CHAPTER

STORAGE FACILITIES 6

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6.1 Introduction

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. This chapter provides general design criteria for detention/retention storage basins. Detention facilities are those storage facilities 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. For information on infiltration systems, which also provide for runoff storage and control, the reader if referred to Chapter 8 of this manual on Water Quality Best Management Practices.

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 which could decrease or increase flood peaks in different downstream locations.

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 this occurs in this chapter, the symbol will be defined where it occurs in the text 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
t	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, Vs	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_i	Duration of basin inflow	hrs
t_p	Time to peak	hrs
S, V_S	Storage volume	ft^3
W	Width of basin	ft
Z	Side slope factor	-

6.3 Design Criteria

6.3.1 General Criteria

An analysis of 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 existing- and post-development hydrology.

6.3.2 Release Rate

A combination of storage and controlled release of stormwater runoff shall be required for all development and construction which will increase the peak rate of runoff from the site by more than one cubic foot per second for a 10-year frequency storm event. The peak release rate of stormwater from all developments where detention is required shall not exceed ninety (90) percent of the peak stormwater runoff rate from the area in its natural undeveloped state for all intensities from the 2-year up to and including the 100-year frequency. Note: If runoff is controlled through a water quality BMP then it can be excluded from the storage control requirements.

6.3.3 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to existing-developed discharge rates for all intensities up to and including the 100-year frequency. 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 flood control volume shall be drained within 48 hours.

6.3.4 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.

6.3.4.1 General

The construction of storage facilities usually require 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. Steeper slopes may be approved by the County in some locations. 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 slope stability evaluations can be found in most soil engineering textbooks, including those by Spangler and Handy (1982) and Sowers (1970).

A minimum freeboard of 1.5 feet above the 100-year design storm high water elevation shall be provided for dam heights of less than 20 feet. Impoundment depths greater than 25 feet or a storage volume greater than 100 acre-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.

6.3.4.2 Detention

Areas above the normal high water elevations of storage facilities should be sloped 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 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. Often a sediment collection forebay is provided with easy maintenance access.

6.3.4.3 Retention

The maximum depth of permanent storage facilities will be determined by site conditions, ground water elevation, 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 4 to 8 feet is generally reasonable unless fishery requirements dictate otherwise. Aeration may be required in permanent 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.

6.3.5 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. It should be noted that small outlets that will be subject to clogging or be difficult to maintain may not be acceptable to DeKalb County. 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, using fully developed land use conditions, with the outlet located at or above the 25-year ponding elevation. The sizing of the outlet works shall be based on results of hydrologic routing calculations.

6.3.6 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. The following procedure is recommended to determine downstream effects. For all storage facilities, channel routing calculations shall 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 hydrographs so as to increase peak flows or flooding conditions.

6.3.7 Safe Dams Act

Under the dam safety act regulations a dam is an artificial barrier that does or may impound water and that is 25 feet or greater in height or has a maximum storage volume of 100 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

6.4.1 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 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. Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming especially when several different designs are evaluated. Standard textbooks on Hydrology and Hydraulics give examples of hand-routing techniques (See Debo & Reese, 1995). For this Manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given in this chapter. Procedures for preliminary detention calculations will be given in this chapter since they provide simple procedures that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs.

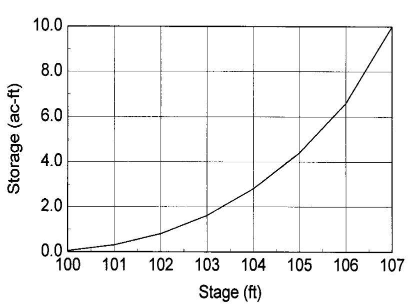


Figure 6-1

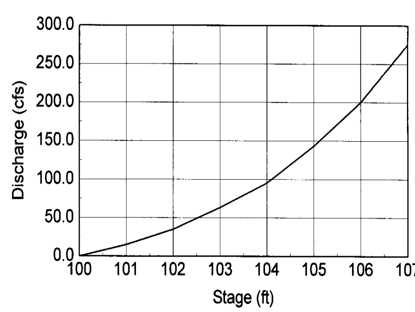


Figure 6-2

Example Stage-Storage Curve **Example Stage-Discharge Curve**

6.4.2 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 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:

