Chapter 12

STORAGE FACILITIES

ODOT ROADWAY DRAINAGE MANUAL

November 2014

Chapter 12
STORAGE FACILITIES

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12.1 INTRODUCTION

12.1.1 Overview

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this type of design may result in major drainage and flooding problems downstream. The engineering community is now more conscious of the quality of the environment and the impact that uncontrolled increases in runoff can have on our customers. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and, often, the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities, to large lakes and reservoirs.

This chapter provides policy, criteria, design guidelines, sizing and routing procedures from the AASHTO *Drainage Manual* (1), Chapter 14 “Storage Facilities” for detention/retention storage basins that are associated with highway drainage. For more detailed information, see HEC-22 (2) and HDS-2 (3). The USEPA provides guidelines for these facilities on their NPDES website that provides a National Menu of Stormwater Best Management Practices. Chapter 15 “Permits” provides detailed discussion of *Clean Water Act* requirements. Storage and flood routing associated with culverts is addressed in the Chapter 9 “Culverts.” The hydraulics designer should have determined the appropriate design frequencies, performed hydrologic analysis using the procedures of Chapter 7 “Hydrology” and have the preliminary roadway geometrics before using the design procedures in this chapter. This chapter provides the following:

- symbols and definitions (Section 12.2),
- policy (Section 12.3),
- design criteria (Section 12.4),
- design procedure (Section 12.5),
- dry pond (detention basin) (Section 12.6),
- wet pond (retention basin) (Section 12.7),
- infiltration and filtration controls (Section 12.8),
- land-locked retention (Section 12.9),
- storage water budget (Section 12.10), and
- construction and maintenance considerations (Section 12.11).

12.1.2 Location Considerations

The location of storage facilities is very important because it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits, and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Thus, it is important for the hydraulics designer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the drainage basin. Effective stormwater management must be coordinated with a regional basin-wide plan. The state should encourage and participate in such planning.

12.1.3 Detention and Retention

Stormwater storage facilities are often referred to as either detention or retention facilities. For this chapter, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. Recharge or infiltration basins are a special type of detention basin designed to drain into the groundwater table. Retention facilities are designed to contain a permanent pool of water. Because most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities. Storage facilities may be small in terms of storage capacity and dam height where serving a single outfall from a watershed of a few acres, or they may be larger facilities serving as regional stormwater management control. The benefits of storage facilities can be divided into two major control categories of quality and quantity.

12.1.3.1 Quality

The control of stormwater quality using storage facilities offers the following potential benefits:

- decreased downstream channel erosion,
- controlled sediment deposition, and
- improved water quality through:
    - stormwater filtration, and
    - capture of the first flush with detention for 24 hours or more.

12.1.3.2 Quantity

Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- prevention or reduction of peak runoff rate increases caused by urban development,
- mitigation of downstream drainage capacity problems,
- recharge of groundwater resources,
- reduction or elimination of the need for downstream outfall improvements, and
- maintenance of historic low-flow rates by controlled discharge from storage.

12.1.4 ODOT Design Practice

ODOT practice is to design and analyze storage facilities using reservoir routing calculations. Routing calculations required to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations, the hydraulics designer can use many available reservoir routing software applications (see Chapter 16 “Hydraulic Software”). ODOT manages stormwater quantity by limiting peak runoff rates if required to match one or more of the following values:

- historic rates for specific design conditions (e.g., post-development peak equals pre-development peak for a particular frequency of occurrence);
- non-hazardous discharge capacity of the downstream drainage system; or
- a specified value for allowable discharge set by a regulatory jurisdiction.

For a watershed without an adequate outfall, the total volume of runoff is critical and storage facilities are used to store the increases in volume and to control discharge rates. In rare cases, reservoir routing may be used to minimize a drainage structure size where there is considerable natural storage immediately upstream.

12.1.5 NATIONAL DAM SAFETY PROGRAM

The National Dam Safety Program (NDSP), which was formally established by the *Water Resources and Development Act of 1996*, includes: grant assistance to the states, dam safety research and dam safety training. National responsibility for the promotion and coordination of dam safety lies with FEMA. Responsibility for administration of the provisions of the NDSP is given to the states. Rules and regulations relating to applicable dams are promulgated by the responsible state agency.

Under the Federal regulations, a dam is an artificial barrier that does or may impound water that is 25 ft or greater in height or has a maximum storage volume of 50 acre-ft or more (4). The Oklahoma Dam Safety Program is administered by the Oklahoma Water Resources Board (OWRB). Information is available at their website: http://www.owrb.ok.gov/hazard/dam/dams.php. Their fact sheet indicates that they use the Federal definition for a small dam: “Construction application may not be necessary if the dam will be less than 25 feet in height above the stream bed or if the lake impounded by the dam will less than 50 acre-feet of water; however, approval is required regardless of size if there are houses or habitable structures located below the dam.”

12.2 SYMBOLS AND DEFINITIONS

To provide consistency within this chapter and throughout this manual, the symbols in Figure 12.2-A will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined in the text or equations.

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
C	Weir coefficient	—
D	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
F	Infiltration rate	in/h
G	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Inflow rate	cfs
L	Length	ft
O	Outflow rate	cfs
R	Ratio of width to length of basin at base	
Q_i	Peak inflow rate	cfs
Q_o	Peak outflow rate	cfs
S_a	Surface area	acre(s)
T	Routing time period	
t_i	Time base on hydrograph	s
T_i	Duration of basin inflow	h
t_p	Time to peak	h
V	Storage volume	ft^3, acre-ft
W	Width of basin	ft
Z	Side slope factor	ft

Figure 12.2-A — SYMBOLS AND DEFINITIONS

12.3 POLICY

To protect ODOT drainage facilities (e.g., bridges, culverts, ditches) from being overloaded by extra runoff from new adjacent developments, ODOT requests that these new adjacent developments should provide storage to reduce developed peak runoff rates to pre-developed (existing) peak runoff rates for 6 major frequencies, including the 2-year, 5-year, 10-year, 25-year, 50-year and 100-year return period rainfall. The developers should use detention facility(ies) to achieve this request (see also Section 3.4, Chapter 3 ”Policy”).

12.4 DESIGN CRITERIA

12.4.1 General Criteria

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout a drainage system. Storage may be developed in depressed areas in parking lots, road embankments and freeway interchanges, parks, and other recreational areas and small lakes, ponds and depressions within developments. The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis (e.g., 100-year flood). The design criteria for storage facilities should include:

- release rate (Section 12.4.2),
- storage volume (Section 12.4.3),
- grading and depth requirements (Section 12.4.4),
- outlet works (Section 12.4.5), and
- location (Section 12.4.6).

Construction and maintenance considerations are discussed in Section 12.11.

The general design criteria of the design of the detention facility(ies) is summarized as follows :

- Use the NRCS (formerly SCS) Type II or Type III (depending on the location of the site) 24-hour duration rainfall distribution in computing the peak discharges and in hydrographs routing (see SCS TR-55). The use of the hypothetical rainfall distribution hydrograph, the SCS simplified hydrograph and the Modified Rational Method triangular hydrograph are NOT accepted.

- The Curve Number (CN) of the land should be based on the local soil as shown in the NRCS Soil Report of the related County and also on the proposed land use change.

- Use six major frequencies in the routing, including the 2-year, 5-year, 10-year, 25-year, 50-year and 100-year return period rainfall.

- The use of the U.S. Corps of Engineers (USACE) programs HEC-1 or HEC-HMS in the routing is recommended. Other commercial programs (e.g., Bentley PondPack, Eagle Points) can be used as long as all the ODOT Design Criteria are satisfied.

- The 24-hour (or the 1-day, whichever is greater) duration rainfall precipitation data of the site should be obtained from the 1999 USGS WRI No. 99-4232 instead of from the 1966 U.S. Weather Bureau TP-40 and NOAA HYDRO-35.

- If the size of the proposed pond is greater than 20 acres, the hydraulics designer need also to conform with the requirements from the Oklahoma Water Resources Board (OWRB) for detention ponds.

- The size of the principal spillway structure of the proposed detention pond should not be greater than the downstream structure size.

12.4.2 Release Rate

The release rate of control structures should:

- be equal or less than the pre-developed peak runoff rates (for the 2, 5, 10, 25, 50 and 100-year storm),
- provide for emergency overflow of the 100-year discharge, and
- provide for multi-stage control if required to control both runoff from the 2-year and 100-year storms.

12.4.3 Storage Volume

Storage volume should be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates for the design storm. To control the design storm, the basin storage should be equal to the area between the preconstruction and post construction hydrographs. Routing calculations should be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions should be restored before completion of the project. Detention basins should be designed to drain within the average period between storm events or no later than 72 hours. Additional detention storage may be needed to provide for water quality to satisfy environmental regulations.

12.4.4 Grading and Depth Requirements

General grading and depth criteria for storage facilities are discussed in Section 12.4.4.1. Specific criteria related to detention are discussed in Section 12.4.4.2 and to retention facilities in Section 12.4.4.3.

12.4.4.1 General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments should be less than 25 ft in height and should have side slopes no steeper than 1V:3H (follow Federal/state dam safety regulations). Side slopes should be benched at intervals of 5 ft. Riprap-protected embankments should be no steeper than 1V:2H. Geotechnical slope stability analysis is recommended for embankments greater than 10 ft in height and is mandatory for embankment slopes steeper than those given above.

A minimum freeboard of 1 ft above the 100-year storm high-water elevation should be provided for impoundment depths of less than 25 ft. Impoundment depths greater than 25 ft or volumes greater than 50 acre-ft are subject to the requirements of the National Dam Safety Program (see Section 14.4).

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements and required freeboard. Aesthetically pleasing features are also important in urbanizing areas.

12.4.4.2 Detention

Areas above the normal high-water elevations of storage facilities should be sloped to allow drainage toward the facility and to prevent standing water. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2% bottom slope is recommended. A low-flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing water conditions.

12.4.4.3 Retention

The maximum depth of permanent storage facilities will be determined by site conditions, design constraints and environmental requirements. In general, if the facility provides a permanent pool of water, consider providing a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions). A depth of 5 ft to 10 ft is generally reasonable unless fishery requirements dictate otherwise. Aeration may be required in permanent pools to prevent anaerobic conditions. Where aquatic habitat is required, contact the appropriate wildlife experts for site-specific criteria relating to such elements as depth, habitat and bottom and shore geometry.

12.4.5 Outlet Works

Outlet works selected for storage facilities should be able to accomplish the design functions of the facility. Typically, outlet works include a principal spillway and an emergency spillway. Outlet works can take the form of combinations of drop inlets, pipes, weirs and orifices. Slotted riser pipes are discouraged because of clogging problems. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency spillway.

12.4.6 Location

In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin, it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. For storage facilities that affect critical facilities downstream, channel routing calculations should proceed downstream to a confluence point where the drainage area being analyzed represents 10% of the total drainage area. At this point, the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph should be assessed for detrimental effects on downstream areas.

12.5 DESIGN PROCEDURE

12.5.1 Data Needs

The following data will be needed to complete storage design and routing calculations:

- inflow hydrograph for all selected design storms;
- stage-storage curve for proposed storage facility (see Figure 12.5-A for an example). For large storage volumes (e.g., for reservoirs), use acre-ft; otherwise, use cubic feet; and
- stage-discharge curve for all outlet control structures (see Figure 12.5-B for an example).

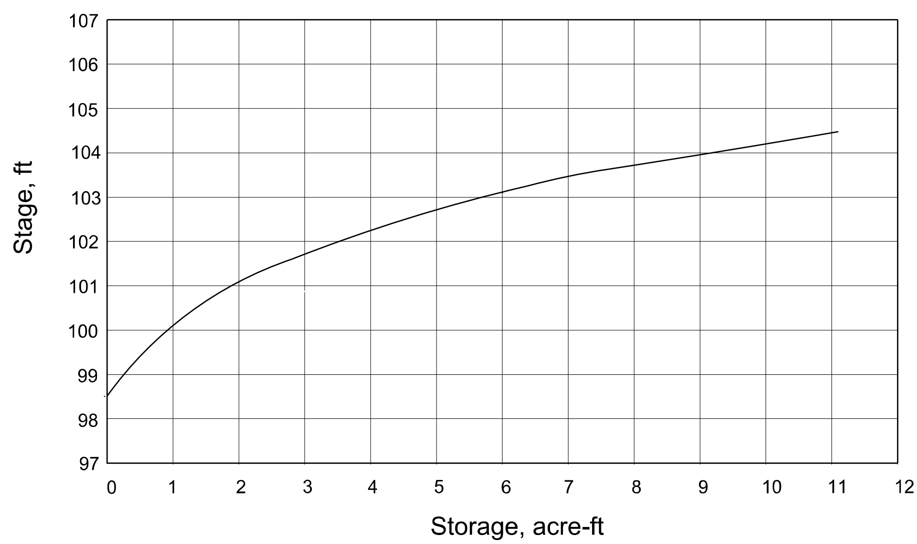


Figure 12.5-A — EXAMPLE STAGE-STORAGE CURVE

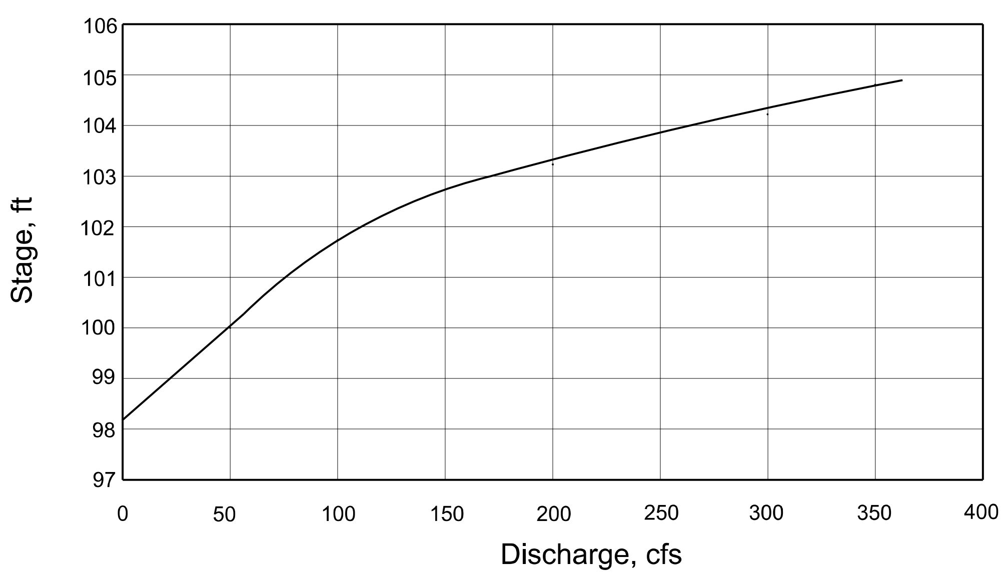


Figure 12.5-B — EXAMPLE STAGE-DISCHARGE CURVE

Using this data, a design procedure is used to route the inflow hydrograph through the storage facility to establish an outflow hydrograph (see Figure 12.5-C). If the desired outflow results are not achieved, basin and outlet geometry are varied to yield new stage-storage and stage-discharge curves and the routing procedure is redone until the desired outflow hydrograph is achieved.

