to provide adequate support.
- Backfilling shall conform to "**Structure Backfill**".
- Other details (anti-seep collars, valves, etc.) shall be as shown on the drawings.

Reinforced Concrete Pipe - All of the following criteria shall apply for reinforced concrete pipe:

- Materials - Reinforced concrete pipe shall have bell and spigot joints with rubber gaskets and shall equal or exceed ASTM C-361.
- Bedding - Reinforced concrete pipe conduits shall be laid in a concrete bedding / cradle for their entire length. This bedding / cradle shall consist of high slump concrete placed under the pipe and up the sides of the pipe at least 50% of its outside diameter with a minimum thickness of 6 inches. Where a concrete cradle is not needed for structural reasons, flowable fill may be used as described in the "Structure Backfill" section of this standard. Gravel bedding is not permitted.
- Laying pipe - Bell and spigot pipe shall be placed with the bell end upstream. Joints shall be made in accordance with recommendations of the manufacturer of the material. After the joints are sealed for the entire line, the bedding shall be placed so that all spaces under the pipe are filled. Care shall be exercised to prevent any deviation from the original line and grade of the pipe. The first joint must be located within 4 feet from the riser.
- Backfilling shall conform to "**Structure Backfill**".
- Other details (anti-seep collars, valves, etc.) shall be as shown on the drawings.

Plastic Pipe - The following criteria shall apply for plastic pipe:

- Materials - PVC pipe shall be PVC-1120 or PVC-1220 conforming to ASTM D-1785 or ASTM D-2241. Corrugated High Density Polyethylene (HDPE) pipe, couplings and fittings shall conform to the following: 4" - 10" inch pipe shall meet the requirements of AASHTO M252 Type S, and 12" through 24" inch shall meet the requirements of AASHTO M294 Type S.
- Joints and connections to anti-seep collars shall be completely watertight.
- Bedding -The pipe shall be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material shall be removed and replaced with suitable earth compacted to provide adequate support.
- Backfilling shall conform to "**Structure Backfill**".
- Other details (anti-seep collars, valves, etc.) shall be as shown on the drawings.

Drainage Diaphragms - When a drainage diaphragm is used, a registered professional engineer will supervise the design and construction inspection.

Concrete

Concrete shall meet the requirements of your local Department of Transportation or State Highway Administration Standard Specifications for Construction and Materials.

Rock Riprap

Rock riprap shall meet the requirements of the local Department of Transportation or State Materials Testing Agency.

Geotextile shall be placed under all riprap and shall meet the requirements of the local Department of Transportation or State Materials Testing Agency.

Care of Water during Construction

All work on permanent structures shall be carried out in areas free from water. The Contractor shall construct and maintain all temporary dikes, levees, cofferdams, drainage channels, and stream diversions necessary to protect the areas to be occupied by the permanent works. The contractor shall also furnish, install, operate, and maintain all necessary pumping and other equipment required for removal of water from various parts of the work and for maintaining the excavations, foundation, and other parts of the work free from water as required or directed by the engineer for constructing each part of the work. After having served their purpose, all temporary protective works shall be removed or leveled and graded to the extent required to prevent obstruction in any degree whatsoever of the flow of water to the spillway or outlet works and so as not to interfere in any way with the operation or maintenance of the structure. Stream diversions shall be maintained until the full flow can be passed through the permanent works. The removal of water from the required excavation and the foundation shall be accomplished in a manner and to the extent that will maintain stability of the excavated slopes and bottom required excavations and will allow satisfactory performance of all construction operations. During the placing and compacting of material in required excavations, the water level at the locations being refilled shall be maintained below the bottom of the excavation at such locations which may require draining the water sumps from which the water shall be pumped.

Stabilization

All borrow areas shall be graded to provide proper drainage and left in a sightly condition. All exposed surfaces of the embankment, spillway, spoil and borrow areas, and berms shall be stabilized by seeding, liming, fertilizing and mulching in accordance with local Natural Resources Conservation Service Standards and Specifications.

Erosion and Sediment Control

Construction operations will be carried out in such a manner that erosion will be controlled and water and air pollution minimized. State and local laws concerning pollution abatement will be followed. Construction plans shall detail erosion and sediment control measures.