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

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 prismoidal 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:

$$\mathbf{V = 1.047\ D\ (R_1^2 + R_2^2 + R^1R_2)} \qquad \textbf{(6.4)}$$

$$\mathbf{V = 1.047\ D\ (3R_1^2 + 3ZDR_1 + Z_2D^2)} \qquad \textbf{(6.5)}$$

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

6.4.3 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 spillways: principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design floods without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or 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.

6.4.4 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 2- through the 100-year design storms using the procedures outlined in the Hydrology Chapter of this manual. Both existing- and post-development hydrographs are required.

Step 2 - Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Section 6.6).

Step 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.

Step 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.

Step 5 - Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges from the 2- through the 100-year design storms exceed 90% of the existing-development peak discharges, then revise the available storage volume, outlet device, etc., and return to step 3.

Step 6 - Evaluate the downstream effects of detention outflows for the design storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system until a confluence point is reached where the drainage area being analyzed represents 10 percent of the total drainage area.

Step 7 - Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

6.5 Outlet Hydraulics

6.5.1 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.

6.5.2 Sharp-Crested Weirs

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

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

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

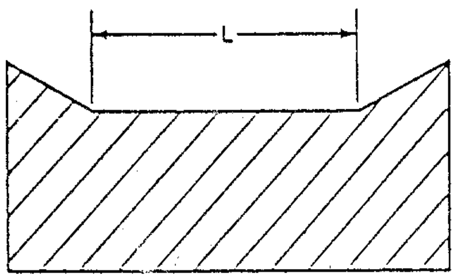


Figure 6-3
Sharp-Crested Weir
No End Contractions

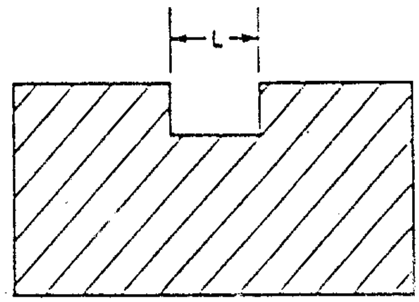


Figure 6-4
Sharp-Crested Weir
And Head

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

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

Where: Variables are the same as equation 6.4.

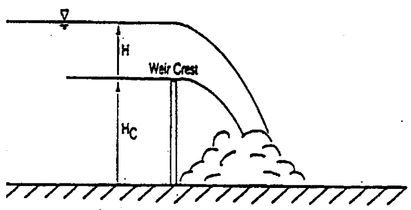


Figure 6-5 Sharp-Crested Weir, Two End Contractions

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):

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

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

6.5.3 Broad-Crested Weirs

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

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

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.

6.5.4 V-Notch Weirs

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

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

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

6.5.5 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-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. Design equations for proportional weirs are (Sandvik, 1985):

