Omaha Regional Stormwater Design Manual

Storage Facilities

Chapter 6

Revised June 2014

City of Omaha Environmental Quality Control Division
www.omahastormwater.org

Table of Contents

Appendices

List of Tables

List of Figures

Chapter 6 Storage Facilities

6.1 Introduction

6.1.1 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 landowners. The temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. 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. This Chapter pertains to permanent detention and retention facilities. In general, this Chapter does not pertain to temporary storage facilities to control construction sediment and erosion during initial building phases. For those, see Chapter 9.

6.1.2 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, which could decrease or increase flood peaks in different downstream locations. Thus, it is important for the engineer to design storage facilities as drainage structures that both control runoff from a defined area and interact with other drainage structures within the drainage basin. Effective stormwater management should be coordinated on a regional, or basin-wide, planning basis.

6.1.3 Detention And Retention

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For the purposes 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 a storm event has passed. See Figure 6-1. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this manual. Retention facilities are designed to contain a permanent pool of water. See Figure 6-2. 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.

Although detention and retention facilities are similar in calculation, the initial cost, long term maintenance, and ownership details of each may be significantly different.

6.1.4 Computer Programs

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations, there are many available reservoir routing computer programs. Also, the storage indication method can be used, which simplifies calculations. All storage facilities shall be designed and analyzed using reservoir routing calculations. Watershed routing for storage facilities shall be performed manually using the procedures outlined in this Chapter, or using HEC-HMS. If prior written approval is provided by the Director of Public Works, other appropriate modeling software may be used for storage design.

6.1.5 Plan Review

- Detention or retention storage construction plans shall be submitted by the owner to the Nebraska Department of Natural Resources for approval, or shall be certified by the owner that Nebraska Department of Natural Resources approval is not required.

- Supporting calculations for hydrologic and hydraulic analysis and design shall be submitted by the owner to the Public Works Department for review and approval. As a minimum, supporting calculations shall include design storm inflow and outflow hydrographs; stage-storage-discharge curves; and cumulative inflow-outflow elevation curves for the design storms.

- Soils investigation documentation shall be provided (i.e., suitability for water storage, settlement potential, slope stability, and influence of groundwater) that is appropriate for the structure hazard classification.

- Construction plans for detention or retention storage, including the outlet structure, shall be submitted by the owner to the Public Works Department for review and approval.

- The owner shall provide, at the end of construction, a separate written statement prepared by a licensed surveyor or engineer to the Director of Public Works that the grading and construction of storage facilities has been completed in conformance with the approved construction plans.

6.1.6 Maintenance of Storage Facilities

Failure to provide proper maintenance reduces both hydraulic capacity and pollutant removal efficiency. Effective maintenance relies on clear assignment of tasks and a regular inspection schedule. Provision of maintenance for detention/retention facilities will be provided consistent with the policies established in the Papillion Creek Watershed Partnership, Watershed Master Plan, 2009, in compliance with the applicable local codes and regulations, and implemented through advance formal agreements between the entities with jurisdiction or responsibility.