OPERATION AND MAINTENANCE

An operation and maintenance plan in accordance with Local or State Regulations will be prepared for all ponds. As a minimum, a dam inspection checklist shall be included as part of the operation and maintenance plan and performed at least annually. Written records of maintenance and major repairs needs to be retained in a file.

Supplemental Stormwater Pond and Wetland Specifications

1. It is preferred to use the same material in the embankment as is being installed for the core trench. If this is not possible because the appropriate material is not available, a dam core with a shell may be used. The cross-section of the stormwater facility should show the limits of the dam

core (up to the 10-year water surface elevation) as well as the acceptable materials for the shell. The shape of the dam core and the material to be used in the shell, should be provided by the geotechnical engineer.

2. If the compaction tests for the remainder of the site improvements is using Modified Proctor (AASHTO T-180), then to maintain consistency on-site, modified proctor may be used in lieu of standard proctor (AASHTO T-99). The minimum required density using the modified proctor test method shall be at least 92% of maximum dry density with a moisture content of ±2% of the optimum.

3. For all stormwater management facilities, a geotechnical engineer must be present to verify compaction in accordance with the selected test method. This information needs to be provided in a report to the design engineer, so that certification of the construction of the facility can be made.

4. A 4-inch layer of topsoil shall be placed on all disturbed areas of the dam embankment. Seeding, liming, fertilizing, mulching, etc. shall be in accordance with NRCS Soil Standards and Specifications or your local Standards and Specifications for Soil Erosion and Sediment Control. The purpose of the topsoil is to establish a good growth of grass which is not always possible with some of the materials that may be placed for the embankment fill.

5. Filter fabric placed beneath the rip-rap shall meet state or local department of transportation requirements for a Class "C" filter fabric. Some acceptable filter fabrics that meet the Class "C" criteria include:

- Mirafi 180-N
- Amoco 4552
- Webtec N07
- Geolon N70
- Carthage FX-70S

This is only a partial listing of available filter fabrics based on information provided by the manufacturers to the 1997 Specifier's Guide dated December 1996. It is the responsibility of the engineer to verify the adequacy of the material, as there are changes in the manufacturing process and the type of fabric used, which may affect the continued acceptance.

6. The design engineer and geotechnical engineer should make the determination that the settlement of the pond will not cause excessive joint extension. For further information on joint extension analysis, see NRCS Publication TR-18.

7. Fill placement shall not exceed a maximum of 8-inch. Each lift shall be continuous for the entire length of the embankment.

8. The embankment fill **shall not** be placed higher than the centerline of the principle spillway until after the principle spillway has been installed. If the embankment needs to be excavated to install the principle spillway, the side slope shall be no less than 2:1.

9. The side slopes of a cut to repair a dam, install a principle spillway for an excavated pond, or other repair work, shall be done on a slope of no less than 2:1.

Construction Specifications for Infiltration Practices

Infiltration Trench General Notes and Specifications

The infiltration trench systems may not receive run-off until the entire contributing drainage area to the infiltration system has received final stabilization.

1. Heavy equipment and traffic shall be restricted from traveling over the infiltration trench to minimize compaction of the soil.

2. Excavate the infiltration trench to the design dimensions. Excavated materials shall be placed away from the trench sides to enhance trench wall stability. Large tree roots must be trimmed flush with the trench sides in order to prevent fabric puncturing or tearing of the filter fabric during subsequent installation procedures. The side walls of the trench shall be roughened where sheared and sealed by heavy equipment.

3. A Class "C" geotextile or better shall interface between the trench side walls and between the stone reservoir and gravel filter layers. A partial list of non-woven filter fabrics that meet the Class "C" criteria is contained below. Any alternative filter fabric must be approved by the plan approval authority.

- Mirafi 180-N
- Amoco 4552
- WEBTEC N70
- GEOLON N70
- Carthage FX-80S

The width of the geotextile must include sufficient material to conform to trench perimeter irregularities and for a 6-inch minimum top overlap. The filter fabric shall be tucked under the sand layer on the bottom of the infiltration trench for a distance of 6 to 12 inches. Stones or other anchoring objects should be placed on the fabric at the edge of the trench to keep the trench open during windy periods. When overlaps are required between rolls, the uphill roll should lap a minimum of 2 feet over the downhill roll in order to provide a shingled effect.