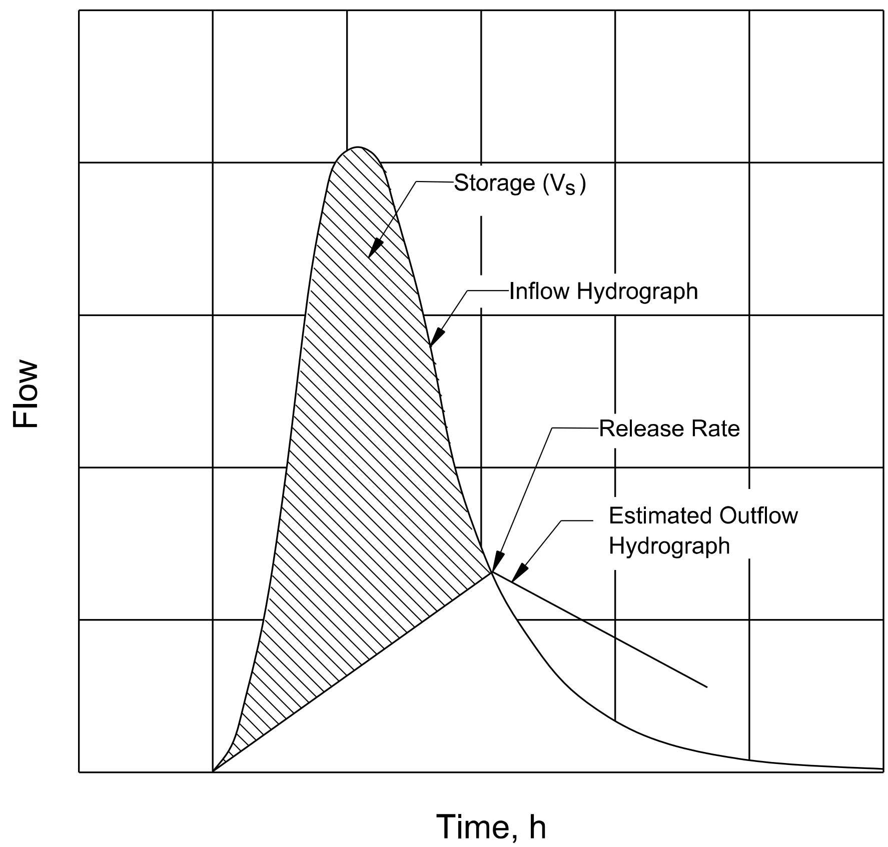


Figure 12.5-C — INFLOW AND OUTFLOW HYDROGRAPHS FROM A STREAM REACH

12.5.2 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve is usually developed using a topographic map and one of the following formulas—the average-end area, frustum of a pyramid or prismoidal formulas. The storage basin may be underground if right-of-way is restricted. Storage basins above ground are often irregular in shape to blend well with the surrounding terrain and to improve aesthetics. Storage basins underground are typically uniform in cross section.

The average-end area formula is usually preferred as the method to be used on non-geometric areas. The average-end area formula is expressed as:

$$V_{1,2} = \left[\left(A_1 + A_2 \right) / 2 \right] d \qquad \text{Equation 12.5(1)}$$

Where:

$V_{1,2}$ = storage volume, ft^3, between elevations 1 and 2
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, ft^2
d = change in elevation between points 1 and 2, ft

The frustum of a pyramid is expressed as:

$$V = d\left[A_1 + (A_1A_2)^{0.5} + A_2\right]/3 \qquad \text{Equation 12.5(2)}$$

Where:

V = volume of frustum of a pyramid, ft^3
d = change in elevation between points 1 and 2, ft
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, ft^2

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)\,ZD^2 + (4/3)\,Z^2D^3 \qquad \text{Equation 12.5(3)}$$

Where:

V = volume of trapezoidal basin, ft^3
L = length of basin at base, ft
W = width of basin at base, ft
D = depth of basin, ft
Z = side slope factor, ratio of horizontal to vertical (V:H)
r = ratio of width to length of basin at the base

Estimating the trial dimensions of a basin for a given basin storage volume can be accomplished by rearranging Equation 12.5(2) as shown in Equation 12.5(3):

$$L = \left\{-ZD\,(r + 1) + \left[(ZD)^2 (r + 1)^2 - 5.33\,(ZD)^2 r + ((4r\,V)/D)\right]^{0.5}\right\} / 2r \qquad \text{Equation 12.5(4)}$$

12.5.3 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two outlets, principal and emergency. The principal outlet is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency outlet or spillway.

A typical outlet structure may include culverts, weirs, orifices or a combination of all three. Because culverts are typically used as the primary outlet, the procedures presented in Chapter 9 “Culverts” can be used to develop stage-discharge data. When analyzing release rates, the tailwater influence of the culvert should be considered on the emergency control structure (orifice and/or weirs) to determine the effective head on each opening. Avoid slotted riser pipe outlet facilities due to debris-plugging potential. For design information on weirs and orifices, see HEC-22 (2).

The emergency outlet or spillway should be sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. When the outlet and spillway are designed, consider the potential threat to downstream life and property.

The stage-discharge curve should reflect the discharge characteristics of both the principal and emergency outlets. Develop a composite stage-discharge curve, which combines the discharge rating curve for all components of the outlet control structure. Figure 12.5-D illustrates an example composite stage-discharge curve.

12.5.4 Routing Procedure

A commonly used method for routing an inflow hydrograph through a detention pond is the Storage Indication or Modified Puls Method. This method begins with the continuity equation that states that the inflow minus the outflow equals the change in storage (I – O = S). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by Equation 12.5(5). This relationship is illustrated graphically in Figure 12.5-E:

$$\frac{\Delta S}{\Delta t} = \frac{I_2 + I_2}{2} - \frac{O_1 + O_2}{2}$$

Equation 12.5(5)

Where:

ΔS = change in storage, ft^3
Δt = time interval, min
I = inflow, ft^3
O = outflow, ft^3

In Equation 12.5(5), subscript 1 refers to the beginning and subscript 2 refers to the end of the time interval.

Equation 12.5(5) can be rearranged so that all known values are on the left side of the equation and all unknown values are located on the right-hand side of the equation, as shown in Equation 12.5(6). Now the equation with two unknowns, S_2 and O_2, can be solved with one equation:

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right)$$

Equation 12.5(6)

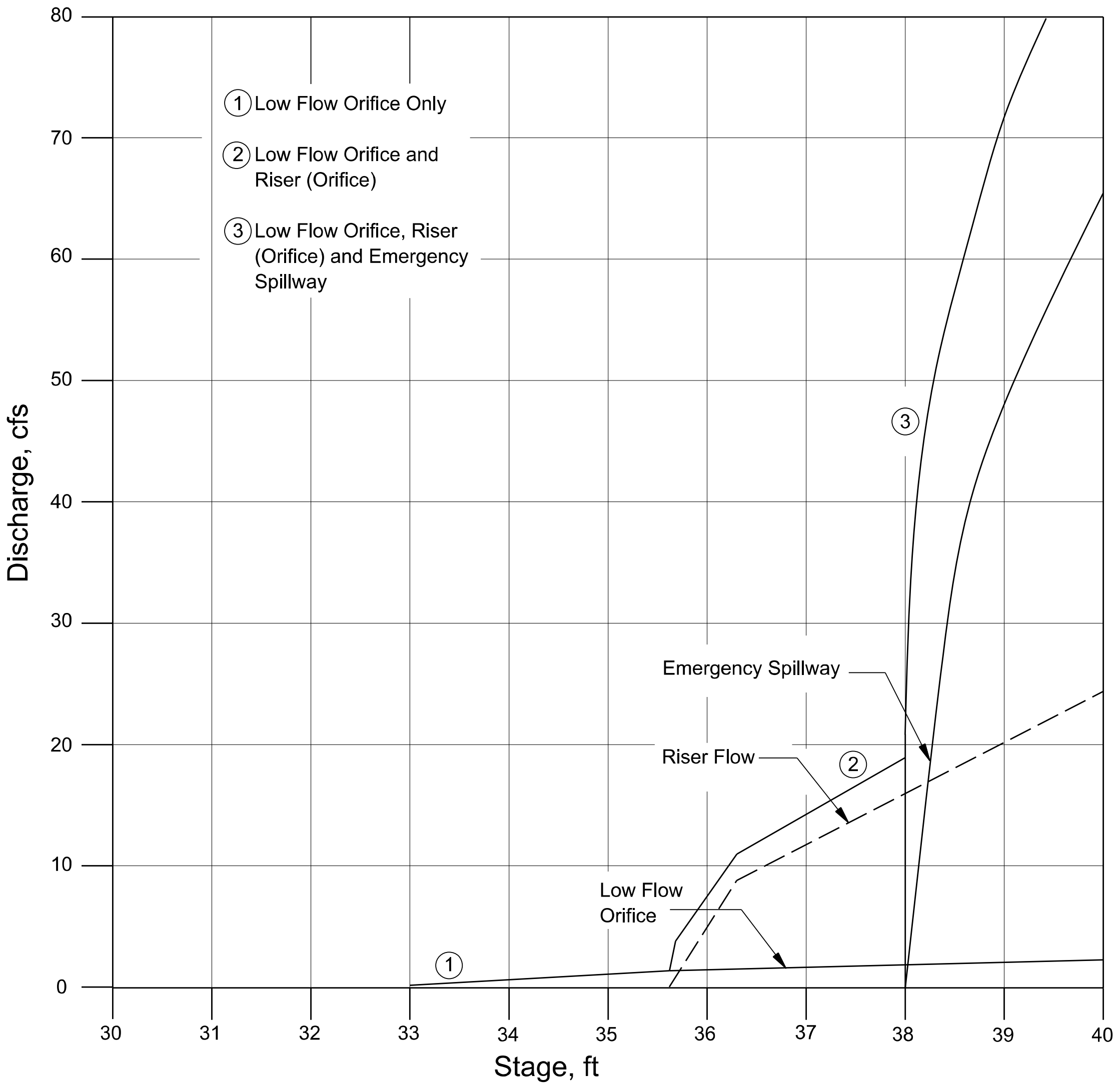


Figure 12.5-D — COMBINED STAGE-DISCHARGE RELATIONSHIP

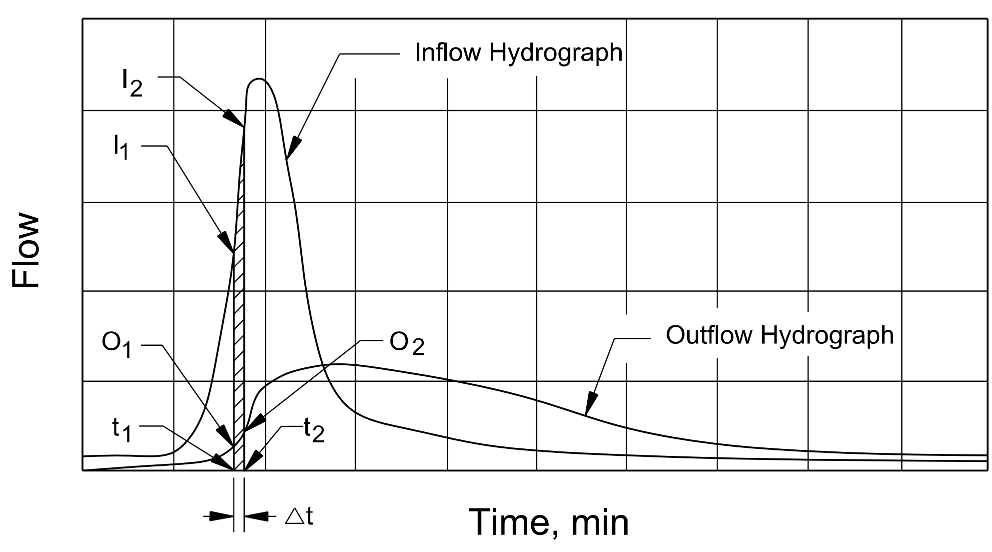


Figure 12.5-E — ROUTING HYDROGRAPH SCHEMATIC

12.5.5 Preliminary Detention Calculations

12.5.5.1 Storage Volume

While the use of the Modified Rational Method triangular hydrograph is not accepted in the final design of the detention facility (see sub-section 12.4.1), the hydraulics designer can use it in the preliminary estimate of the required storage volume of the detention facility.

A preliminary estimate of the storage volume required for peak-flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 12.5-F.

