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Preface

This document was extracted from the City of Griffin Stormwater Design Manual. The Stormwater Design Manual was created by the City of Griffin to address the impacts of urban development and stormwater runoff on the environment.

6.0 Storage Facilities

6.1 Introduction

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. 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 general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations.

Location Considerations

It should be noted that the location of storage facilities is very important as 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 that could decrease or increase flood peaks in different downstream locations. Thus it is important for the engineer 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 on a regional or basin-wide planning basis.

Detention And Retention

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For the purpose of 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. Retention facilities are designed to contain a permanent pool of water. Since 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. If special procedures are needed for detention or retention facilities these will be specified.

When to Use Storage Facilities

For each development over one and one-tenths (1.1) acres in size, a stormwater impact evaluation prepared by a registered professional engineer is required. If this evaluation indicates that a proposed development will increase runoff from the property to a level that cannot be accommodated within the downstream drainage system, then storage facilities may be used to control the runoff from the proposed development to a level that can be accommodated within the downstream drainage system. See The City of Griffin Land Development Guidelines for further information on stormwater detention facilities.

6.2 Symbols and Definitions

To provide consistency within this chapter as well as throughout this manual, the following symbols 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 they occur in this chapter, the symbol will be defined where it occurs in the test or equations.

Table 6-1 Symbols and Definitions

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
Δt	Routing time period	sec
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
Q	Flow or outflow rate	cfs
S, V_s	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_i	Duration of basin inflow	hrs
t_p	Time to peak	hrs
V_s, S	Storage volume	ft^3
W	Width of basin	ft
Z	Side slope factor	-

6.3 Design Criteria

General Criteria

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 (i.e., 100-year flood). The design criteria for storage facilities should include:

- Release rate,
- Storage volume,
- Grading and depth requirements,
- Safety considerations and landscaping,
- Outlet works, and location.

Note: The same hydrologic procedure shall be used to determine pre-and post-development hydrology.

Release Rate

Control structure release rates shall approximate pre-developed peak runoff rates for the 2-year through 25-year storms, with emergency overflow capable of handling the 100-year discharge. Design calculations are required to demonstrate that the facility will limit runoff from the 2-, 5-, 10-, and 25-year developed discharge rates to pre-developed peak discharge rates.

Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates for the 2-year through 25-year storms. Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project. For detention basins, all detention volume shall be drained within 72 hours.

Grading and Depth

Following is a discussion of the general grading and depth criteria for storage facilities followed by criteria related to detention and retention facilities.

6.3.1.1 General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments shall be less than 20 feet in height and shall have side slopes no steeper than 3:1(horizontal to vertical). Riprap-protected embankments shall be no steeper than 2:1. Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluation can be found in most soil engineering textbooks, including those by Spangler and Handy (1982) and Sowers and Sowers (1970).

A minimum freeboard of 1 foot above the 100-year design storm high water elevation shall be provided for impoundment depths of less than 20-feet. Impoundment depths greater than 20 feet are subject to the requirements of the Safe Dams Act unless the facility is excavated to this depth.

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.

6.3.1.2 Detention

Areas above the normal high water elevations of storage facilities should be sloped at a minimum of 5 percent toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2 percent 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 present standing water conditions. Often a sediment collection forebay is provided with easy maintenance access.

6.3.1.3 Retention

The maximum depth of permanent storage facilities will be determined by site conditions. Design constraints, and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A depth of 6 to 8 feet is generally reasonable unless fishery requirements dictate otherwise. Aeration may be required impermanent pools to prevent anaerobic conditions. Where aquatic habitat is required, wildlife experts should be contacted for site-specific criteria relating to such things as depth, habitat, and bottom and shore geometry. In some cases a shallow bench along the perimeter is constructed to encourage emergent vegetation growth to enhance the pollution reduction capabilities or aesthetics of the pond.

Outlet Works

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow, and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of drop inlets, pipes, weirs, and orifices. Slotted riser pipes are discouraged because of clogging problems, but curb openings may be used for parking lot storage. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet. For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency outlet is the 100-year flood. The sizing of a particular outlet works shall be based on results of hydrologic routing calculations.

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. 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. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream.

The following procedure is recommended to determine downstream effects. For all proposed storage facilities, channel routing calculations should proceed downstream to a confluence point where the drainage area being analyzed represents ten percent of the total drainage area. At this point the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph can be assessed and shown not to have detrimental effects on downstream areas.

Detention can be located within floodplains and still effectively control flooding through the use of timing calculations. In this situation the flood peak coming down the stream rarely coincides with local on-site flooding. It is often advantageous to allow the on-site water to pass with simple erosion control and a properly sized conveyance system. Then locate the detention pond to “skim” the peak from the oncoming flood hydrograph through the use of a side-channel weir or a simple flow through depression along the banks.

Safe Dams Act

Under the dam safety act regulations a dam is an artificial barrier that does or may impound water and that is 20 feet or greater in height and has a maximum storage volume of 30 acre-feet or more. A number of exemptions are allowed from the Safe Dams Act and any questions concerning a specific design or application should be addressed to the Georgia Department of Natural Resources.

6.4 General Procedure

Data Needs

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

- Inflow hydrograph for all selected design storms for fully developed and pre-developed conditions.
- Stage-storage curve for proposed storage facility (see Figure 6-1 below for an example).
- Stage-discharge curve for all outlet control structures (see Figure 6-2 below for an example).