4. A 6 inch sand filter layer may be placed on the bottom of the infiltration trench in lieu of filter fabric, and shall be compacted using plate compactors. The sand for the infiltration trench shall be washed and meet AASHTO Std. M-43, Size No. 9 or No. 10. Any alternative sand gradation must be approved by the Engineer or the reviewing agency.

5. The stone aggregate should be placed in lifts and compacted using plate compactors. A maximum loose lift thickness of 12 inches is recommended. The gravel (Rounded "Bank Run" gravel is preferred) for the infiltration trench shall be washed and meet one of the following AASHTO Std. M-43; Size No. 2 or No. 3.

6. Following the stone aggregate placement, the filter fabric shall be folded over the stone aggregate to form a 6-inch minimum longitudinal lap. The desired fill soil or stone aggregate shall be placed over the lap at sufficient intervals to maintain the lap during subsequent backfilling.

7. Care shall be exercised to prevent natural or fill soils from intermixing with the stone aggregate. All contaminated stone aggregate shall be removed and replaced with uncontaminated stone aggregate.

8. Voids can be created between the fabric and the excavation sides and shall be avoided. Removing boulders or other obstacles from the trench walls is one source of such voids, therefore, natural soils should be placed in these voids at the most convenient time during construction to ensure fabric conformity to the excavation sides.

9. Vertically excavated walls may be difficult to maintain in areas where soil moisture is high or where soft cohesive or cohesionless soils are predominate. These conditions may require laying back of the side slopes to maintain stability.

10. PVC distribution pipes shall be Schedule 40 and meet ASTM Std. D 1784. All fittings and perforations (1/2 inch in diameter) shall meet ASTM Std. D 2729. A perforated pipe shall be provided only within the infiltration trench and shall terminate 1 foot short of the infiltration trench wall. The end of the PVC pipe shall be capped.

11. The corrugated metal distribution pipes shall conform to AASHTO Std. M-36, and shall be aluminized in accordance with AASHTO Std. M-274. Coat aluminized pipe in contact with concrete with an inert compound capable of effecting isolation of the deleterious effect of the aluminum on the concrete. Perforated distribution pipe shall be provided only within the infiltration trench and shall terminate 1 foot short of the infiltration trench wall. An aluminized metal plate shall be welded to the end of the pipe.

12. The observation well is to consist of 6-inch diameter PVC Schedule 40 pipe (ASTM Std. D 1784) with a cap set 6 inches above ground level and is to be located near the longitudinal center of the infiltration trench. Preferably the observation well will not be located in vehicular traffic areas. The pipe shall have a plastic collar with ribs to prevent rotation when removing cap. The screw top lid shall be a "Panella" type cleanout with a locking mechanism or special bolt to discourage vandalism. A perforated (1/2 inch in diameter) PVC Schedule 40 pipe shall be provided and placed vertically within the gravel portion of the infiltration trench and a cap provided at the bottom of the pipe. The bottom of the cap shall rest on the infiltration trench bottom.

13. If a distribution structure with a wet well is used, a 4-inch PVC drain pipe shall be provided at opposite ends of the infiltration trench distribution structure. Two (2) cubic feet of porous backfill meeting AASHTO Std. M-43 Size No. 57 shall be provided at each drain.

14. If a distribution structure is used, the manhole cover shall be bolted to the frame.

NOTE: PVC pipe with a wall thickness classification of SDR-35 meeting ASTM standard D3034 is an acceptable substitution for PVC Schedule 40 pipe.

Infiltration Basins Notes and Specifications

The sequence of various phases of basin construction shall be coordinated with the overall project construction schedule. A program should schedule rough excavation of the basin with the rough grading phase of the project to permit use of the material as fill in earthwork areas. The partially excavated basin, however, **cannot** serve as a sedimentation basin.

Specifications for basin construction should state: (1) the earliest point in progress when storm drainage may be directed to the basin, and (2) the means by which this delay in use is to be accomplished. Due to the wide variety of conditions encountered among projects, each should be separately evaluated in order to postpone use as long as is reasonably possible.