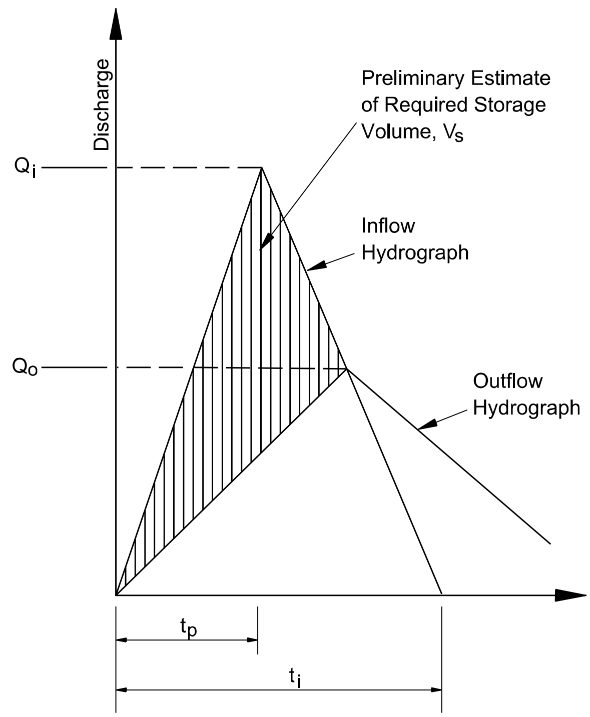


Figure 12.5-F — TRIANGULAR SHAPED HYDROGRAPHS
(Preliminary Storage Volume Estimate)

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5T_i(Q_i - Q_o) \quad \text{Equation 12.5(7)}$$

Where:

V_S	=	storage volume estimate, ft^3
Q_i	=	peak inflow rate, cfs
Q_o	=	peak outflow rate, cfs
t_i	=	duration of basin inflow, sec
t_p	=	time to peak, sec

Any consistent units may be used for Equation 12.5(7).

12.5.5.2 Preliminary Basin Dimensions

Use the following procedure to develop the preliminary basin dimensions:

- Plot the control structure location on a contour map.
- Select a desired depth of ponding for the design storm.
- Divide the estimated storage volume needed by the desired depth to obtain the surface area required of the reservoir.

Based on site conditions and contours, estimate the geometric shape(s) required to provide the estimated reservoir surface area.

12.5.6 Step by Step Routing Procedure

A general procedure for using the above data in the design of storage facilities is presented below:

Step 1. Compute inflow hydrograph for runoff from the six storms (2, 5, 10, 25, 50 and 100) using the procedures outlined in Chapter 7 "Hydrology." Both pre- and post-development hydrographs are required for the design storms.

Step 2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Section 12.5.5).

Step 3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. Use the maximum storage requirement calculated from Step 2.

Step 4. Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. Size the outlet structure to convey the peak outflow rate from the desired outflow hydrograph.

Step 5. Perform routing calculations using inflow hydrographs from Step 1 and Equation 12.5(6) to check the preliminary design. If the routed post-development peak discharges from the design storm exceed the pre-development peak discharge or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

Step 6. Consider emergency overflow from runoff due to larger storm and established freeboard requirements.

Step 7. Assess the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems.

Step 8. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results. HEC 22 (2), Example 8.9 uses the USGS nationwide urban hydrograph to compute runoff for existing and proposed conditions. Example 8.3 provides step-by-step hand calculations.

12.5.7 Example Problem

The procedure in Section 12.4.6 is illustrated in the following example, which uses the FHWA Hydraulic Toolbox (see Chapter 16 "Hydraulics Software").

Given: The 25-year hydrograph is provided.

Find: The size of culvert and storage pond that is needed so that this 25-year storm will not overtop a 10-ft high highway crossing of the floodplain.

Step 1. Compute inflow hydrograph; see Figure 12.5-G.

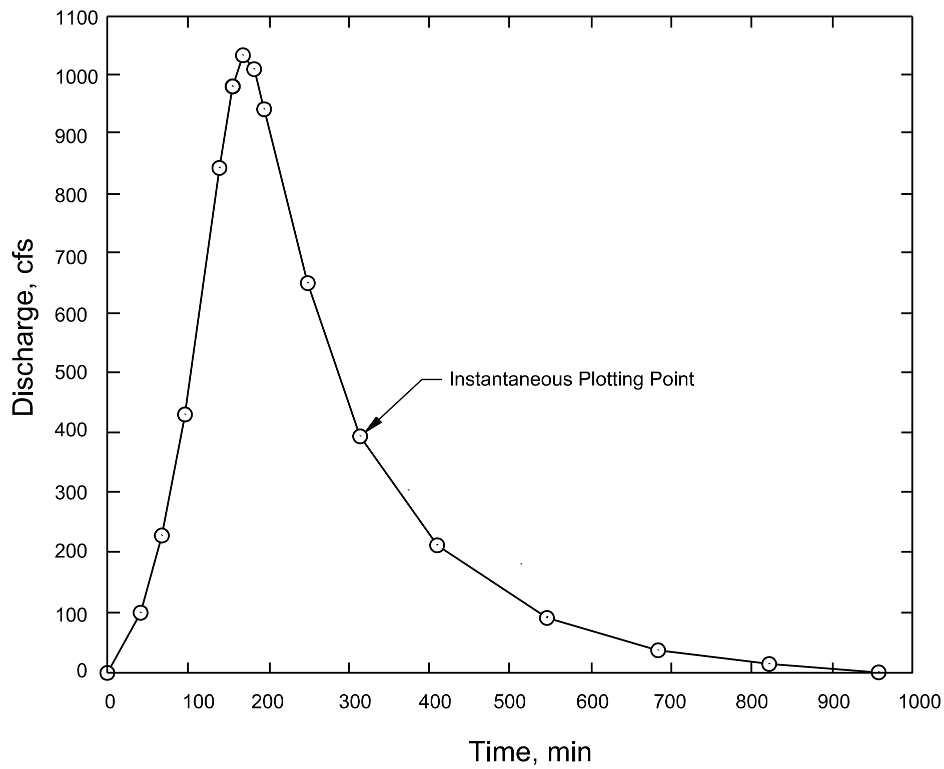


Figure 12.5-G — 25-YEAR HYDROGRAPH

The points are provided in the following table:

Time (min)	Discharge (cfs)
0	0
41	96
68	223
96	429
137	841
151	979
164	1030
178	1010
192	944
246	652
315	395
411	206
548	89
685	34
821	8.6
958	0

Step 2. Perform preliminary calculations to evaluate detention storage.

Step 3. Determine the physical dimensions necessary to hold the estimated volume. Steps 2 through 4 will be combined because the Hydraulic Toolbox has an option for calculating a trapezoidal basin with known geometry.

Step 4. Size the outlet structure. Use a concrete culvert with a headwall. Use invert data for the 100-ft long culvert on a 2% slope. For the performance curve, increase the discharge until overtopping occurs. Save the output to an Excel spreadsheet and paste the performance curve into the outlet columns.

Step 5. Perform routing calculations:

Basin			Culvert Diameter	Maximum Storage	Pond Elevation
Depth	Width	Length			
10 ft	500 ft	2500 ft	7 ft	271.7 acre-ft	108.90 ft
		2300 ft	7 ft	264.3 acre-ft	109.40 ft
		2100 ft	7 ft	256.0 acre-ft	109.90 ft

The diameter of the culvert and the size of the basin can be varied until the target elevation is achieved. In this example, a 7-ft diameter culvert with a trapezoidal-shaped basin that is 500-ft wide by 2100-ft long will store 256 acre-ft of water at the peak elevation of 109.90 ft. The storage and discharge table for the final alternative is provided below and routed hydrograph is shown in Figure 12.5-H.

Water Elevation (ft)	**Storage (acre-ft)**	**Outflow Discharge (cfs)**
100.00	0.0000	0.00
101.11	27.0045	23.05
102.22	54.4545	46.10
103.33	82.3535	92.35
104.44	110.7048	145.41
105.56	139.5118	208.27
106.67	168.7780	275.70
107.78	198.5067	340.21
108.89	228.7014	397.90
110.00	259.3654	449.54

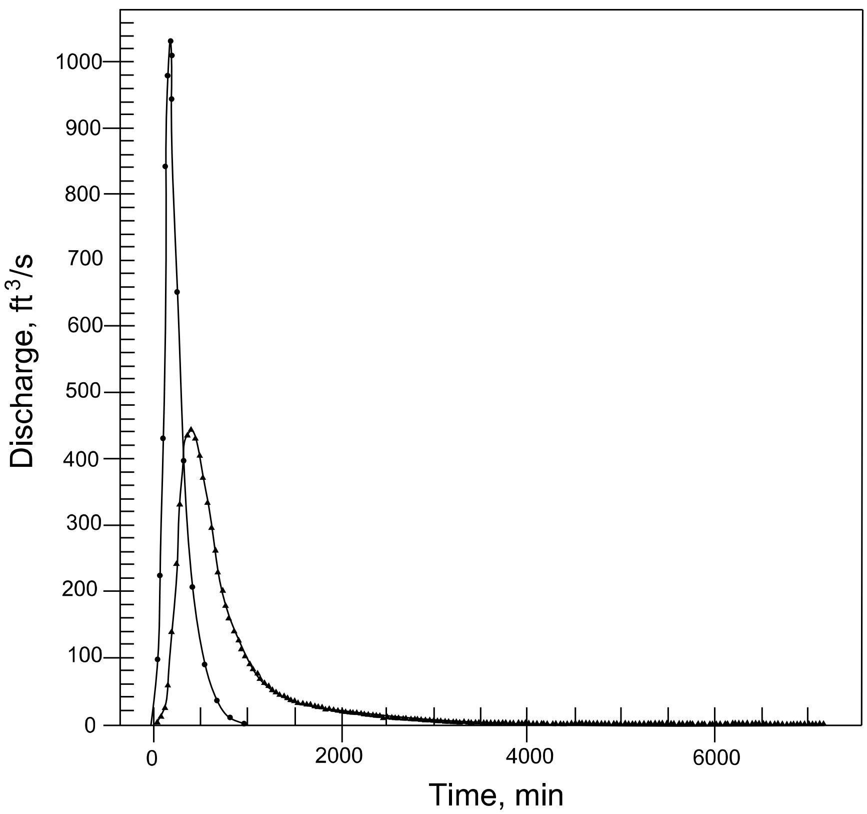


Figure 12.5-H — INFLOW AND OUTFLOW HYDROGRAPHS FOR STORAGE EXAMPLE

12.6 DRY POND (Detention Basin)

12.6.1 Introduction

Detention basins are depressed areas that store runoff during wet weather and are dry the remainder of the time. Detention basins are common because of their comparatively low cost, few design limitations, ability to serve large and small watersheds and potential to be incorporated into other uses (e.g., recreational areas).

The dry pond, as discussed in this section, is not designed for runoff control (see previous sections above), but is designed to satisfy EPA's water quality requirements.

Figure 12.6-A provides an overview of dry pond considerations that are discussed in the following sections.

Quality	Detain WQV for 30 hours (minimum 3 in orifice).
Quantity	Control design flow and maintain non-erosive velocity.
Shape	Length-to-width ratio > 3; wedge shaped (wider at outlet).
Maintenance	Inspect once a year, preferably during wet weather, mow as required (at least twice a year); remove sediment (every 5 years to 10 years).
Other Considerations	Side slopes provide easy maintenance access (1V:3H); 2% bottom slope to prevent ponding; sediment forebay to reduce maintenance; safety requirements (depth and perimeter ledges).
Pollutant Removal	Moderate.

Figure 12.6-A — OVERVIEW OF CONSIDERATIONS FOR A DRY POND

12.6.2 Quality

To design the basin for stormwater quality control (EPA's requirements), the water quality volume (WQV) must be routed through the basin. The WQV should be 0.05 ft multiplied by the project area. The WQV should be detained and released over a period of 30 h. The following equation may be used to obtain the average outflow from the pond:

$$Q_{avg} = (WQV) / T \qquad \text{Equation 12.6(1)}$$

Where:

Q_{avg} = average outflow from the basin, cfs
WQV = water quality volume, ft^3
T = detention time, s

Knowing the average outflow, one can use an appropriate orifice equation to determine the size of the outlet. However, the minimum size of the outlet should be 3 in; this is to prevent the outlet from clogging, and it may lead to a detention time less than 30 h. Figure 12.6-B shows the WQV and average outflows for different drainage areas.

Drainage Area (acres)	WQV (ft^3)	Average 30-h Outflow (cfs)	Drainage Area (acres)	WQV (ft^3)	Average 30-h Outflow (cfs)
2.5	4590	0.04	27.2	50,493	0.47
4.9	9181	0.08	30.0	56,496	0.51
7.4	13,771	0.13	32.1	59,674	0.55
9.9	18,361	0.17	34.6	64,264	0.60
12.4	22,952	0.21	37.1	68,855	0.64
14.8	27,542	0.25	39.5	73,445	0.68
17.3	32,132	0.30	42.0	78,035	0.72
19.8	36,722	0.34	44.5	82,625	0.77
22.2	41,313	0.38	46.9	87,216	0.81
24.7	45,903	0.42	49.4	91,806	0.85

Figure 12.6-B — VOLUMES AND FLOWS ACCORDING TO SIZE OF AREA

12.6.3 Quantity

For quantity purposes, design the pond to reduce the post-construction peak flow from the design storm to the preconstruction level. If the 100-year storm is not used in design, the basin should be able to pass a 100-year storm safely. To control these storms, the basin's storage should be equal to the area between the pre- and post-construction hydrographs.

After a storage volume has been determined for each event, a 2- and a 10-year. The design storm should be routed through the facility to ensure that the peak flows from the post-construction watershed are not greater than the corresponding preconstruction peak flows. Finally, a 100-year storm should be routed through the facility to ensure that the embankment will not be damaged or fail during the passage of that storm. Several outlets may be used to control the different storms—one for a 2-year storm; one for a 10-year storm; and an emergency spillway to control anything larger, including the 100-year storm (see Figure 12.6-C). To improve the efficiency of the outlet, it may be necessary to include an antivortex device (see Figure 12.6-D).