Using these data a design procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet geometry until the desired outflow hydrograph is achieved (see example 6.8).

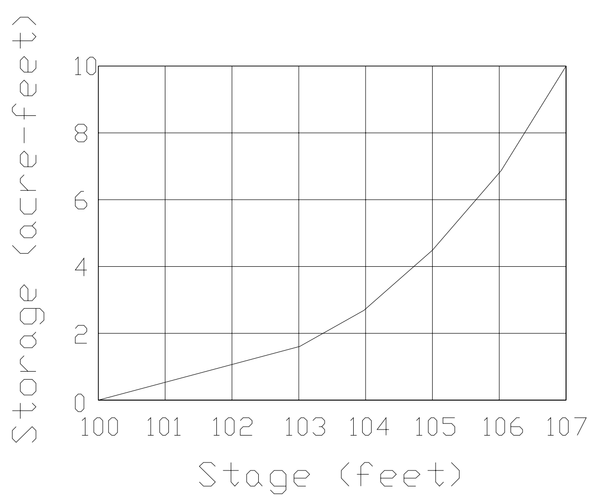


Figure 6-1 Example Stage-Storage Curve

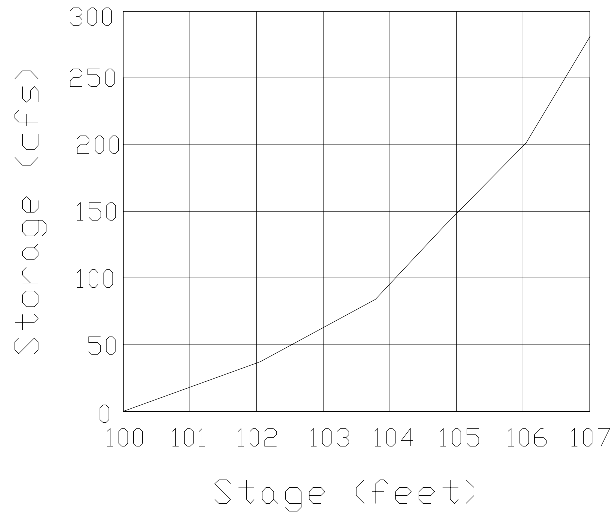


Figure 6-2 Example Stage-Discharge Curve

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 are usually developed using a topographic map and the double-end area frustum of a pyramid, prismoidal formulas or circular conic section. The double-end area formula is expressed as:

$$\mathbf{V_{1,2} = [(A_1 + A_2)/2]d} \qquad \textbf{(6.1)}$$

Where: $V_{1,2}$ = storage volume, ft^3, between elevations 1 and 2
A_1 = surface area at elevation 1, ft^2
A_2 = surface area at elevation 2, ft^2
d = change in elevation between points 1 and 2, ft

The frustum of a pyramid is expressed as:

$$\mathbf{V = d/3\ [A_1 + (A_1 \times A,)^{0.5} + A_2]/3} \qquad \textbf{(6.2)}$$

Where: V = volume of frustum of a pyramid, ft^3
d = change in elevation between points 1 and 2, ft
A_1 = surface area at elevation 1, ft^2
A_2 = surface area at elevation 2, ft^2

The prismoisal formula for trapezoidal basins is expressed as:

$$\mathbf{V = LWD = (L = W)\ ZD^2 = 4/3\ Z^2\ D^3} \qquad \textbf{(6.3)}$$

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

The circular conic section formula is:

$$\mathbf{V = 1.047\ D\ (R_1^2 = R_2^2 = R_1R_2)} \qquad \mathbf{(6.4)}$$
$$\mathbf{V = 1.047\ D\ (3R_1^2 = 3ZDR_1 = Z_2D^2)} \qquad \mathbf{(6.5)}$$

Where R_1 and R_2 = bottom and surface radii of the conic section, ft
D = depth of basin, ft
Z = side slope factor, ratio of horizontal to vertical

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 spillways: principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir or other appropriate outlet can be used for the principal spillway of outlet. The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway should be designed taking into account the potential threat to downstream life and property if the storage facility were to fail.

The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway.

Procedure

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

1. Compute inflow hydrograph for runoff from the 2-, 5-, 10-, and 100-year design storms using the procedures outlined in the Hydrology Chapter. Both pre-and post-development hydrographs are required.

2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Section 6.7).

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.

4. Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If the routed post-development peak discharges from the 2-through 25-year design storms exceed the pre-development peak discharges, 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.

6. Perform routing calculations using the 100-year hydrograph, for developed land use conditions, to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created then the storage facility must be designed to control the increased flows from the 100-year storm. If not then consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.

7. Evaluate the downstream effects of detention outflows from all design storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed through the downstream channel system until a confluence point is reached where the drainage area being analyzed represents 10 percent of the total drainage area.

8. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream. See the Energy Dissipation Chapter for information on controlling outlet velocities and the design of energy dissipators.

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

10 Percent Limit

The 10 percent limit procedure utilizes a hydrologic-hydraulic computer model to analyze the downstream effects of stormwater runoff from developments of different size, shape, physical characteristics, and location within larger drainage basins. Based on the model, the effects of a development process stabilizes at the point where the proposed development represents approximately 10 percent of the drainage area, depending on the size of the development and the amount of increase impervious area.