1. Initial basin excavation should be carried to within 1 foot of the final elevation of the basin floor. Final excavation to the finished grade should be deferred until all disturbed areas on the watershed have been stabilized or protected. The final phase excavation should remove all accumulated sediment. Relatively light tracked equipment is recommended for this operation to avoid compaction of the basin floor. After the final grading is completed, the basin provide a well-aerated, highly porous surface texture.

2. Infiltration basins may be lined with a 6- to 12-inch layer of filter material such as coarse sand (AASHTO Std. M-43, Sizes 9 or 10) to help prevent the buildup of impervious deposits on the soil surface. The filter layer can be replaced or cleaned when it becomes clogged. When a 6-inch layer of coarse organic material is specified for discing (such as hulls, leaves, stems, etc.) or spading into the basin floor to increase the permeability of the soils, the basin floor should be soaked or inundated for a brief period, then allowed to dry subsequent to this operation. This induces the organic material to decay rapidly, loosening the upper soil layer.

3. Establishing dense vegetation on the basin side slopes and floor is recommended. A dense vegetative stand will not only prevent erosion and sloughing, but will also provide a natural means of maintaining relatively high infiltration rates. Erosion protection of inflow points to the basin shall also be provided.

4. Selection of suitable vegetative materials for the side slope and all other areas to be stabilized with vegetation and application of required lime, fertilizer, etc. shall be done in accordance with the NRCS Standards and Specifications or your local Standards and Specifications for Soil Erosion and Sediment Control.

5. Grasses of the fescue family are recommended for seeding primarily due to their adaptability to dry sandy soils, drought resistance, hardiness, and ability to withstand brief inundations. The use of fescues will also permit long intervals between mowings. This is important due to the relatively steep slopes which make mowing difficult. Mowing twice a year, once in June and again in September, is generally satisfactory. Refertilization with 10-6-4 ratio fertilizer at a rate of 500 lb per acre (11 lb per 1000 sq ft) may be required the second year after seeding.

Construction Specifications for Sand Filters, Bioretention and Open Channels

Sand Filter Specifications

Material Specifications for Sand Filters

The allowable materials for sand filter construction are detailed in Table 1.

Sand Filter Testing Specifications

Underground sand filters, facilities within sensitive groundwater aquifers, and filters designed to serve urban hot spots are to be tested for water tightness prior to placement of filter layers. Entrances and exits should be plugged and the system completely filled with water to demonstrate water tightness.

All overflow weirs, multiple orifices and flow distribution slots to be field-tested as to verify adequate distribution of flows.

Sand Filter Construction Specifications

Provide sufficient maintenance access; 12-foot-wide road with legally recorded easement. Vegetated access slopes to be a maximum of 10%; gravel slopes to 15%; paved slopes to 25%.

Absolutely no runoff is to enter the filter until all contributing drainage areas have been stabilized.

Surface of filter bed to be *completely level*.

All sand filters should be clearly delineated with signs so that they may be located when maintenance is due.

Surface sand filters shall be planted with appropriate grasses as specified in your local NRCS Standards and Specifications guidance.

"Pocket" sand filters (and residential bioretention facilities treating areas larger than an acre) shall be sized with an "ornamental" stone window covering approximately 10% of the filter area. This surface shall be 2" to 5" size stone on top of a pea gravel layer (3/4 inch stone) approximately 4 to 6" of pea gravel.

Specifications Pertaining to Underground Sand Filters

Provide manhole and/or grates to all underground and below grade structures. Manholes shall be in compliance with standard specifications for each jurisdiction but diameters should be 30" minimum (to comply with OSHA confined space requirements) but not too heavy to lift. Aluminum and steel louvered doors are also acceptable. Ten inch long (minimum) manhole steps (12" o.c.) shall be cast in place or drilled and mortared into the wall below each manhole. A 5' minimum height clearance (from the top of the sand layer to the bottom of the slab) is required for all permanent underground structures. Lift rings are to be supplied to remove/replace top slabs. Manholes may need to be grated to allow for proper ventilation; if required, place manholes *away* from areas of heavy pedestrian traffic.

Underground sand filters shall be constructed with a dewatering gate valve located just above the top of the filter bed should the bed clog.

Underground sand beds shall be protected from trash accumulation by a wide mesh geotextile screen to be placed on the surface of the sand bed; screen is to be rolled up, removed, cleaned and re-installed during maintenance operations.