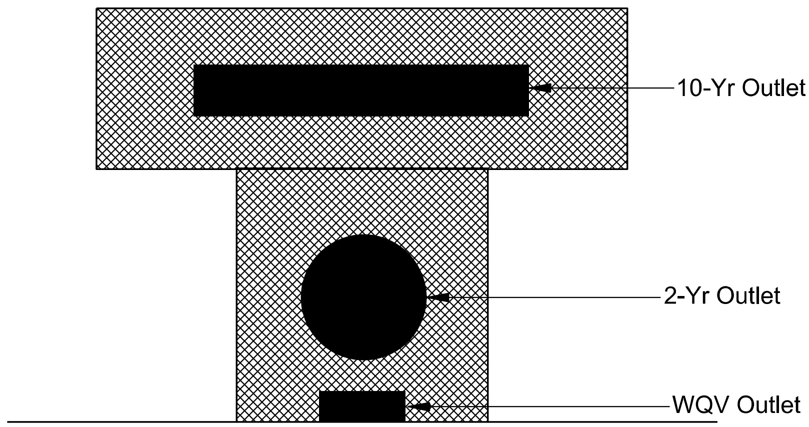
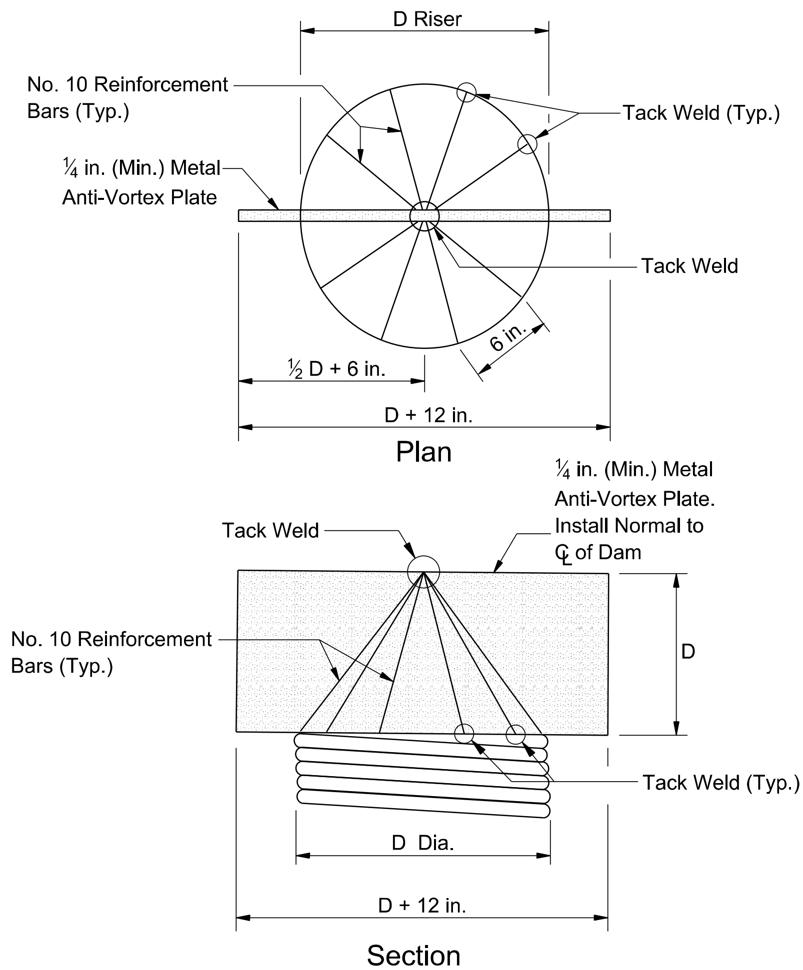


Figure 12.6-C — CONCRETE RISER

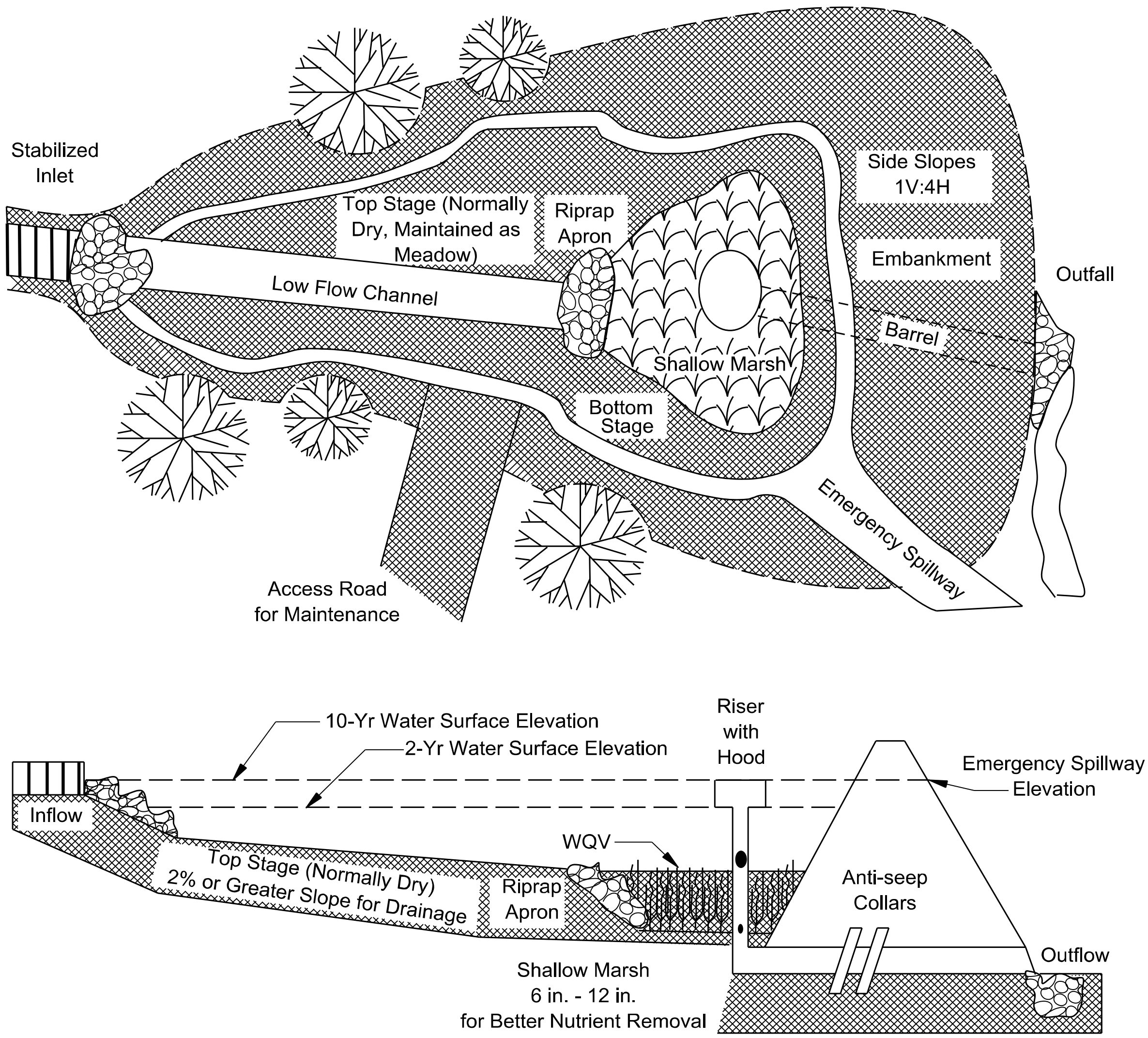


Source: FHWA-TS-79-225 (5)

Figure 12.6-D — ANTIVORTEX PLATE AND TRASH RACK

12.6.4 Quantity and Quality Combined

Several design variations may be considered to enhance the capabilities of the facility. One consideration is to shape the basin to improve its pollutant-removal capabilities. The length-to-width ratio should be at least three and a wedge-shaped basin (wider at the outlet) can also improve pollutant removal. The inlet, outlet and side slopes should be stabilized with riprap or vegetation, or both, to prevent erosion. The basin floor should also be vegetated to stabilize the soil and increase biological uptake. The pond floor should be sloped no less than 2% to prevent the ponding of stormwater. The side slopes should allow for easy maintenance access. A marsh or wetland can be established on the pond floor to increase biological uptake. A sediment forebay (a small sediment trap at the inlet of the basin, either a depressed area or a shallow area with a very flat slope where sediment is easily deposited) can be used to catch the sediment before it fills the basin (see Figure 12.6-E).



Source: Schueler (6)

Figure 12.6-E — DRY POND

The basin can also have two stages—one sized to hold the smaller storms and a second, which is rarely inundated and can be used for other purposes, sized to help store the larger storms. Safety considerations include reducing the chance of drowning by the use of warning signs, reducing the maximum depth or including benching and mild slopes, or both, to prevent people from falling in and to facilitate their escape from the basin.

12.6.5 Outlets

Outlets for dry basins can be designed in a wide variety of configurations. Most outlets use modified inlet boxes or risers of concrete or corrugated metal. These structures can be designed to control different storms through the use of several orifices or pipes; for example, a small inlet to control the WQV, an orifice to control a 2-year storm and a larger orifice to control a 10-year storm. This larger flow is usually controlled by stormwater flowing in through the top of the structure. If risers are used an antivortex design may be necessary for flow entering the top of the pipe. Larger flows are usually handled by an emergency spillway. Because the WQV outlet must be small to detain the WQV long enough, it can be easily clogged; thus, a minimum size of 3 in should be used. To prevent clogging, a trash rack may be included in the design to cover the orifices.

12.7 WET POND (Retention Basin)

12.7.1 Introduction

The wet pond, as discussed in this section, is not designed for runoff control (see previous sections above), but is designed to satisfy EPA's water quality requirements.

A wet pond is very similar to a dry detention basin in that it detains stormwater, but it is different in that it maintains a permanent pool during dry weather. Wet ponds are usually more expensive than dry detention basins and usually serve large watersheds. Because of their permanent pool, they may also have esthetic or recreational benefits. HEC-22 (2) contains an illustration of a wet pond and discussion of wet ponds used to enhance water quality. Additional guidance is found in the EPA Storm Water Technology Fact Sheet Wet Detention Ponds (7). Figure 12.7-A provides an overview of wet pond considerations that are discussed in the following sections. These considerations will vary from state to state and the hydraulics designer should use the criteria set by the regulating authority.

Quality	Permanent pool volume is 3 times the WQV.
Quantity	Control design peak flow.
Shape	Length-to-width ratio > 3; wedge shaped (wider at outlet); permanent pool depth from 3 ft – 10 ft; perimeter ledges.
Maintenance	Inspect once a year, preferably during wet weather; mow at least twice a year; remove sediment every 5 to 10 years.
Safety	Provide shallow 1.5 ft deep 10 ft wide safety bench around pond; post signs; limit depths.
Other Considerations	Side slopes provide easy maintenance access 1V:3H; perimeter vegetation; sediment forebay; provide valve to drain pond for maintenance; aeration to control mosquitos and algae, as well as improve DO concentrations in outfall.
Pollutant Removal	Moderate to high.

Figure 12.7-A — OVERVIEW OF CONSIDERATIONS FOR A WET POND

12.7.2 Quality

For quality, the permanent pool must be at least 3 times the WQV for the watershed. The theory behind this is that incoming runoff displaces old stormwater from the pond and the new runoff is detained until it is displaced by more runoff from the next storm. A permanent pool of 3 times the WQV should then provide an adequate detention time for the stormwater. Watershed size, soil conditions and groundwater elevation must be evaluated to ensure the capability of the site to support a permanent wet pond. Figure 12.6-B shows the permanent pool volume for different areas. To enhance pollutant removal, several other considerations may be considered.