If the 10 percent analysis study shows that there is increased peak flows downstream, several alternatives are available to the engineer to deal with the increased flows. These alternatives include installing on-site detention facilities, using extended detention facilities, upgrading drainage structures or conveyance downstream, obtaining easements, controlling runoff from the development site with infiltration or methods other than detention, etc.

Detention facilities are required when they will provide positive benefits to the local drainage system and shall not be required when they will be ineffective or not needed, which shall be determined by the City of Griffin Stormwater Department.

The following basic steps for using the 10 percent downstream analysis include the following:

- Develop hydrographs for the design storms at the discharge point(s) from the proposed development. The proposed developed land use conditions within the development should be used to develop these hydrographs.
- Route these hydrographs through the downstream drainage system to a point downstream where the size of the proposed development represents 10 percent or less of the total drainage area that contributes runoff to this point. This point is called the 10 percent point.
- For all points of interest in the downstream drainage system, between the exit of the proposed development to the 10 percent point, develop hydrographs from the contributing areas. Existing land use conditions should be used for this analysis for all areas not included in the proposed development. Points of interest would include locations where drainage from sub-watersheds intersect, known drainage and flooding problems exist, where structures might be affected by storm runoff, etc. As a minimum, hydrographs at the 10 percent point should be developed with and without the proposed development.
- A comparison of the routed hydrograph from the proposed development with the other downstream hydrographs should indicate whether or not the proposed development will increase downstream peak flows or have little or no effect on these peak flows.
- If major constrictions (e.g., storage facilities, undersized culverts) are present in the downstream analysis area that will affect the general characteristics of the hydrographs, the associated engineering parameters of these constrictions should be included in the analysis.
- In most cases general topographic maps, soils information, and a field check of the drainage system will provide the data needed for this analysis.
- Detailed survey information and backwater analysis should not be needed for most downstream analysis.

6.5 Outlet Hydraulics

Outlets

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data.

Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated below. This discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [(3.27 + 0.4(H/H_c)]\ LH^{1.5}} \qquad \textbf{(6.6)}$$

Where: Q = discharge, cfs
H = head above weir crest excluding velocity head, ft
H_c = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

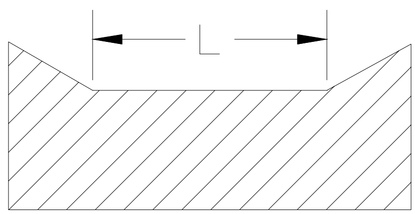


Figure 6-3
Sharp-Crested Weir No End Contractions

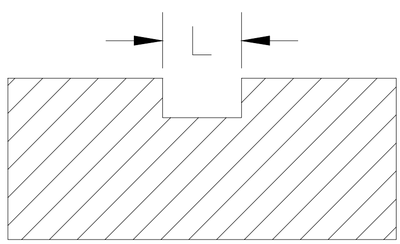


Figure 6-4
Sharp-Crested Weir And Head

A sharp-crested weir with two end contractions is illustrated below. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [(3.27 + .04(H/H_c)]\ (L - 0.2H)\ H^{1.5}} \qquad \textbf{(6.7)}$$

Where: Variables are the same as equation 6.4.

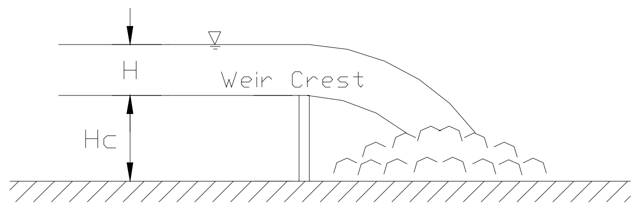


Figure 6-5
Sharp-Crested Weir, Two End Contractions

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$\mathbf{Q_s = Q_f(1 - (H_2/H_1)^{1.5})^{0.385}} \qquad \textbf{(6.8)}$$

Where: Q_s = submergence flow, cfs
Q_f = free flow, cfs
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

Broad-Crest Weirs

The equation for the broad-crested weir is (Brater and King, 1976):

$$\mathbf{Q = CLH^{1.5}} \qquad \textbf{(6.9)}$$

Where: Q = discharge, cfs
C = broad-crested weir coefficient
L = broad-crested weir length, ft
H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 6-2.

V-Notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976):

$$\mathbf{Q = 2.5\ tan(\theta/2)H^{2.5}} \qquad \textbf{(6.10)}$$

Where: Q = discharge, cfs
θ = angle of v-notch, degrees
H = head on apex of notch, ft

Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharged relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are (Sandvic, 1985):

$$\mathbf{Q = 4.97\ a^{0.5}\ b(H - a/3)} \qquad \textbf{(6.11)}$$

$$\mathbf{x/b = 1 - (1/3.17)\ (arc\ tan\ (y/a)^{0.5})} \qquad \textbf{(6.12)}$$

Where: Q = discharge, cfs
Dimensions a, b, h, x, and y are shown below

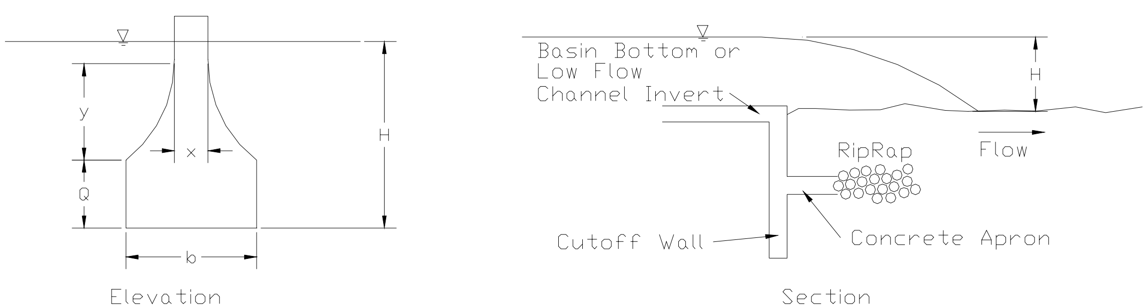


Figure 6-6 Proportional Weir Dimensions

Orifices

Pipes smaller that 12” may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions,

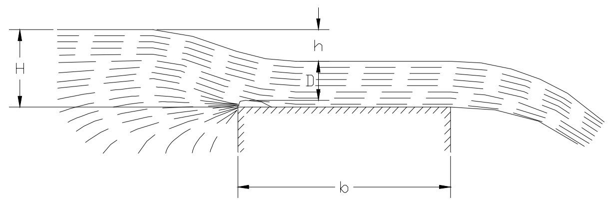
$$\mathbf{Q = 0.6A(2gH)^{0.5} = 3.78D^2H^{0.5}} \qquad \mathbf{(6.13)}$$

Where:
- Q = discharge, cfs
- A = cross-section area of pipe, ft^2
- g = acceleration due to gravity, 32.2 ft/s^2
- D = diameter of pipe, ft
- H = head on pipe, from the center of pipe to the water surface

Combination Outlets

Combinations of weirs, pipes and orifices can be put together to provide a variable control stage-discharge curve suitable for control of multiple storm flows. They are generally of two types: shared outlet control and separate outlet controls. Shared outlet control is typically a number of individual outlet openings, weirs or drops at different elevations on a riser pipe or box that all flow to a common larger conduit or pipe. Separate outlet controls are less common and normally consist of a single opening through the dam of a detention facility in combination with an overflow spillway for emergency use. For a complete discussion of outlets and combination outlets see Municipal Stormwater Management by Debo and Reese.

Table 6-2 Broad Crested Weir Coefficient C Values As A Function of Weir Crest Breadth (b) and Head (H) Weir Crest Breadth (ft)



Measured Head, H[1] (ft)	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	25.37	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1] Measured at least 2.5H upstream the weir.
Reference: Brater and King (1976).

6.6 Preliminary Detention Calculations

Storage Volume

For small drainage areas, 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 Figures 6-7 shown below.

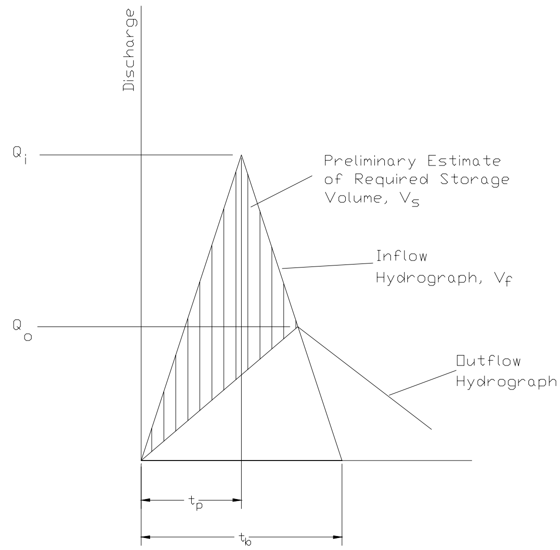


Figure 6-7 Triangular Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)

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

$$\mathbf{Vs = 0.5T_i(Q_i - Q_o)} \qquad \textbf{(6.14)}$$

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

Any consistent units may be used for Equation 6.14

Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff & Singh, 1986).

1. Determine input data, including the allowable peak outflow rate Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

1. Calculate a preliminary estimate of the ratio V_s/V_r, using the input data from Step 1 and the following equation:

$$\mathbf{V_s/V_r = [1.291(1\text{-}Q_o/\ Qi)^{0.753}]/[t_b/t_p)^{0.411}]} \qquad \textbf{(6.15)}$$

Where: V_s = volume of storage, in
V_r = volume of runoff, in
Q_o = outflow peak flow, cfs
Q_i = inflow peak flow, cfs
t_b = time base of the inflow hydrograph, hr (Determined as time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak.)
t_p = time to peak of the inflow hydrograph, hr

2. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated in Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

1. Determine volume of runoff, V_r, peak flow rate of the inflow hydrograph, Q_1, time base of the inflow hydrograph, t_b, time to peak of the inflow hydrograph t_p, and storage volume, V_s.