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

$$x/b = 1 - (1/3.17)\ (\arctan\ (y/a)^{0.5}) \qquad (6.12)$$

Where: Q = discharge, cfs
Dimensions a, b, h, x, and y are shown in Figure 6-6

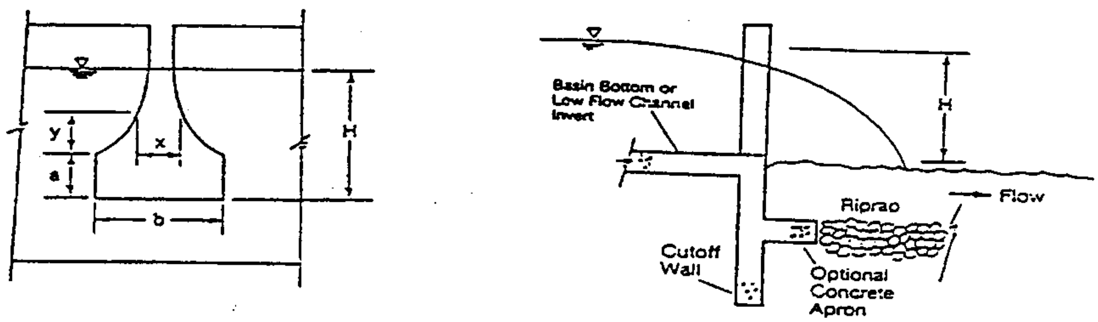


Figure 6-6 Proportional Weir Dimensions

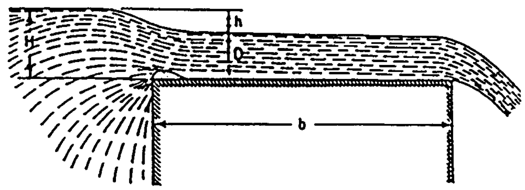
6.5.6 Orifices

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

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

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

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



Measured Head, H[1] (ft)	Weir Crect Breadth (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	2.34	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 of the weir.

Reference: Brater and King (1976).

6.5.7 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 which 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 Storm Water Management by Debo and Reese.

6.6 Preliminary Detention Calculations

6.6.1 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 Figure 6-7 on the next page.

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

$$\mathbf{V_s = 0.5t_b(Q_i - Q_o)} \qquad \textbf{(6.14)}$$

Where: V_s = storage volume estimate, ft^3
Q_i = peak inflow rate, cfs
Q_o = peak outflow rate, cfs
t_b = duration of basin inflow, sec

Any consistent units may be used for Equation 6.14.

6.6.2 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.