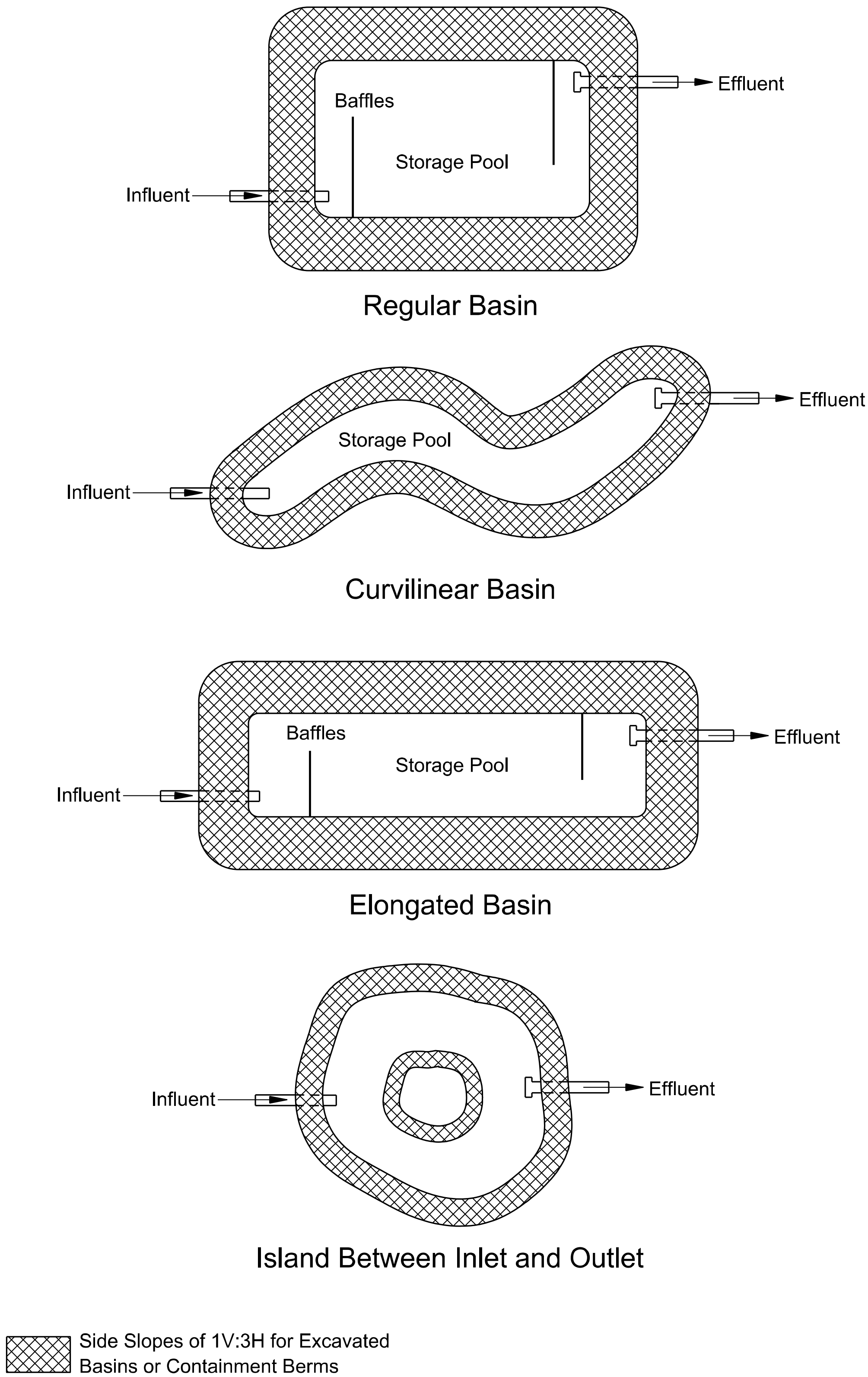
The shape of the basin can significantly affect the pollutant-removal efficiency of a wet pond. The length-to-width ratio should be at least 3:1. Figure 12.7-B shows pond configurations that may be used to increase the length-to-width ratio and allow for maximum flow path length. Pond depth should be between 3 ft and 10 ft if errant vehicles are unlikely; less could allow insect breeding and wind re-suspension of settled particles and more could lead to thermal stratification in the pond and anaerobic conditions in the deep water. An adequate depth to support fish will also aid in controlling insects. A wedge-shaped basin, wider at the outlet, can also improve pollutant removal (see Figure 12.7-C). Consideration should be given for errant vehicles, bicycles and even pedestrians when locating and benching ponds. Ponds in gore areas should provide adequate clear zones for errant vehicle recovery and safety benching to allow for safe passenger exit in more extreme cases.

12.7.3 Quantity

For quantity, the wet pond system is designed similarly to the dry pond. The pond should be designed to reduce the design peak flow and be able to pass a 100-year storm safely if the 100-year storm is not use for design. Routing the storms through the wet pond can be accomplished using the in design procedures in Section 12.4.

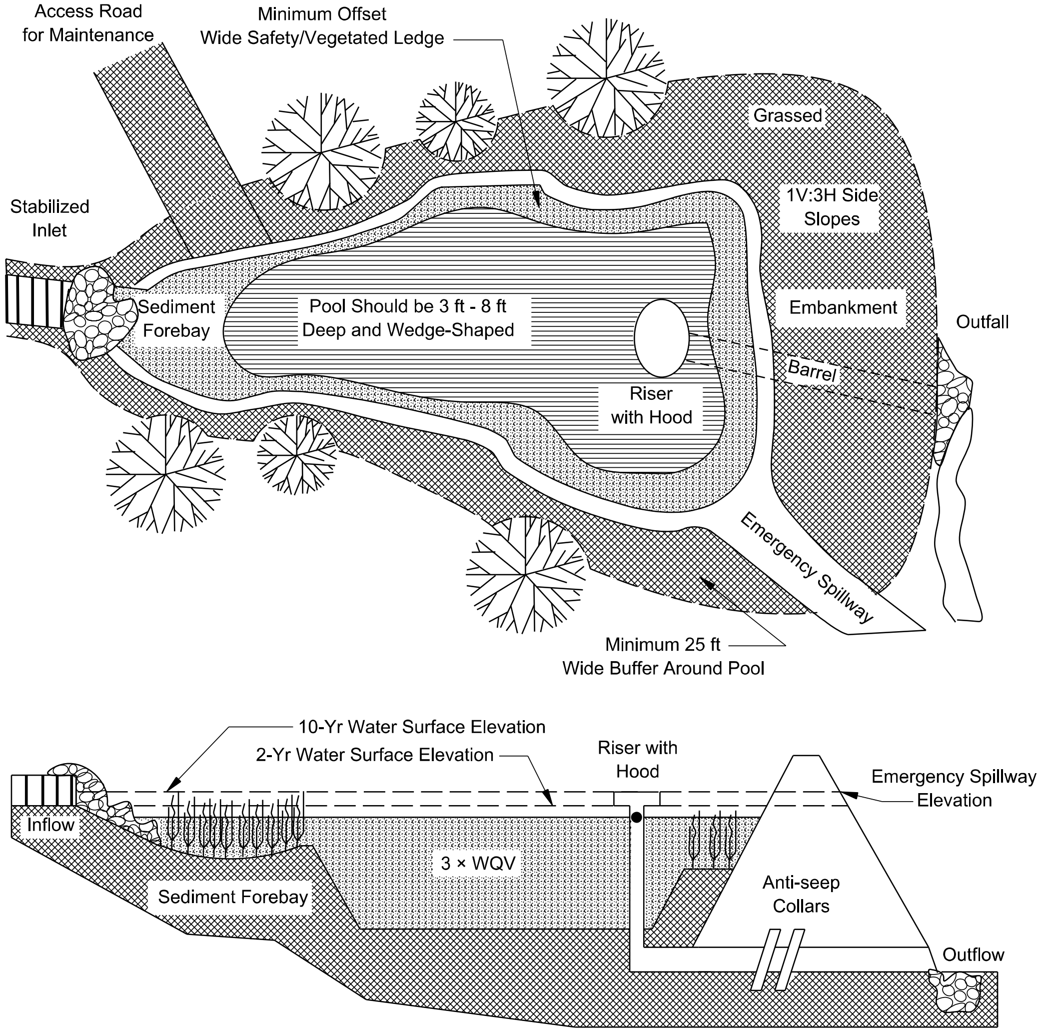
12.7.4 Outlet

Outlets for wet ponds can be designed in a wide variety of configurations. In states where ice is not a concern, riser pipes can be used. These risers can be designed to control different storms through the use of several orifices on the riser. For example, a small orifice is used to control a 2-year storm and a larger orifice to control the design storm. This larger flow is usually controlled by allowing stormwater to flow in through the top of the riser, using the entire riser diameter. In some cases, an antivortex design may be necessary. Larger flood flows are usually handled by an emergency spillway. If the smallest orifice is easily clogged from floating debris or if heated pond water causes problems downstream during the summer, the outlet can be modified so that it will release water from below the surface of the pond. Trash racks may also be included to prevent the outlet from clogging.



Source: Schueler (6)

Figure 12.7-B — METHODS OF INCREASING THE LENGTH-TO-WIDTH RATIO



Source: Schueler (6)

Figure 12.7-C — WET POND

12.8 INFILTRATION AND FILTRATION CONTROLS

12.8.1 Introduction

As stricter Federal and state water quality requirements are promulgated across the country, the emphasis on selecting best management practices (BMPs) that will help meet these more defined and stringent nutrient based water quality criteria becomes more critical. BMPs should be selected based on established pollution control strategy or nutrient management standards set by the governing environmental regulatory agency for the water body in question. Acceptable BMP designs, practices and nutrient or pollutant, removal rates vary by state, region, watershed and oversight agency. A detailed discussion of water quality impairments, acceptable nutrient concentrations, individual pollutants and potential solutions are beyond the scope of this chapter but a general overview is provided.

Infiltration and filtration controls are best management practices (BMPs) where the primary discharge of stormwater is to the permeable soils above the groundwater table. Runoff reduction via infiltration and good disconnection practices are among the best means of addressing both water quantity and water quality mitigation. The BMPs discussed are:

- infiltration trench (Section 12.8.3),
- infiltration basin (Section 12.8.4),
- vegetated filter strip (Section 12.8.5),
- grassed swale (Section 12.8.6), and
- wetlands (Section 12.8.7).

In some cases, the stormwater is intercepted after it has infiltrated a few feet by an underdrain and is discharged to a storm sewer or surface water. One of the primary concerns with the use of infiltration BMPs is the risk of groundwater contamination. This is why there should be at least 2 ft to 5 ft between the bottom of the facility to the seasonable high-water table and 5 ft to the underlying bedrock. Another factor is the residence time in the facility. Sources recommend that the first-flush stormwater be infiltrated within 24 to 72 hours. The infiltration rate is directly related to the soil type and disposition. A soil investigation should be performed at all facility locations prior to construction. Figure 12.8-A provides some considerations in evaluating an infiltration control. These considerations will vary from state to state and the hydraulics designer should use the criteria set by the regulating authority.

12.8.2 Site Selection

To evaluate different sites, a report from the Maryland State Highway Administration (MDSHA) (8) described a procedure that rates different sites by using several parameters (reader is referred to this study for details of the procedure). Upon completion of this procedure, several sites can be compared to determine which is the best for the infiltration BMP or if infiltration is even feasible. Other selection considerations include size of drainage area, proximity to foundations (the facility should be no closer than 10 ft down-gradient and 100 ft up-gradient from a foundation), water supply wells and karst topography.

.Quality	Infiltrate WQV within 72 h.
Quantity	Control 2- and 10- year peak flows (could lead to a large expensive facility; could be used with detention pond to control quantity).
Shape	Dependent on site constraints.
Maintenance	Inspect once a year; preferably during wet weather; mow area twice a year; remove sediment every 5 to 10 years.
Other Considerations	Filter strip to remove sediments 2% to 5% slope with minimum 20 ft length; infiltration rate minimum 1 in/h; depth to groundwater 2 ft to 5 ft and bedrock 5 ft; effects of facility on quality of groundwater.
Pollutant Removal	Moderate to high.

Figure 12.8-A — SUMMARY OF CONSIDERATIONS FOR AN INFILTRATION FACILITY

12.8.2.1 Infiltration Rate

A design infiltration rate with a factor of safety of two is used to determine the outflow from the facility for quantity and quality control. Design infiltration rates of greater than 1 in/h are preferred for infiltration facilities. After a suitable site for the facility has been found, several soil tests must be made before the facility is designed.

First, borings should be dug at the site to determine the soil types, depth to bedrock and groundwater and infiltration rates. These parameters can also be verified from county soil maps. The infiltration rate can be determined with varying elevation heads. This can be done with the "falling-head test" in the field (8). This procedure will yield a curve of outflow versus storage, which can be used to route storms through the facility.

Other considerations for the use of infiltration practices are well head protection areas and prime recharge areas that may require special precautions or pretreatments as part of the design if allowed at all. Shallow residential wells and septic systems are also of concern if within close proximity to the proposed BMP.

12.8.2.2 Observation Well

An observation well should be included in an infiltration facility with a covered bottom (i.e., trenches and pavement) to allow an inspector to determine how well the facility is operating (e.g., whether the stormwater is infiltrating as designed or whether maintenance is required). A schematic of a typical observation well in an infiltration trench is shown in Figure 12.8-B. It may also be necessary to install wells in infiltration basins to determine if they are working properly, but this can be determined visually because the stormwater is stored on the surface whereas the storage in trenches and pavement is hidden from view.

12.8.3 Infiltration Trench

An infiltration trench (see Figure 12.8-C) is a facility where a trench is excavated and then filled with a porous medium. Stormwater is stored in the voids of the fill material until it can be infiltrated. In a variation of this design, the stormwater is collected by an underdrain pipe after the stormwater has been detained and filtered by the trench. Infiltration trenches can be used in median strips or adjacent to parking lots. The bottom of the infiltration trench should be below the frost line and should be 3-5 ft above bedrock and the seasonably high groundwater table.

12.8.3.1 Quality

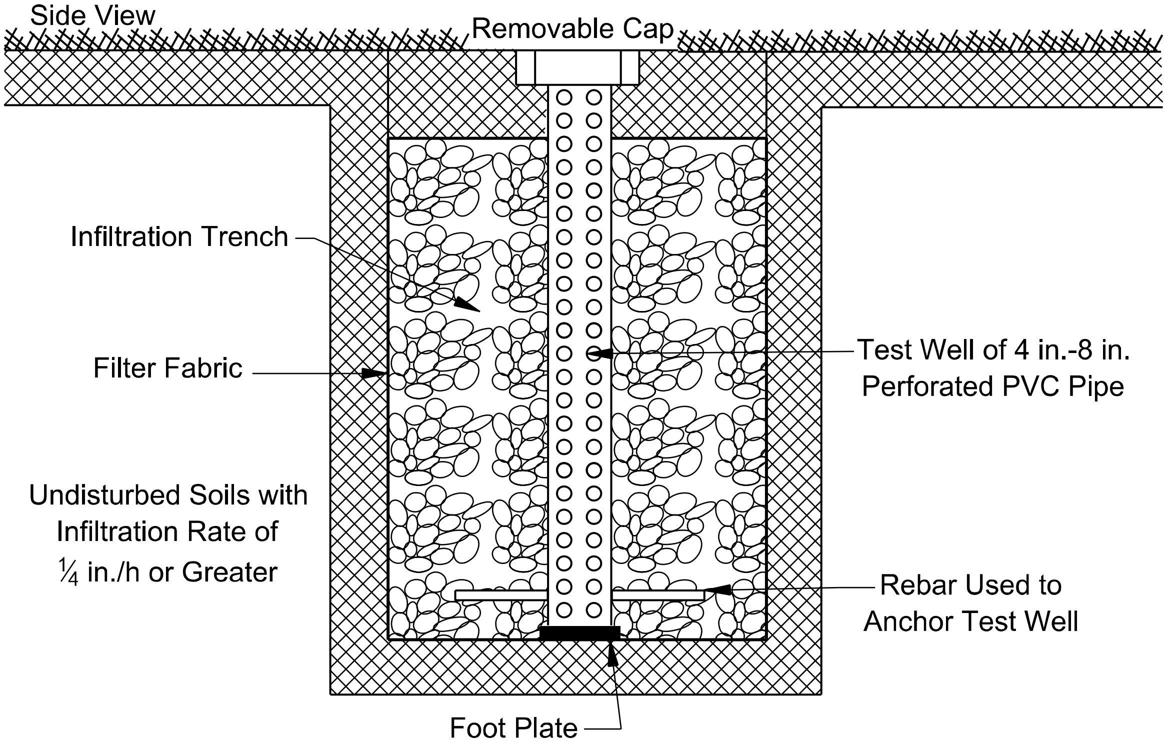
The WQV must be infiltrated within 48 h. The primary removal mechanisms in trenches are sedimentation and filtration, along with some biological uptake. Filtering is achieved in the top layers of the facility as stormwater enters. In the infiltration trench, the main removal mechanisms are sedimentation and adsorption.