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Singh, 1976):

$$\mathbf{Q_o/Q_i = 1 - 0.712(V_s/V_r)^{1.328}(t_b/t_p)^{0.546}} \qquad \textbf{(6.16)}$$

Where: Q_o = outflow peak flow, cfs
Q_i = inflow peak flow, cfs
V_s = volume of storage, in
V_r = volume of runoff, in
t_b = time base of the inflow hydrograph, hr (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak.)
t_p = time to peak of the inflow hydrograph, in hours

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated from step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume (see example 6.8.3).

6.7 Routing Calculations

The following procedure is used to perform routing through a reservoir or storage facility (Puls Method of storage routing).

1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown below.

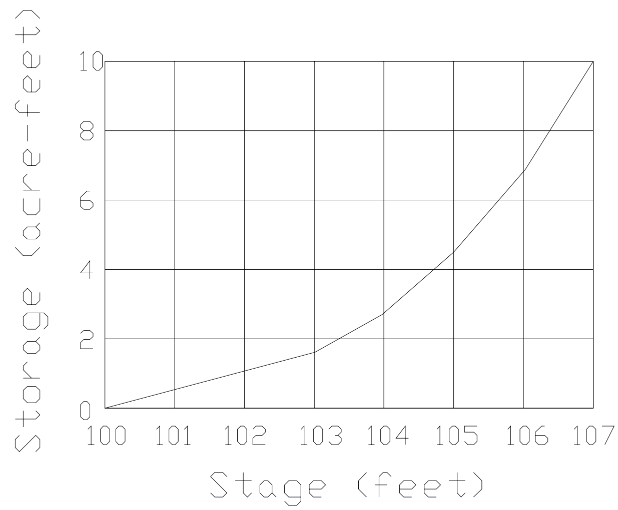


Figure 6-8 Example Stage-Storage Curve

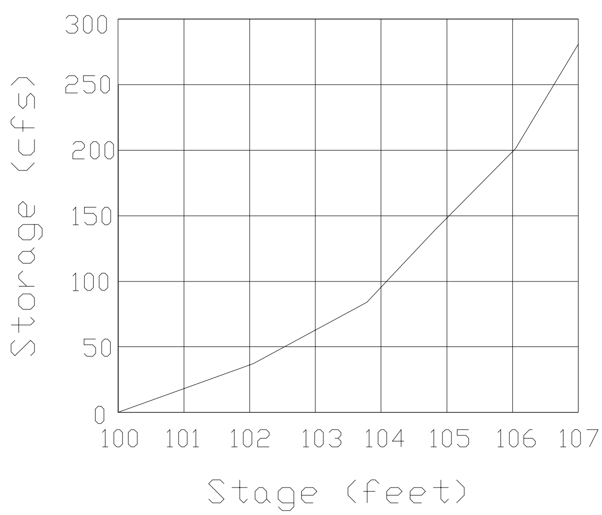


Figure 6-9 Example Stage-Discharge Curve

Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph.

2. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of S$\pm$(O/2) Δt versus stage. An example tabulation of storage characteristics curve data is shown in Table 6-3.

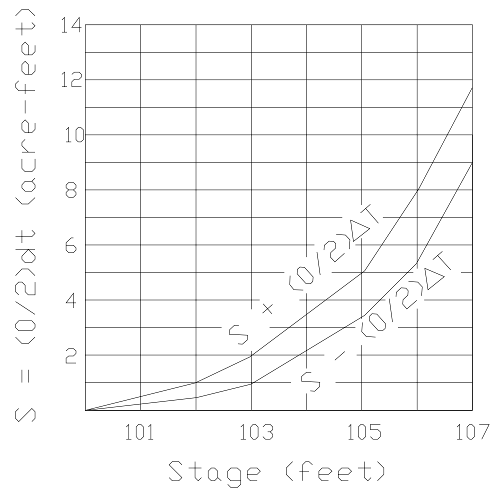


Table 6-3 Storage Characteristics

(1) Stage (ft)	(2) Storage[1] (ac-ft)	(3) Discharge [2] (ac-ft/hr)	(4)	(5) S – (O/2)Δt (ac-ft)	(6) S + (O/2)Δt (ac-ft)
100	0.05	0	0.00	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.58	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10.0	275	22.73	8.11	11.89

[1]Obtained from the Stage-Storage Curve.
[2]Obtained from the Stage-Discharge Curve.
Note: t = 10 minutes = 0.167 hours and 1 cfs = 0.0826 ac-ft/hr.
(If the detention facility contains a permanent pool of water, this can be accounted for by considering the water surface as the zero stage.)

3. For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, S_1-(O_1/2) Δ t can be determined from the appropriate storage characteristics curve (example given below).

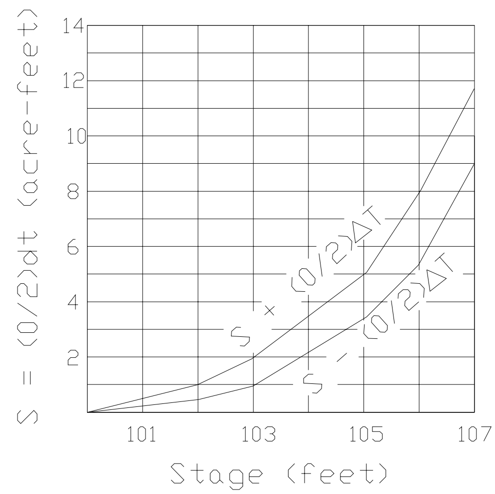


Figure 6-10 Storage Characteristic Curve

4. Determine the value of $S_2 + (O_2/2)\Delta t$ from the following equation:

$$\mathbf{S_2 + (O_2/2)\,\Delta t = [S_1 - (O_1/2)\,\Delta t] + [(I_1 + I_2)2\Delta t]} \qquad \mathbf{(6.17)}$$

Where: S_2 = storage volume at time 2, ft^3
O_2 = outflow rate at time 2, cfs
Δt = routing time period, sec
S_1 = storage volume at time 1, ft^3
O_1 =outflow rate at time 1, cfs
I_1 = inflow rate at time 1, cfs
I_2 = inflow rate at time 2, cfs

Other consistent units are equally appropriate.