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

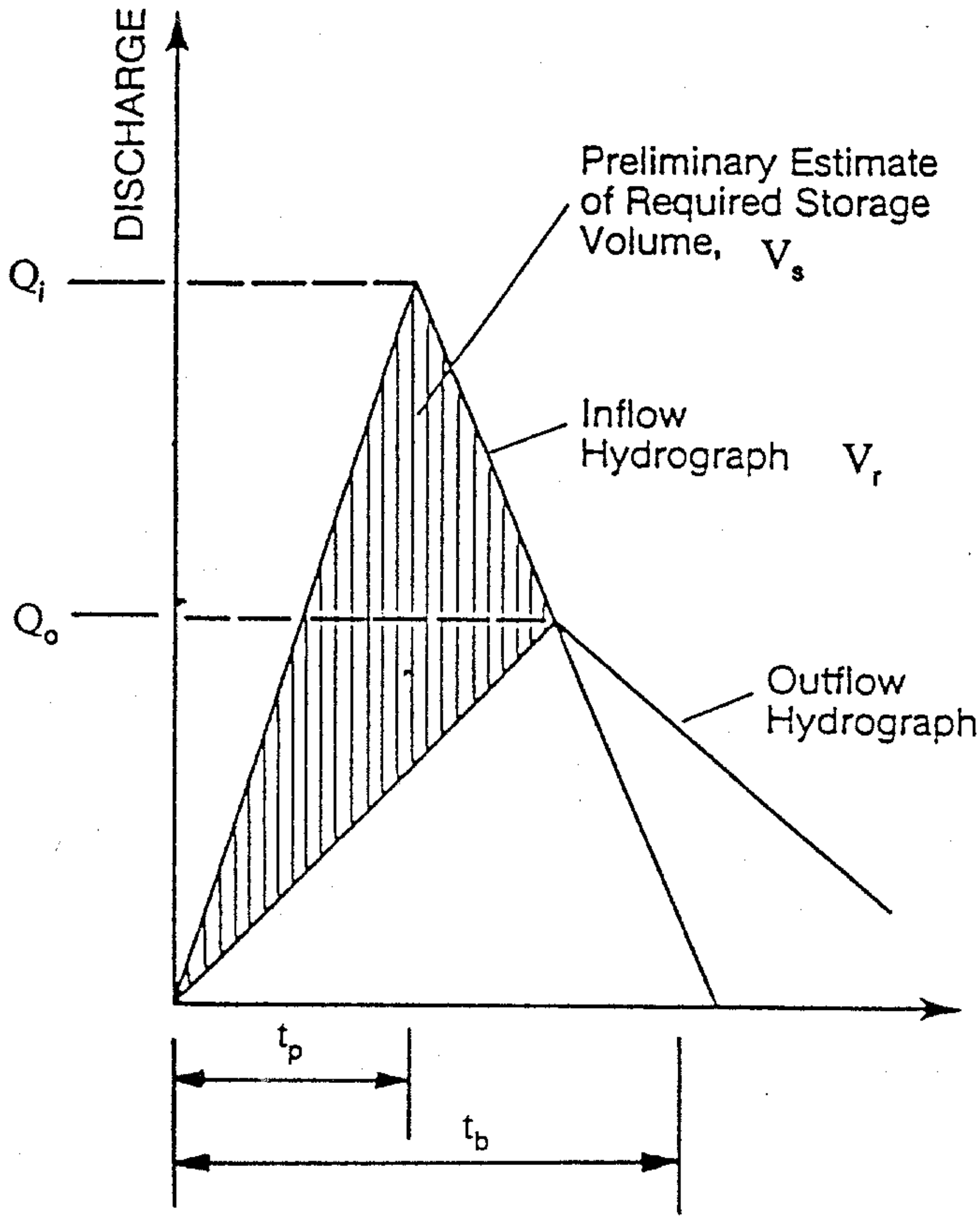


Figure 6-7 Triangular Shaped Hydrographs
(For Preliminary Estimate Of Required Storage Volume)

$$V_s/V_r = [1.291(1-Q_o/Q_i)^{0.753}]/[(t_b/t_p)^{0.411}] \quad (6.15)$$

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 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, hr

3. Multiply the volume of runoff, V_r, times the ratio V_s/V_r, calculated in Step 2 to obtain the estimated storage volume V_s.

6.6.3 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_i, 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):

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

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

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country including the southeast. Thus the methods like the Modified Rational Formula for storage design is not recommended for design calculations in DeKalb County.

Using the procedures given in the Hydrology Chapter of this manual and this chapter, the designer should develop an inflow hydrograph for the design storms, stage-discharge curve, and stage-storage curve for the proposed storage facility. Then using a storage routing computer program, proceed with the design of the storage facility to determine the required storage volume and characteristics of the outlet device(s) that will be needed to control the 2- through the 100-year storms.

6.8 Trash Racks And Safety Grates

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 them to climb to safety; and
- well designed trash racks can have an aesthetically pleasing appearance.

When designed well trash racks serve these purposes 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 about 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 6-8 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 headlosses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses.

Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50 percent is chosen as a working assumption.

$$\mathbf{H_g = K_{g_1} (w/x)^{4/3} (V_u^2/2g) \sin \theta_g} \qquad \textbf{(6.17)}$$

Where:
- H_g =head loss through grate, ft
- K_{g_1}=bar shape factor:
  - 2.42 - sharp edged rectangular
  - 1.83 - rectangular bars with semicircular upstream faces
  - 1.79 - circular bars
  - 1.67 - rectangular bars with semicircular up- and downstream faces
- w =maximum cross sectional bar width facing the flow, in.
- x =minimum clear spacing between bars, in.
- V_u =approach velocity, ft/s
- θ =angle of the grate with respect to the horizontal, degrees

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$\mathbf{H_g = K_{g_2} V_u^2/2g} \qquad \textbf{(6.18)}$$

Where K_{g_2} is defined from a series of fit curves as:

•sharp edged rectangular (length/thickness = 10)