As the stormwater leaves, it is filtered again by the underlying soil, where more pollutants will be removed. Unfortunately, all infiltration facilities are vulnerable to clogging, thereby reducing their effectiveness. Therefore, a vegetated buffer strip filtering the runoff is recommended as part of an infiltration facility. The strip would decrease the amount of suspended solids in the stormwater and thus increase the useful life of the infiltration facility. The filter strip should be at least 20 ft wide. It should also be sloped from 2% to 5% to prevent water from ponding and to ensure a slow velocity.

12.8.3.2 Quantity

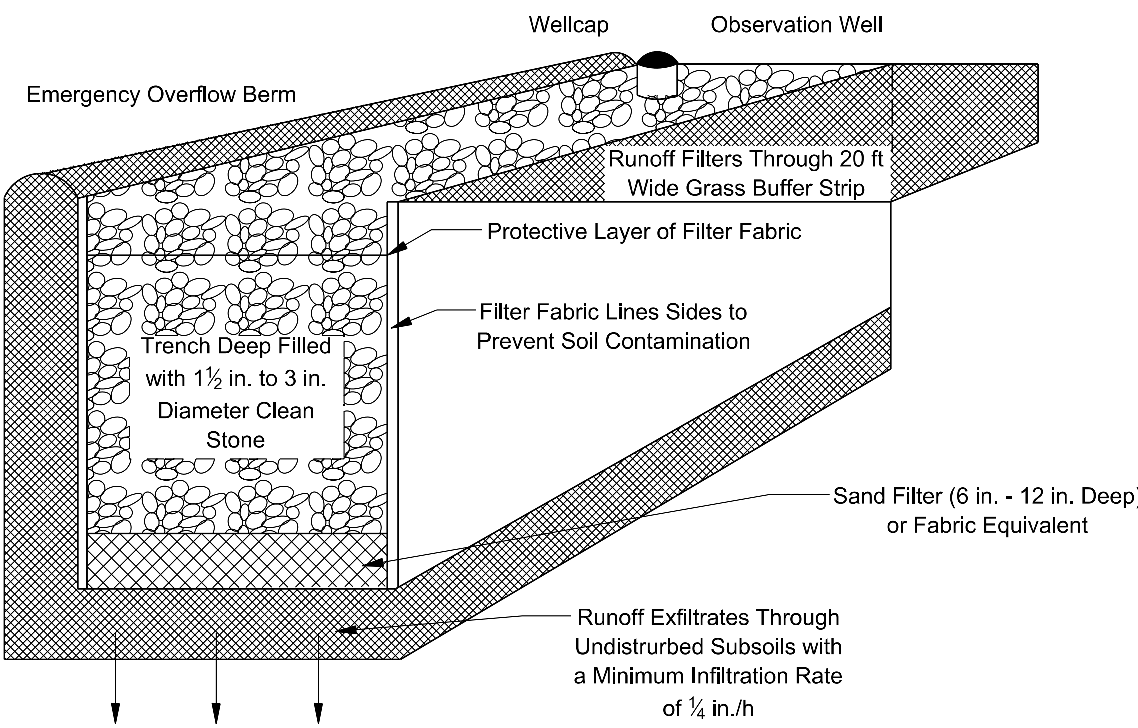
Because of the large size of the trench that would be required to control a 10-year storm, it is suggested that trenches not be used for large drainage areas or areas where the increase in peak flow and, therefore the amount of storage required, is very large.

A trial-and-error process of routing the design storms through the facility can be used to determine the amount of storage required for quantity control. Although the methods used to determine the amount of storage required for the ponds are not derived for infiltration BMPs, they can still be used to obtain an initial estimate of the required storage.



Source: Schueler (6)

Figure 12.8-B — INFILTRATION TRENCH WITH OBSERVATION WELL



Source: Schueler (6)

Figure 12.8-C — INFILTRATION TRENCH

After a storage volume has been determined, the dimensions of the facility can be estimated. The surface area can be manipulated to suit the site conditions if it yields the required storage volume. The amount of surface area required is:

$$S_a = \frac{Vol_s}{V_r d}$$ Equation 12.8(1)

Where:

S_a = surface area, ft^2
Vol_s = storage volume, ft^3
V_r = void ratio (0.4 for $1^1/_2$-in to 3-in aggregate)
d = depth, ft

The inflow hydrograph can be calculated by a number of methods, and the outflow-versus-storage curve can be found from a falling-head test. Thus, the storms can be routed through the facility, and the size of the trench can be changed to reduce the peak outflows to the preconstruction levels of the design storm. When routing the storm through the trench, one can determine whether and how much flow will bypass the trench when it is filled with stormwater. The overflow from the trench must be contained in an adequate channel.

12.8.3.3 Detention Time

Detention time is an important factor in determining the effectiveness of a trench facility. A facility that drains quickly is capable of treating more stormwater volume. A maximum detention time of 72 h is recommended. The actual detention time can be estimated by:

$$T_s = \frac{dV_r}{f}$$ Equation 12.8(2)

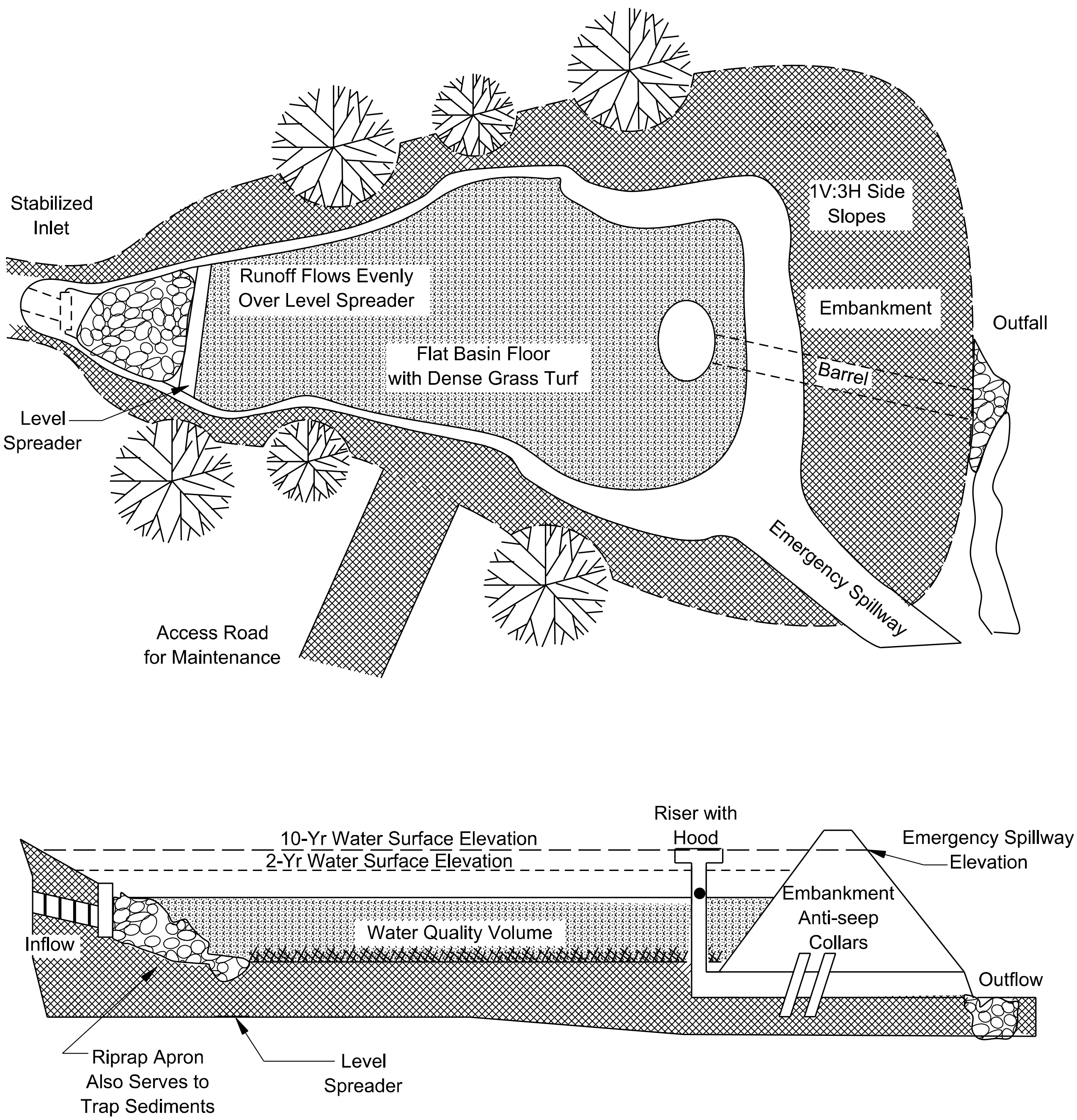
Where:

T_s = storage time or detention time, h
d = depth of storage in the trench, in
V_r = void ratio of stone reservoir
f = steady infiltration rate, in/h

From this equation, it can be seen that detention time is directly related to trench depth. Because the WQV will most likely be much smaller than the storage required for a 10-year storm, the depth of the WQV will be very small in the trench. Therefore, infiltration trenches are much better suited for small drainage areas where the change in peak flow between pre- and post-construction is small. Modifications can be made to the trench design to increase the depth of the WQV storage, but these will increase the cost and could make this BMP option infeasible.

12.8.4 Infiltration Basin

An infiltration basin looks very similar to a dry pond; see Figure 12.8-D. Stormwater from smaller, more frequent storms is infiltrated through the bottom of the basin. Larger storms can be controlled through infiltration or by a "peak-shaving" outlet, or both. The most important consideration for an infiltration basin is keeping the bottom from clogging with sediment. The clogging of basins, along with the overestimating of their infiltration rates, has led to the failure of many infiltration basins.



Source: Schueler (6)

Figure 12.8-D — INFILTRATION BASIN

12.8.4.1 Quality

To protect groundwater quality, the bottom of the infiltration basin must be 5 ft or more above the bedrock and the seasonably high groundwater table. The WQV should be infiltrated within 48 h. The primary removal mechanisms in infiltration basins are sedimentation, filtration and biological uptake. Filtering is provided by the vegetation at the bottom of the pond and, preferably, also by a buffer strip before the stormwater runoff enters the facility. The filter strip should be at least 20 ft wide and should also be sloped from 2% to 5% to prevent water from ponding and to ensure a slow velocity. Vegetation can also contribute to the removal of pollutants through biological uptake. As the stormwater leaves, it is filtered again by the underlying soil.

An estimation of the maximum ponding depth for a desired drain time can be found with the equation

$$d = fT_s \qquad \text{Equation 12.8(3)}$$

Where:

d = depth, in
f = steady infiltration rate, in/hr
T_s = time of storage, h

The recommended maximum allowable storage time is 48 h. Considering that basins may fail because of clogging and an infiltration rate that is lower than expected, a shorter time of storage, say 40 h, might be used to compensate for inaccuracies in estimating infiltration rates.

Several other considerations can enhance the pollutant removal of these facilities. First, vegetation should be established on the basin floor. A dense stand of water-tolerant grass with a deeply penetrating root system would help stabilize the bottom of the basin and help keep the soil open. Vegetation would also provide the biological uptake of nutrients.

Second, the pond bottom should be sloped as close to zero as possible to obtain a uniform depth of stormwater over the basin. The side slopes should be sloped at 1V:3H, or flatter, to allow for easy maintenance access and prevent erosion.

A third consideration pertains to the incoming stormwater. A combination of a level spreader/sediment forebay can be constructed to spread the stormwater evenly, thereby reducing erosion and trap sediments before they clog the basin. Riprap should also be placed at the inlet to help reduce erosion.

12.8.4.2 Quantity

For an infiltration basin, quantity can be controlled very similarly to a detention basin. The basin should be designed to reduce the peak flow from the design storm and be able to pass a 100-year storm safely.

After a required storage volume is estimated, the design storm should be routed through the basin to determine if the estimated value is correct.

12.8.5 Vegetated Filter Strip

A filter strip is a vegetated area that is designed to accept sheet flow. While flowing over the strip, stormwater is filtered by the vegetation, infiltrated and detained. The most common cause for failure of filter strips is runoff bypassing the strip through eroded channels. If the stormwater is not evenly distributed over the entire strip, a channel could form and the strip would lose effectiveness. To prevent the channelization, a level spreader can be used, as shown in Figure 12.8-E.