5. Enter the storage characteristics curve at the calculated value of $S_2 + (O_2/2)\,\Delta t$ determined in Step 4 and read off a new depth of water, H_2.

6. Determine the value of O_2, which corresponds to a state of H_2, determined in Step 5, using the stage-discharge curve.

7. Repeat Steps 1 through 6 by setting new values of I_1, O_1, S_1, and H_1 equal to the previous I_2, O_2, S_2, and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph ahs been routed through the storage basin.

6.8 Example Problem

Example

This example demonstrated the application of the methodology presented in this chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre-and post-development conditions are assumed to have been developed using hydrologic methods from the Hydrology Chapter. Note: In this example only the 5-and 25-year hydrographs are used. The 2-and 10-year hydrographs should also be checked to determine if the final design is adequate.

Design Discharge and Hydrographs

Storage facilities shall be designed for runoff from the 2-, 5-, 10-, 25-, and 50-year design storms and an analysis done using the 100-year design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures. Example peak discharges from the 5-and 25-year design storm events are as follows:

- Pre-developed 5-year peak discharge = 150 cfs
- Pre-developed 25-year peak discharge = 200 cfs
- Post-development 5-year peak discharge = 190 cfs
- Post-development 25-peak discharge = 250 cfs

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 150 and 200 cfs for the 5-and 25-year storms, respectively.

Example runoff hydrographs are shown in Table 6-4 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 5-and 25-year storms.

Table 6-4 Example Runoff Hydrographs

Pre-Development Runoff			Post-Development Runoff	
(1)	(2)	(3)	(4)	(5)
Time (Hrs)	5-Year (cfs)	25-Year (cfs)	5-Year (cfs)	25-Year (cfs)
0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190>150	250>200
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

Finally, the 2-and 10-year hydrographs should then be routed through the storage facility to be sure these storms are adequately controlled.

Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlines in Section 6.6. For runoff from the 2-and 10-year storms, the required storage volumes V_s, are computed using equation 6.14;

$$V_s = O.5T_i(Q_i - Q_o)$$

5-year storm: $V_s = [O.5(1.2 \times 3{,}600)(190 - 150)]/43{,}560 = 1.98$ acre-feet

25-year storm: $V_s - [O.5(1.25 \times 3{,}600)(250 - 200)]/43{,}560 = 2.58$ acre-ft.

Design and Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from both the 5-and 25-year design storms are presented below. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 5-and 25-year design storms to be provided when the corresponding allowable peak discharges occurred. Storage values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 3.1, a weir length of 4 feet, and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

Table 6-5 Stage-Discharge-Storage Data

(1) Stage (ft)	(2) Q (cfs)	(3) S (acre-feet)	(4) $S_1 - (0/2)$ (acre-feet)	(5) $S_1 + (0/2)\ \Delta t$ (acre-feet)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.30	0.22
1.4	20	0.42	0.50	0.33
1.8	30	0.56	0.68	0.43
2.2	40	0.69	0.85	0.52
2.5	50	0.81	1.02	0.60
2.9	60	0.93	1.18	0.68
3.2	70	1.05	1.34	0.76
3.5	80	1.17	1.50	0.84
3.7	90	1.28	1.66	0.92
4.0	100	1.40	1.81	0.99
4.5	120	1.63	2.13	1.14
4.8	130	1.75	2.29	1.21
5.0	140	1.87	2.44	1.29
5.3	150	1.98	2.60	1.36
5.5	160	2.10	2.76	1.44
5.7	170	2.22	2.92	1.52
6.0	180	2.34	3.08	1.60
6.4	200	2.58	3.41	1.76
6.8	220	2.83	3.74	1.92
7.0	230	2.95	3.90	2.00
7.4	250	3.21	4.24	2.17

Storage routing was conducted for runoff from both the 5-and 25-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the Stage-Discharge Data given above and the Storage Characteristics Curves given on Figures 6-8 and 6-9, and 0.1 hour time steps are shown below for runoff from the 5-and 25-year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 5-and 25-year design storms.