$K_{g_2} = 0.00158 - 0.03217 A_r + 7.1786 A_r^2$

•sharp edged rectangular (length/thickness = 5)

$K_{g_2} = -0.00731 + 0.69453 A_r + 7.0856 A_r^2$

•round edged rectangular (length/thickness = 10.9)

$K_{g_2} = -0.00101 + 0.02520 A_r + 6.0000 A_r^2$

•circular cross section

$K_{g_2} = 0.00866 + 0.13589 A_r + 6.0357 A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

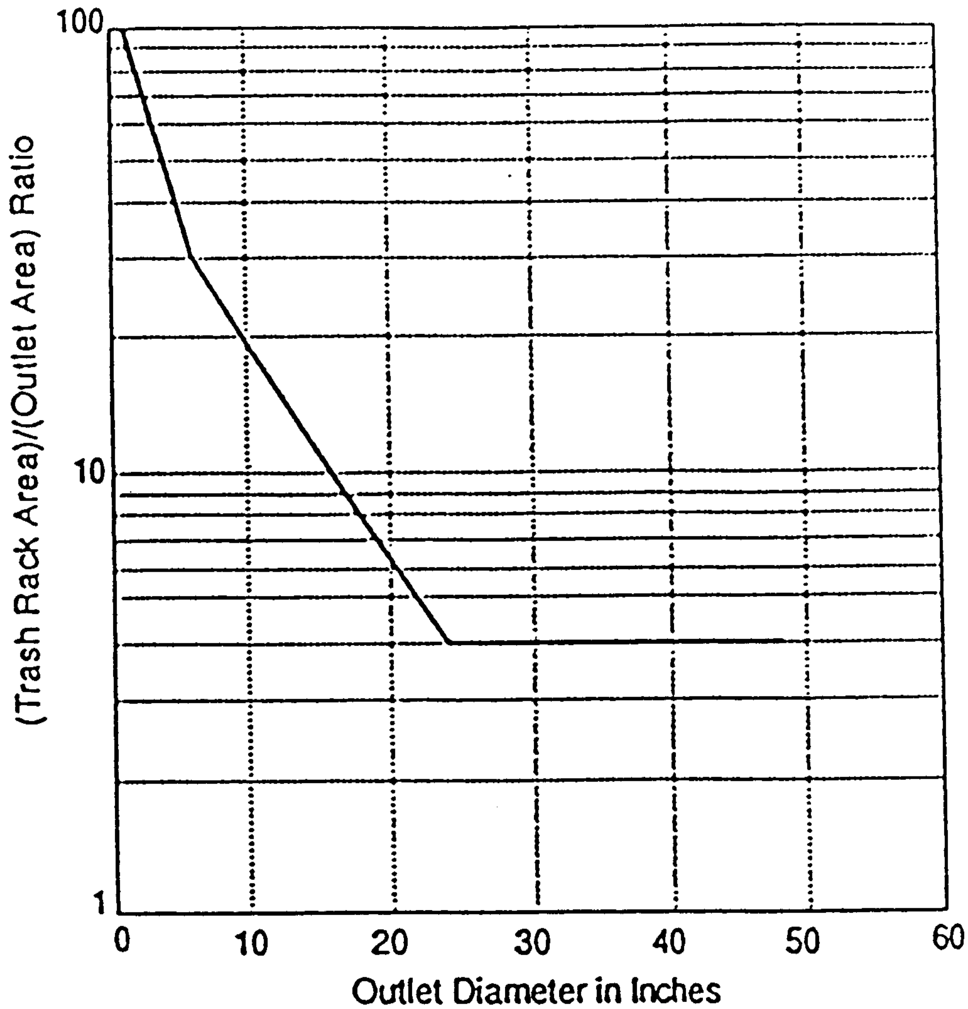


Figure 6-8 Minimum Rack Size vs. Outlet Diameter (UDCFD, 1992)

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