Table 1. Sand Material Specifications

Parameter	Specification	Size	Notes
Sand	clean AASHTO M-6 or ASTM C-33 concrete sand	0.02" to 0.04"	Sand substitutions such as Diabase and Graystone #10 are not acceptable. No calcium carbonated or dolomitic sand substitutions are acceptable. "rock dust" cannot be substituted for sand.
Peat	ash content: < 15% pH range: 5.2 to 4.9 loose bulk density 0.12 to 0.15 g/cc	n/a	The material must be Reed-Sedge Hemic Peat, shredded, uncompacted, uniform, and clean.
Underdrain Gravel	AASHTO M-43	0.25" to 0.75"	
Geotextile Fabric (if required)	ASTM D-751 (puncture strength - 125 lb.) ASTM D-1117 (Mullen Burst Strength - 400 psi) ASTM D-1682 (Tensile Strength - 300 lb.)	0.08" thick equivalent opening size of #80 sieve	Must maintain 125 gpm per sq. ft. flow rate. Note: a 4" pea gravel layer may be substituted for geotextiles meant to "separate" sand filter layers.
Impermeable Liner (if required)	ASTM D 751 (thickness) ASTM D 412 (tensile strength 1,100 lb., elongation 200%) ASTM D 624 (Tear resistance - 150 lb./in) ASTM D 471 (water adsorption: +8 to -2% mass)	30 mil thickness	Liner to be ultraviolet resistant. A geotextile fabric should be used to protect the liner from puncture.
Underdrain Piping	ASTM D-1785 or AASHTO M-278	6" rigid schedule 40	3/8" perf. @ 6" on center, 4 holes per row: minimum of 3" of gravel

		PVC	over pipes; not necessary underneath pipes
Concrete (Cast-in-place)	See local DOT Standards and Specs. f'c = 3500 psi, normal weight, air-entrained; re-inforcing to meet ASTM 615-60	n/a	on-site testing of poured-in-place concrete required: 28 day strength and slump test; all concrete design (cast-in-place or pre-cast) *not using previously approved State or local standards* requires design drawings sealed and approved by a licensed professional structural engineer.
Concrete (pre-cast)	per pre-cast manufacturer	n/a	SEE ABOVE NOTE
non-rebar steel	ASTM A-36	n/a	structural steel to be hot-dipped galvanized ASTM A123

Specifications for Bioretention

Material Specifications

The allowable materials to be used in bioretention area are detailed in Table 2.

Planting Soil

The soil shall be a uniform mix, free of stones, stumps, roots or other similar objects larger than two inches. No other materials or substances shall be mixed or dumped within the bioretention area that may be harmful to plant growth, or prove a hindrance to the planting or maintenance operations. The planting soil shall be free of noxious weeds.

The planting soil shall be tested and shall meet the following criteria:

pH range 5.2 - 7.0
organic matter 1.5 - 4%
magnesium 35 lb./ac
phosphorus P_2O_5 75 lb./ac
potassium K_2O 85 lb./ac
soluble salts not to exceed 500 ppm

All bioretention areas shall have a minimum of one test. Each test shall consist of both the standard soil test for pH, phosphorus, and potassium and additional tests of organic matter, and soluble salts. A textural analysis is required from the site stockpiled topsoil. If topsoil is imported, then a texture analysis shall be performed for each location where the top soil was excavated.

Since different labs calibrate their testing equipment differently, all testing results shall come from the same testing facility.

Should the pH fall out of the acceptable range, it may be modified (higher) with lime or (lower) with iron sulfate plus sulfur.

Compaction

It is very important to minimize compaction of both the base of the bioretention area and the required backfill. When possible, use excavation hoes to remove original soil. If bioretention areas is excavated using a loader, the contractor should use wide track or marsh track equipment, or light equipment with turf type tires. Use of equipment with narrow tracks or narrow tires, rubber tires with large lugs, or high pressure tires will cause excessive compaction resulting in reduced infiltration rates and storage volumes and is not acceptable. Compaction will significantly contribute to design failure.