Table 6-6 Storage Routing For The 5-Year Storm

(1) Time (hrs)	(2) Inflow (cfs)	(3) $[(I_1+I_2)]/2$ (acre-ft)	(4) H1 (ft)	(5) $S_1-(O_1/2)\Delta t$ (acre-ft) (6)-(8)	(6) $S_2+(O_2/2)\Delta t$ (acre-ft) (3) + (5)	(7) H (ft)	(8) Outflow (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	38	0.16	0.00	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130 <150 OK
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	850

Table 6-7 Storage Routing For The 25-Year Storm

(1) Time (hrs)	(2) Inflow (cfs)	(3) $[(I_1+I_2)]/2$ (acre-ft)	(4) H1 (ft)	(5) $S_1-(O_1/2)\Delta t$ (acre-ft) (6)-(8)	(6) $S_2+(O_2/2)\Delta t$ (acre-ft) (3) + (5)	(7) H (ft)	(8) Outflow (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.6	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173 <200 OK
0.5	90	1.05	5.80	1.30	2.35	4.00	137
0.6	50	0.58	4.95	1.25	1.83	4.10	1.3
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.00	0.90	0.22	0.22	0.60	6

For the routing calculations the following equation was used:

$$S_2 + (O_2/2)\Delta t = [S_1-(O_1/2)\Delta t\} + [I_1 + I_2)/2\Delta t]$$

Also, column 6 = column 3 + column 5

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, runoff from the 200-year storm should be routed through the storage facility and downstream to determine if structures or adjacent land areas will be damaged. If flood damages will result, the storage facility must then be designed to limit the runoff from the 100-year storm to undeveloped conditions. Also, the 100-year routed storm should be used to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability and maintenance, grading to prevent standing water, and provisions for public safety. Also, the 2- and 10-year storms should be checked.

6.9 Trash Racks and Safety Gates

Trash racks and safety grates serve several functions:

- they trap larger debris well away from the entrance to the outlet works where they will not clog the critical portions of the works;
- they trap debris in such a way that relatively easy removal is possible;
- they keep people and large animals out of confined conveyance and outlet areas;
- they provide a safety system whereby persons caught in them will be stopped prior to the very high velocity flows immediately at the entrance to outlet works and persons will be carried up and onto the outlet works allowing for an ability to climb to safety, and
- well designed trash racks can have an aesthetically pleasing appearance.

When designed well trash racks serve there purpose without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985, Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors including: head losses through the rack, structural convenience, safety, and size of outlet.

Trash racks at entrances to pipes and conduits should be sloped at 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level, the slower the approach flow the flatter the angle. Rack opening rules-of-thumb abound in the literature. Figure 6-12 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g. a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack. Alternately debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978, UDFCD, 1991). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate. Nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations though many have difficult to estimate variables. For a discussion of head loss related to grates with example empirical loss equations see Debo & Reese, 1994.

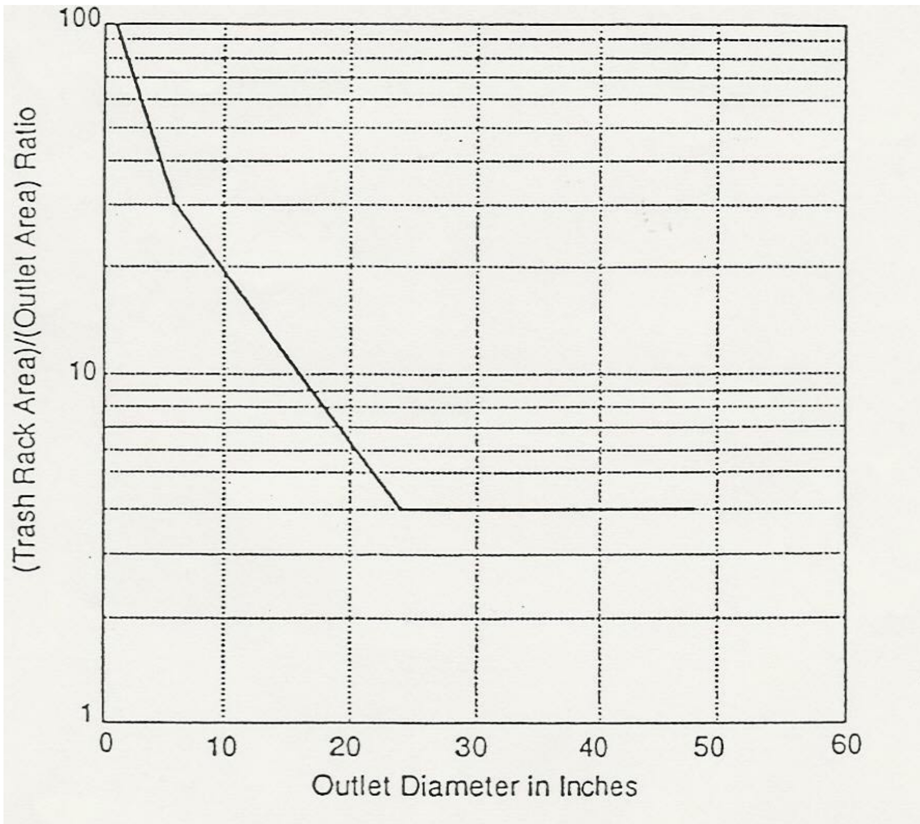


Figure 6-12 Minimum Rack Size vs. Outlet Diameter (UDCFD, 1992)

6.10 References

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Table 6-8 General Application Controls