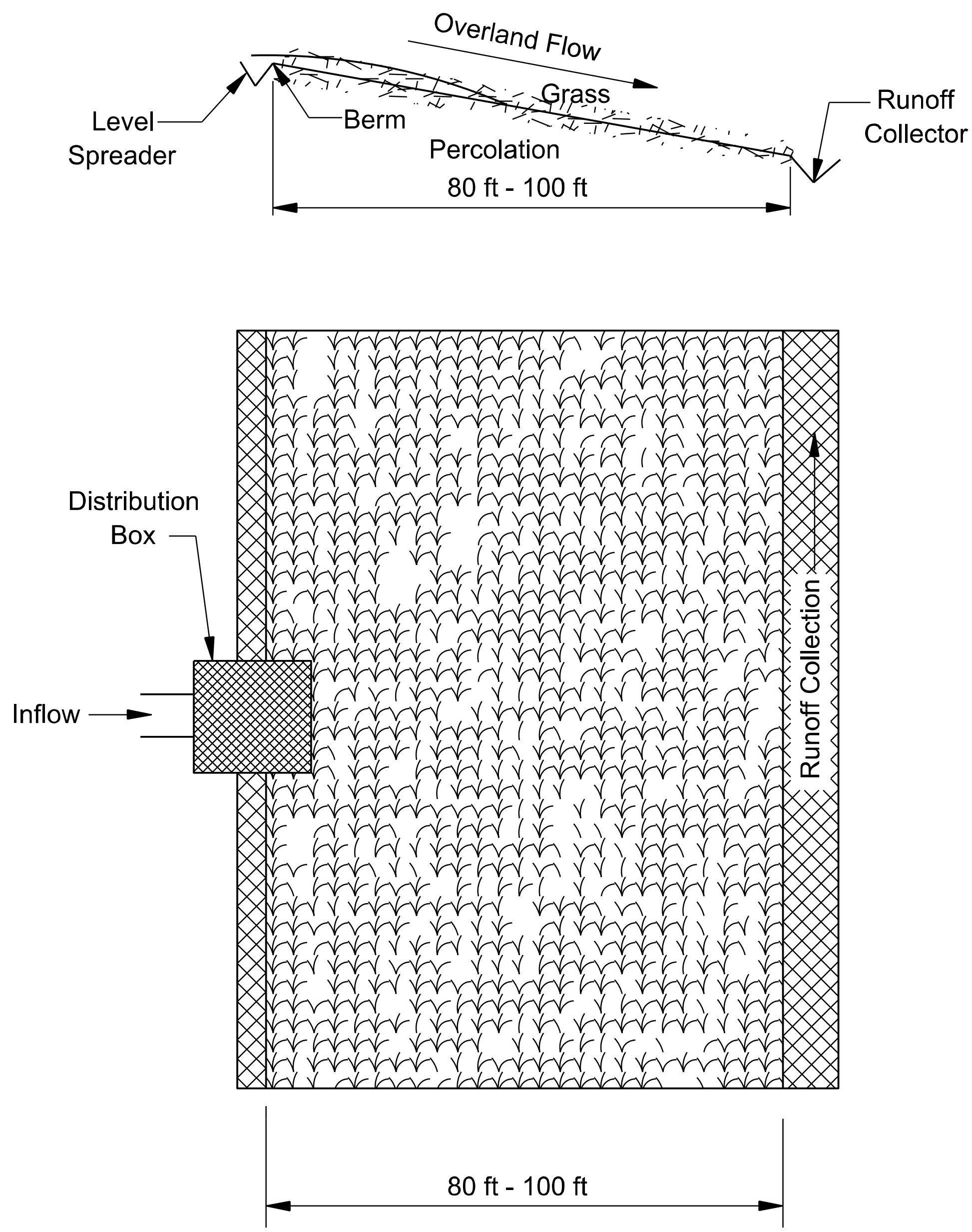


Figure 12.8-E — LEVEL SPREADER SYSTEM

Filter strips can be used to filter runoff before it enters a structural facility, or they could be used alone. A study by Yu, et al., (9) found that the level spreader was at least as cost effective as a wet pond for pollutant removal in stormwater. However, its use for quantity control is limited to small drainage areas, with small increases in peak flows.

Filter strips should be constructed of dense, soil-binding, deep-rooted, water-resistant plants. They are usually constructed of grass, but forested strips are also feasible (they can have higher pollutant removal rates but should be longer because of their lack of cover and susceptibility to erosion). For the filter strips to be effective, their slope should be no more than 5% and their length should be at least 20 ft.

Figure 12.8-G was developed by Wong and McCuen (10) for determining the required length of a grassed filter strip. If the slope of the strip, roughness coefficient (Manning's n) and desired trap efficiency are known, the length required can be found by using Figure 12.8-G. The example in Figure 12.8-G is for a slope of 2%, an n of 0.20 and a trap efficiency of 95%; the required filter strip length is 200 ft.

As previously stated, the use of a level spreader is intended to spread runoff evenly and prevent the formation of channels in the filter strip. Several designs have been developed; the main consideration is that the overflow from the level spreader be distributed equally across the filter strip. This can be done through the use of a rock-filled trench or a plastic-lined trench that acts as a small detention pond. The bottom and filter-side lip should have a zero slope to ensure an even distribution of runoff onto the strip. Figure 12.8-E depicts a level spreader.

12.8.6 Grassed Swale

Grassed swales are roadside stormwater conveyances that can store, filter and infiltrate runoff. Originally, they were an inexpensive way of rapidly transporting runoff from a site. In contrast, runoff should be slowed down and detained for stormwater management purposes.

Some studies have been conducted on the use of swales for runoff quality control and a wide variety of estimates of their effectiveness have been reported. From these studies, design guidelines have been developed for constructing swales so that the pollutant removal efficiency is improved.

Several studies conducted by Yousef, et al (11) for the Florida Department of Transportation found roadside swales to be effective in removing many highway pollutants. An equation was developed to calculate the length of a swale that allows all of the stormwater to infiltrate for a given runoff flow rate:

$$L = \frac{KQ^{5/8}S^{3/16}}{n^{3/8}f} \qquad \text{Equation 12.8(4)}$$

Where:

L	=	length of swale, ft
K	=	constant (see Figure 12.8-F)
Q	=	average runoff flow rate, cfs

S = longitudinal slope, ft/ft
n = Manning's roughness coefficient
f = infiltration rate, in/hr

Z (Side Slope) (1V:ZH)	K	Z (Side Slope) (1V:ZH)	K
1	981 000	6	485 000
2	854 000	7	443 000
3	712 000	8	408 500
4	612 000	9	380 000
5	540 000	10	357 600

Source: Wanielista (12)

Figure 12.8-F — SWALE LENGTH CONSTANT, K

If this leads to a swale that is too long, another equation may be used to determine the placement of swale blocks or check dams to compensate for the reduction in length where required because of site limitation, etc. By modifying the equation, the volume of storage required in the swale can be determined by:

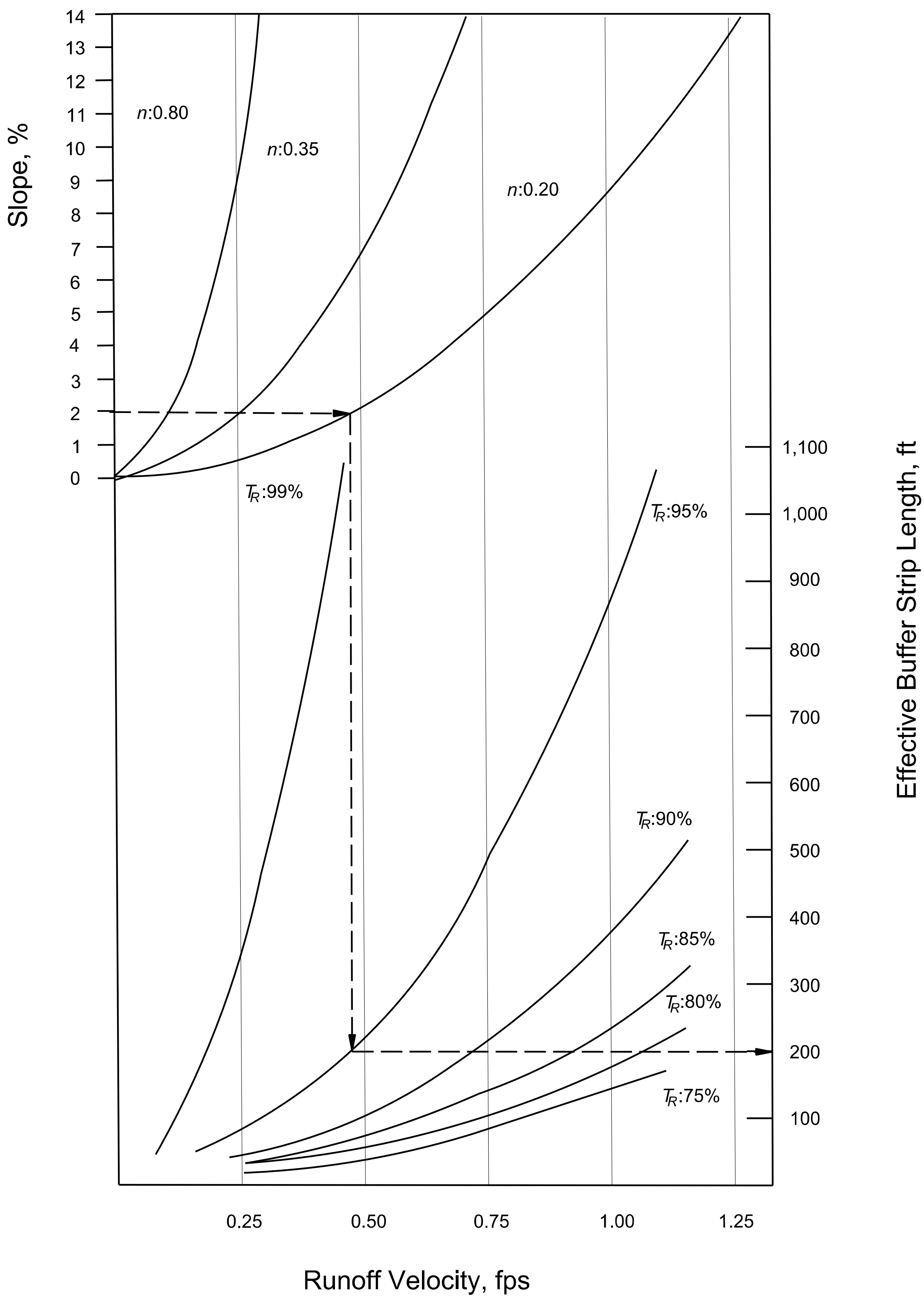
$$Vol = Q(\Delta t) - \left(\frac{Ln^{3/8} f}{KS^{3/16}}\right)^{8/5} (\Delta t) \qquad \text{Equation 12.8(5)}$$

Where:

$Q(\Delta t)$ = volume of runoff at the end of time interval Δt
Δt = time interval, s
$\left(\frac{Ln^{3/8} f}{KS^{3/16}}\right)^{8/5}$ = the value of runoff percolated during time interval Δt by a certain swale with length L

Units of all other variables are the same as defined in Equation 12.8(4). The answer obtained by solving Equation 12.8(5) is, therefore, the volume that must be stored behind the check dam in the swale so that all stormwater is allowed to infiltrate.

The pollutant removal efficiency of a swale can be improved through enhancing filtering by grass in the channel. To enhance grass filtering, the swale should be designed as a triangle, with at least 1V:3H side slopes, or a parabola, with a 6:1 top width-to-depth ratio. The grass in the swale should be dense, deep rooted and water tolerant. The grass should be high enough to cover the depth of runoff in the swale but not so high that it is flattened by the flowing stormwater.



Source: Wong and McCuen (10)

Figure 12.8-G — REMOVAL RATES (R_R) FOR BUFFER STRIPS

12.8.7 Wetlands

Wetlands have the ability to remove many pollutants and wetlands detain stormwater. However, the processes that occur in wetlands are not fully understood, and the amount of wetland area required to treat stormwater can be very large. Stormwater is known to degrade wetlands either by smothering wetland vegetation with sediment or by causing extreme fluctuations in water depth. It has been recommended that wetlands and marshes be used in conjunction with other BMPs (e.g., on the bottom of dry ponds and on the fringes of wet ponds). Although a substantial amount of information is available on using wetlands as a final treatment process of wastewater, very little is known on using wetlands for treating stormwater. A report by Marble (13) provided guidelines for designing replacement wetlands. With regard to using wetlands for stormwater management (SWM), Marble reported that urban runoff is a good source of nutrients for the development of wetlands and that wetlands downstream of an impoundment may have reduced aquatic diversity because of reductions in the outflow detritus. Marble further stated that wetlands have the ability to remove sediments and toxins through sedimentation. However, the loadings of toxins and sediments should be low to moderate, and the ratio of wetland area to watershed area should be kept high. The functions of wetlands with regard to water quality are very complex. Hemond and Benoit (14) noted the following:

> *The wetland is not a simple filter; it embodies chemical, physical and biotic processes that can detain, transform, release or produce a wide variety of substances. Because wetland water quality functions result from the operation of many individual, distinct and quite dissimilar mechanisms, it is necessary to consider the nature of each individual process.*

The very limited number of studies undertaken on the use of wetlands for SWM indicates a wide disparity in the efficiency of wetlands to remove pollutants. A study by Martin (15) suggested that wetlands, when used in conjunction with another BMP (e.g., a wet detention pond), can be quite effective in treating highway stormwater runoff. However, more research is needed before wetlands can be used for water quality or volume control without post-construction monitoring to verify that the wetland is functioning as desired. See the AASHTO *Drainage Manual* (1) Volume Two, Chapter 8 “Wetlands” for further information.

12.9 LAND-LOCKED RETENTION

12.9.1 Introduction

Watershed areas that drain to a central depression with no positive outlet (e.g., playa lakes) are typical of many topographic areas including karst topography and can be evaluated using a mass-flow routing procedure to estimate flood elevations. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrogeologic phenomenon that requires good field measurements and a thorough understanding of local conditions. Because outflow rates for flooded conditions are difficult to calculate, field measurements are desirable.