Compaction can be alleviated at the base of the bioretention facility by using a primary tilling operation such as a chisel plow, ripper, or subsoiler. These tilling operations are to refracture the soil profile through the 12 inch compaction zone. Substitute methods must be approved by the engineer. Rototillers typically do not till deep enough to reduce the effects of compaction from heavy equipment.

Rototill 2 to 3 inches of sand into the base of the bioretention facility before back filling the required sand layer. Pump any ponded water before preparing (rototilling) base.

When back filling the topsoil over the sand layer, first place 3 to 4 inches of topsoil over the sand, then rototill the sand/topsoil to create a gradation zone. Backfill the remainder of the topsoil to final grade.

When back filling the bioretention facility, place soil in lifts 12" or greater. Do not use heavy equipment within the bioretention basin. Heavy equipment can be used around the perimeter of the basin to supply soils and sand. Grade bioretention materials with light equipment such as a compact loader or a dozer/loader with marsh tracks.

Plant Material

Recommended plant material for bioretention areas can be found in your local NRCS Standards and Specifications guidance.

Plant Installation

Mulch around individual plants only. Shredded hardwood mulch is the only accepted mulch. Pine mulch and wood chips will float and move to the perimeter of the bioretention area during a storm event and are not acceptable. Shredded mulch must be well aged (6 to 12 months) for acceptance.

The plant root ball should be planted so $1/8^{th}$ of the ball is above final grade surface.

Root stock of the plant material shall be kept moist during transport and on-site storage. The diameter of the planting pit shall be at least six inches larger than the diameter of the planting ball. Set and maintain the plant straight during the entire planting process. Thoroughly water ground bed cover after installation.

Trees shall be braced using 2" X 2" stakes only as necessary and for the first growing season only. Stakes are to be equally spaced on the outside of the tree ball.

Grasses and legume seed shall be tilled into the soil to a depth of at least one inch. Grass and legume plugs shall be planted following the non-grass ground cover planting specifications.

The topsoil specifications provide enough organic material to adequately supply nutrients from natural cycling. The primary function of the bioretention structure is to improve water quality. Adding fertilizers defeats, or at a minimum, impedes this goal. Only add fertilizer if wood chips or mulch is used to amend the soil. Rototill urea fertilizer at a rate of 2 pounds per 1000 square feet.

Underdrains

Under drains to be placed on a 3'-0" wide section of filter cloth. Pipe is placed next, followed by the gravel bedding. The ends of under drain pipes not terminating in an observation well shall be capped.

The main collector pipe for underdrain systems shall be constructed at a minimum slope of 0.5%. Observation wells and/or clean-out pipes must be provided (one minimum per every 1000 square feet of surface area).

Miscellaneous

The bioretention facility may not be constructed until all contributing drainage area has been stabilized.

Table 2. Materials Specifications for Bioretention

Parameter	Specification	Size	Notes
Plantings	see your local NRCS Standards and Specifications guidance.	n/a	plantings are site-specific
Planting Soil [4' deep]	sand 35 - 60% silt 30 - 55% clay 10 - 25%	n/a	USDA soil types loamy sand, sandy loam or loam
Mulch	shredded hardwood		aged 6 months, minimum
pea gravel diaphragm and curtain drain	pea gravel: ASTM D 448 ornamental stone: washed cobbles	pea gravel: No. 6 stone: 2" to 5"	
Geotextile	Class "C" apparent opening size (ASTM-D-4751) grab tensile strength (ASTM-D-4632) burst strength (ASTM-D-4833)	n/a	for use as necessary beneath underdrains only
underdrain gravel	AASHTO M-43	0.25" to 0.75"	
underdrain piping	ASTM D 1785 or AASHTO M-278	6" rigid schedule 40 PVC	3/8" perf. @ 6" on center, 4 holes per row; minimum of 3" of gravel over pipes; not necessary underneath pipes

poured in place concrete (if required)	See local DOT Standards and Specs.; f'c = 3500 lb. @ 28 days, normal weight, air-entrained; re-inforcing to meet ASTM 615-60	n/a	on-site testing of poured-in-place concrete required: 28 day strength and slump test; all concrete design (cast-in-place or pre-cast) *not using previously approved State or local standards* requires design drawings sealed and approved by a licensed professional structural engineer.
sand [1' deep]	AASHTO M-6 or ASTM C-33	0.02" to 0.04"	Sand substitutions such as Diabase and Graystone #10 are not acceptable. No calcium carbonated or dolomitic sand substitutions are acceptable. No "rock dust" can be used for sand.