STRUCTURAL CONTROL CATEGORY	STRUCTURAL CONTROL	STORMWATER TREATMENT SUITABILITY			WATER QUALITY PERFORMANCE				SITE APPLICABILITY					IMPLEMENTATION CONSIDERATIONS			
		Overbank Flood Protection	Channel Protection	Water Quality	TSS/Sediment Removal	Nutrient Removal	Bacteria Removal	Hotspot Application	Drainage Area (acres)	Space Req'd (% of tributary imp. Area)	Site Slope	Minimum Head Required	Depth to Water Table	Residential Usage	Ultra Urban	Capital Cost	Maintenance Burden
Ponds	Wet Pond	X	X	X	Good	Good	Good	X	25 min **	2-3%	15% max	6 to 8 ft	2 feet, if hotspot or aquifer	X		Low	Low
	Wet ED Pond	X	X	X	Good	Good	Good	X						X		Low	Low
	Micropool ED Pond	X	X	X	Good	Fair	ND	X	10 min**					X		Low	Moderate
	Multiple Ponds	X	X	X	Good	Good	Good	X	25 min **					X		Low	Low
Wetlands	Shallow Wetland	X	X	X	Good	Good	Good	X	25 min	3-5%	8% max	3 to 5 ft	2 feet, if hotspot or aquifer	X		Moderate	Moderate
	Shallow ED Wetland	X	X	X	Good	Good	Good	X						X		Moderate	Moderate
	Pond/Wetland	X	X	X	Good	Good	Good	X				6 to 8 ft					
	Pocket Wetland		X	X	Good	Good	Good	X	5 min			2 to 3 ft	below WT	X	X	Moderate	High
Filtering Systems	Surface Sand Filter			X	Good	Fair	Fair	X	10 max***	2-3%	6% max	5 ft	2 feet		X	High	High
	Perimeter Sand Filter			X	Good	Fair	Fair	X	2 max***			2 to 3 ft			X	High	High
	Bioretention Filter			X	Good	Fair	ND	X	5 max***	5%		5 ft		X	X	Moderate	Moderate
Open Channels	Dry Swale			X	Good	Fair	Poor	X	5 max	10-20%	4% max	3 to 5 ft	2 feet	X		Moderate	Low
	Wet Swale			X	Good	Fair	Poor	X	5 max			1 ft	below WT	X		High	Low
Infiltration	Infiltration Trench			X	Good	Good	Fair		5 max	2-3%	6% max	1ft	4 feet	X	X	High	High

Table 6-9 Limited/Special Application Controls

STRUCTURAL CONTROL CATEGORY	STRUCTURAL CONTROL	STORMWATER TREATMENT SUITABILITY			WATER QUALITY PERFORMANCE				SITE APPLICABILITY					IMPLEMENTATION CONSIDERATIONS			
		Overbank Flood Protection	Channel Protection	Water Quality	TSS/Sediment Removal	Nutrient Removal	Bacteria Removal	Hotspot Application	Drainage Area (acres)	Space Req'd (% of tributary imp. Area)	Site Slope	Minimum Head Required	Depth to Water Table	Residential Usage	Ultra Urban	Capital Cost	Maintenance Burden
Wetlands		X	X	X				X			2% max				X	High	High
Filtering Systems	Organic Filter			X				X	5 max**	2-3%	6% max	2 to 4 ft	2 feet			High	High
	Underground Sand Filter			X				X	2 max**	None	6% max	5 to 7 ft	2 feet		X	High	High
	Filter Strips			X				X			6% max					Low	Moderate
	Catch Basin Inserts			X				X	less than 1	None					X	Moderate	High
Open Channels	Grass Channel			X					5 max		4% max					Low	Moderate
Infiltration	Dry Wells			X					less than 1	None					X	Low	Moderate
Chemical Treatment	Alum Treatment System			X				X							X	High	High
Detention		X	X						5 max	None					X	High	Low

Table 6-10 General Application Controls – Specific Criteria

STRUCTURAL CONTROL CATEGORY	PHYSIOGRAPHIC FACTORS		SOILS	SPECIAL WATERSHED CONSIDERATIONS		
	Low Relief	High Relief		High Quality Stream	Aquifer Protection	Reservoir Protection
Ponds	Limit maximum normal pool depth to about 4 feet (dugout) Providing pond drain can be problematic	Embankment heights restricted	"A" soils may require pond liner "B" soils may require infiltration testing	Require control of CP	May require liner if "A" soils are present Pretreat hotspots 2 to 4 ft SD from Water Table	Require control of CP
Wetlands		Embankment heights restricted	"A" soils may require pond liner	Require control of CP	May require liner if "A" soils are present Pretreat hotspots 2 to 4 ft SD from Water Table	Require control of CP
Filtering Systems	Several design variations will likely be limited by low head		Clay or silty soils may require pretreatment	Should be used in treatment train with another control to provided CP	Needs to be designed with no exfiltration (I.e. outflow to groundwater	
Open Channels	Generally feasible however slope <1% may lead to standing water in dry swales	Often infeasible if slopes are 4% or greater		Should be used in treatment train with another control to provided CP	Hotspot runoff must be adequately treated	Hotspot runoff must be adequately treated
Infiltration	Minimum distance to water table of 2 feet	Maximum slope of 6% Trenches must have flat bottom	Infiltration rate > 0.5 inch/hr	Should be used in treatment train with another control to provided CP	SD from well and water table No hotspot runoff	SC from bedrock and water table Pretreat runoff
Chemical Treatment						
Detention	Providing drain can be problematic					