12.9.2 Mass Routing

The steps presented below for the mass-routing procedure are illustrated by the example given in Figure 12.9-A:

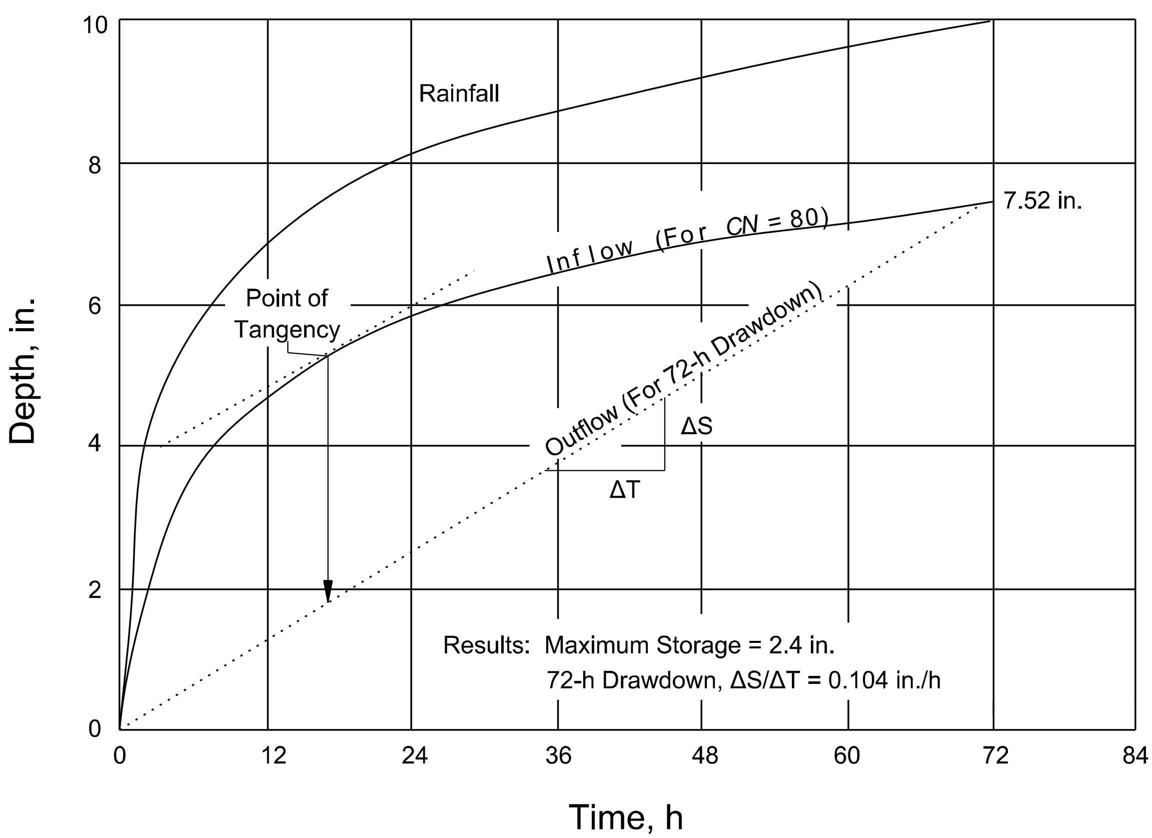


Figure 12.9-A — MASS ROUTING CURVE

Step 1. Obtain cumulative rainfall data for the 100-year frequency, 10-day duration design event from Figure 12.9-B.

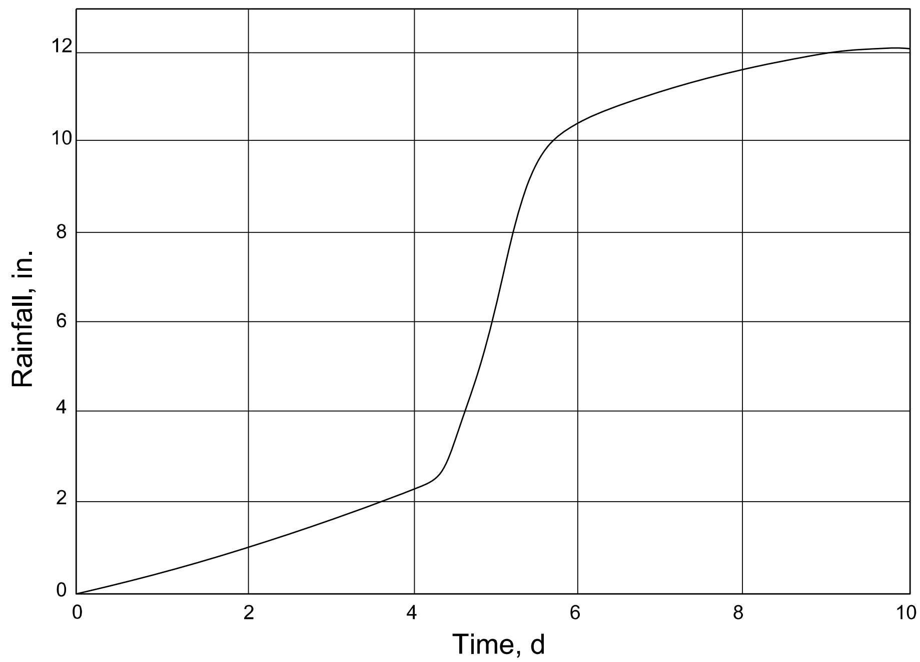


Figure 12.9-B — CUMULATIVE RAINFALL DATA FOR 100-YEAR, 10-Day DESIGN STORM (Site Specific)

Step 2. Calculate the cumulative inflow to the land-locked retention basin using the rainfall data from Step 1 and runoff procedure from the Chapter 7 "Hydrology." Plot the mass inflow to the retention basin.

Step 3. Develop the basin outflow from field measurements of hydraulic conductivity, considering worst-case watertable conditions. Hydraulic conductivity should be established using in-situ test methods, then results compared to observed performance characteristics of the site. Plot the mass outflow as a straight line with a slope corresponding to worst-case outflow in in/h.

Step 4. Draw a line tangent to the mass-inflow curve from Step 2, which has a slope parallel to the mass-outflow line from Step 3.

Step 5. Locate the point of tangency between the mass-inflow curve of Step 2 and the tangent line drawn for Step 4. The distance from this point of tangency and the mass-outflow line represents the maximum storage required for the design runoff.

Step 6. Determine the flood elevation associated with the maximum storage volume determined in Step 5. Use this flood elevation to evaluate flood protection requirements of the project. The zero-volume elevation should be established as the normal wet season water surface or watertable elevation or the pit bottom, whichever is highest.

Step 7. If runoff from the project area discharges into a drainage system tributary to the land-locked depression, detention storage facilities are required to comply with the pre-development discharge requirements for the project.

Unless the storage facility is designed as a retention facility, including water budget calculations, environmental needs and provisions for preventing anaerobic conditions, relief structures shall be provided to prevent standing water conditions.

12.10 STORAGE WATER BUDGET

Water budget calculations are required for all permanent pool facilities and should consider performance for average annual conditions. The water budget should consider all significant inflows and outflows including, but not limited to, rainfall, runoff, infiltration, exfiltration, evaporation and outflow.

Average annual runoff may be computed using a weighted runoff coefficient for the tributary drainage area multiplied by the average annual rainfall volume. Infiltration and exfiltration should be based on site-specific soils testing data. Evaporation may be approximated using the mean monthly pan evaporation or free-water surface evaporation data from NOAA Technical Report 33 (16).

12.10.1 Example Water Budget

A shallow basin with an average surface area of 3 acres and a bottom area of 2 acres is planned for construction at the outlet of a 100-acre watershed. The watershed is estimated to have a post-development runoff coefficient of 0.3. Site-specific soils testing indicates that the average infiltration rate is approximately 0.1 in/h. Determine for average annual conditions if the facility will function as a retention facility with a permanent pool.

From rainfall records, the average annual rainfall is approximately 50 in
The mean annual evaporation is 35 in

The average annual runoff is estimated as:

Runoff = (0.3)(50 in)(100 acres) = 1500 acres-in

The average annual evaporation is estimated as:

Evaporation = (35 in)(3 acres) = 105 acres-in

The average annual infiltration is estimated as:

Infiltration = (0.1 in/hr)(24 h/d)(365 d/yr)(2 acres)
Infiltration = 1752 acres-in

Neglecting basin outflow and assuming no change in storage, the runoff (or inflow) less evaporation and infiltration losses is:

Net Budget = 1500 – 105 – 1752 = -357 acres-in

Thus, the proposed facility will not function as a retention facility with a permanent pool.

Revise pool design as follows:

Average surface area = 2 acres and bottom area = 1 acres

Recompute the evaporation and infiltration:

Evaporation = (35 in)(2 acres) = 7 acres-ft
Infiltration = (0.1 in/hr)(24 h/d)(365 d/yr)(1 acre) = 876 acres-in

The revised runoff less evaporation and infiltration losses is:

Net Budget = 1500 – 70 – 876 = +554 acres-in

The revised facility is assumed to function as a retention facility with a permanent pool.

12.11 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

12.11.1 General

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage facilities. To assure acceptable performance and function, storage facilities that require extensive maintenance are discouraged. Facilities should be designed to minimize the following maintenance problems that are typical with urban detention facilities:

- weed growth,
- grass and vegetation maintenance,
- sedimentation control,
- bank deterioration,
- standing water or soggy surfaces,
- mosquito control,
- blockage of outlet structures,
- litter accumulation, and
- maintenance of fences and perimeter plantings.

Proper design should focus on the elimination or reduction of maintenance requirements by addressing the potential for problems to develop:

- Address weed growth and grass maintenance by constructing side slopes that can be maintained using available power-driven equipment (e.g., tractor mowers).

- Control sedimentation by constructing traps to contain sediment for easy removal or low-flow channels to reduce erosion and sediment transport.

- Control bank deterioration with protective lining or by limiting bank slopes.

- Eliminate standing water or soggy surfaces by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or constructing underdrain facilities to lower watertables. If standing water is addressed, mosquito control will not be a major problem.

- Select outlet structures to minimize the possibility of blockage (i.e., very small pipes tend to block quite easily and should be avoided). Ice accumulation should also be considered.

- Locate the facility for easy access so that maintenance can be conducted on a regular basis to address litter and damage to fences and perimeter plantings.

12.11.2 Sediment Basins

Often detention facilities are used as temporary sediment basins. To control the maintenance of these facilities, establish criteria to determine when these facilities should be cleaned and how much of the available storage can be used for sediment storage.

The following is an example methodology that can be used to develop a sediment basin maintenance schedule. Figure 12.11-A can be used to estimate sediment-trap efficiency for sediment basins. The procedure for using this figure is as follows:

1. Establish sediment-generation criteria (e.g., 1786 ft^3 silt per disturbed acre draining to the sediment basin). The amount of sediment from a given area is estimated using the Revised Universal Soil Loss Equation (RUSLE).

2. Estimate total volume available for sediment storage from the geometric shape of the basin (e.g., 19,421 ft^3).

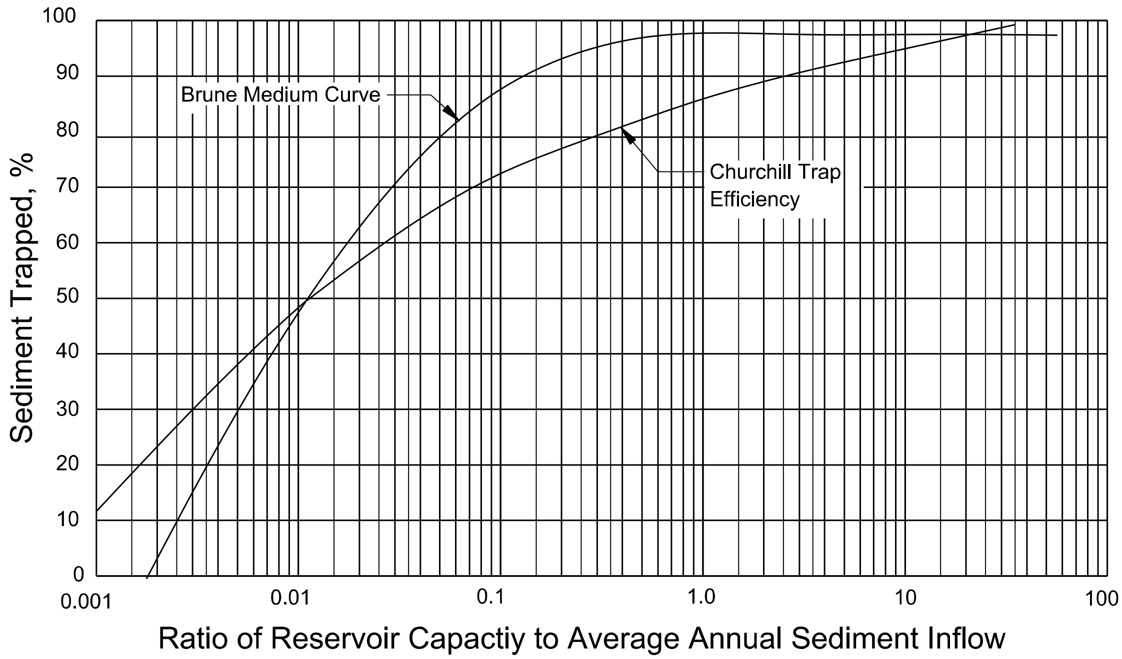
3. Calculate minimum silt storage needed given the silt generation criteria (e.g., 1786 ft^3 per disturbed acre $\times$ 9.88 acres of disturbed area = 17,646 ft^3).

4. Trap efficiency can be estimated from Figure 12.11-A as follows:

   - (19,421 cu ft available storage/17,646 cu ft) is near 1.1.
   - From Figure 14-21, Trap Efficiency = 90% (Churchill) and 98% (Brune).

5. Determine how frequently the sediment basin should be cleaned.

   - Because the basin will trap most of the sediment and most of the capacity will be filled annually, the basin should be cleaned annually.



Source: USBR (17)

Notes:

1. *Capacity is total sediment basin volume up to emergency spillway crest.*
2. *A general guideline is to use the Brune method for large storage or normal ponded reservoirs and the Churchill curve for settling basins, small reservoirs, flood-retarding structures, semi-dry reservoirs or reservoirs that are continuously sluiced.*

Figure 12.11-A — EFFICIENCY OF SEDIMENT BASINS

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