Specifications for Open Channels and Filter Strips

Material Specifications

The recommended construction materials for open channels and filter strips are detailed in Table 3.

Dry Swales

- Roto-till soil/gravel interface approximately 6" to avoid a sharp soil/gravel interface.
- Permeable soil mixture (20" to 30" deep) should meet the bioretention "planting" soil specifications.
- Check dams, if required, shall be placed as specified.
- System to have 6" of freeboard, minimum.
- Side slopes to be 3:1 minimum; (4:1 or greater preferred).
- No gravel or perforated pipe is to be placed under driveways.
- Bottom of facility to be above the seasonably high water table.
- Seed with flood/drought resistant grasses; see your local NRCS Standards and Specifications guidance.
- Longitudinal slope to be 1 to 2%, maximum [up to 5% with check dams].
- Bottom width to be 6' maximum to avoid braiding; larger widths may be used if proper berming is supplied. Width to be 2' minimum.

Wet Swales

Follow above information for dry swales, with the following exceptions: the seasonally high water table may inundate the swale; but not above the design bottom of the channel [NOTE: if the water table is stable within the channel; the WQv storage may start at this point]

Excavate into undisturbed soils; do not use an underdrain system.

Filter Strips

Construct pea gravel diaphragms 12" wide, minimum, and 24" deep minimum.

Pervious berms to be a sand/gravel mix (35-60% sand, 30-55% silt, and 10-25% gravel). Berms to have overflow weirs with 6 inch minimum head.

Slope range to be 2% minimum to 6% maximum.

Plant Selection

Use recommended grass species for establishing permanent ground cover; see your local NRCS Standards and Specifications guidance.

Table 3. Open Vegetated Swale and Filter Strip Materials Specifications

Parameter	Specification	Size	Notes
Dry swale soil	USCS; ML, SM, SC	n/a	soil with a higher percent organic content is preferred
Dry Swale sand	ASTM C-33 fine aggregate concrete sand	0.02" to 0.04"	
Check Dam (pressure treated)	AWPA Standard C6	6" by 6" or 8" by 8"	*do not* coat with creosote; embed at least 3' into side slopes
Check Dam (natural wood)	Black Locust, Red Mulberry, Cedars, Catalpa, White Oak, Chestnut Oak, Black Walnut	6" to 12" diameter; notch as necessary	*do not* use the following, as these species have a predisposition towards rot: Ash, Beech, Birch, Elm, Hackberry, hemlock, Hickories, Maples, Red and Black Oak, Pines, Poplar, Spruce, Sweetgum, Willow
Filter Strip sand/gravel pervious berm	sand: per dry swale sand gravel; AASHTO M-43	sand: 0.02" to 0.04" gravel: ½" to 1"	mix with approximately 25% loan soil to support grass cover crop; see Bioretention planting soil notes for more detail.
pea gravel diaphragm and curtain drain	ASTM D 448	varies (No. 6) or (1/8" to 3/8")	use clean bank-run gravel
under drain gravel	AASHTO M-43	0.25" to 0.75"	
under drain	ASTM D -1785 or AASHTO M-278	6" rigid Schedule 40 PVC	3/8" perf. @ 6" o.c.; 4 holes per row
Geotextile	See local DOT Standards and Specs	n/a	
rip rap	per local DOT criteria	size per local DOT requirements based on 10-year design flows	

Construction and Maintenance Checklists

In order to effectively ensure proper implementation long-term functioning of stormwater practices, local government staff need to consistently evaluate them. This section presents checklists for both the construction and long-term operation and maintenance of stormwater management practices for the following practices:

- Pond Construction
- Pond Operation, Management and Maintenance Inspection
- Infiltration Trench Operation, Management and Maintenance Inspection
- Infiltration Basin Operation, Management and Maintenance Inspection
- Bioretention Construction
- Sand Filter Construction
- Bioretention Operation, Management and Maintenance Inspection
- Sand Filter Operation, Management and Maintenance Inspection
- Open Channel Construction
- Open Channel Operation, Management and Maintenance Inspection