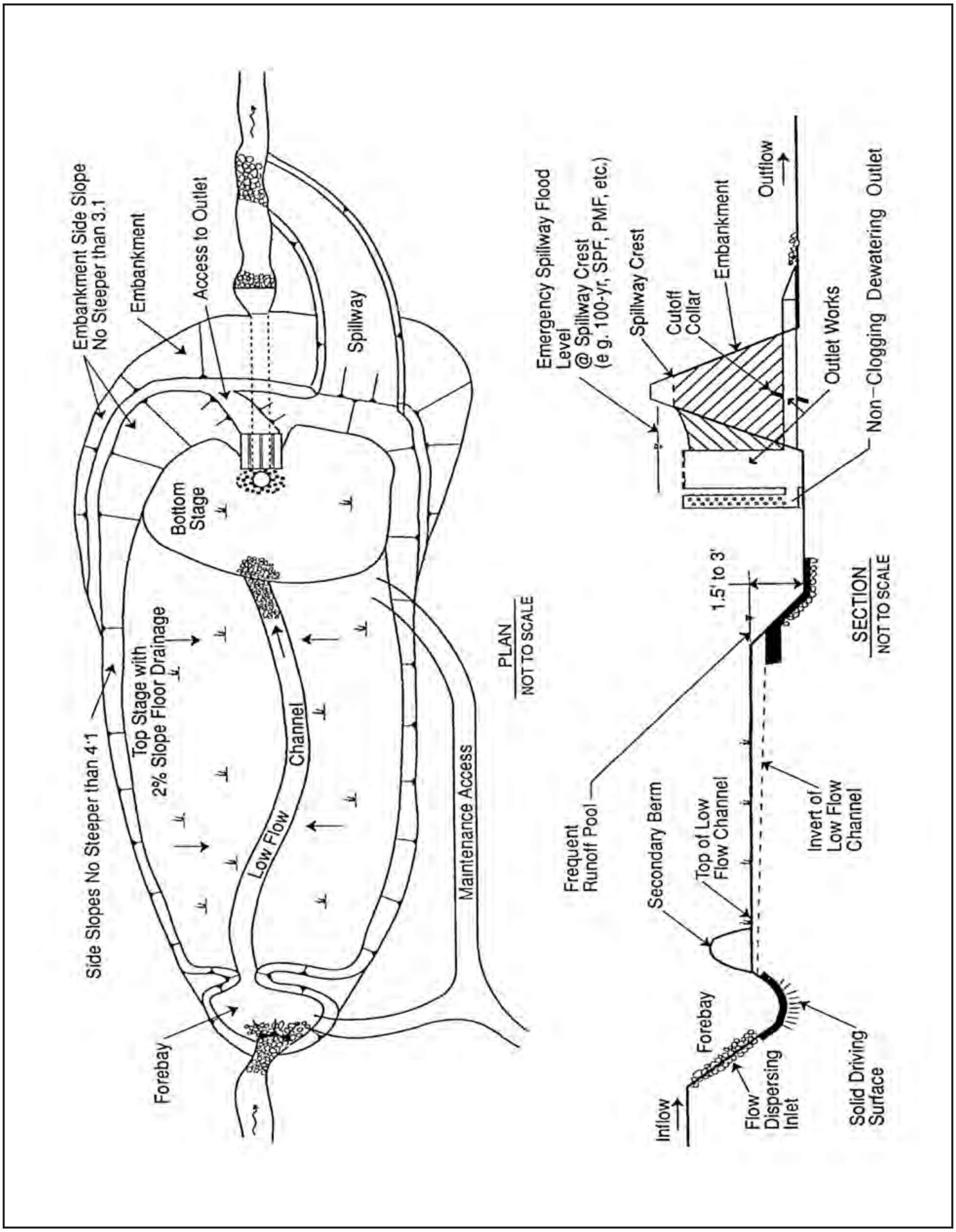


Figure 6-1 Typical Dry Detention Stormwater Storage

Source: Adapted from Denver Urban Drainage and Flood Control District, 1992

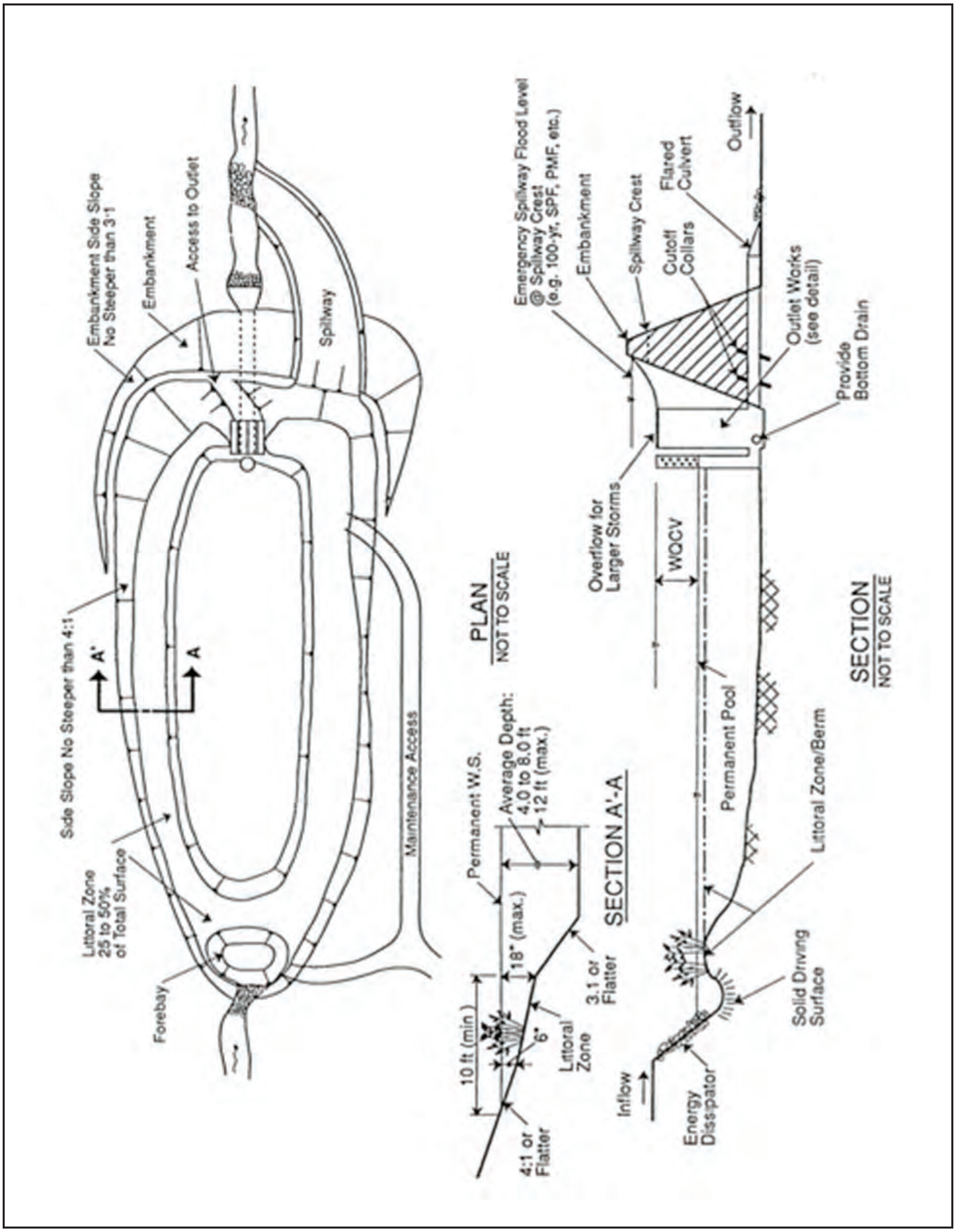


Figure 6-2 Typical Retention (Wet) Stormwater Storage

Source: Adapted from Denver Urban Drainage and Flood Control District, 1992

6.2 Uses

6.2.1 Introduction

The use of storage facilities for stormwater management has become a common practice in recent years. The benefits of storage facilities can be divided into two major control categories of quality and quantity.

6.2.2 Quality

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

- decreased downstream channel erosion (with proper design) through velocity control and flow reduction,
- reduced pollution loading through deposition, chemical reaction and biological uptake mechanisms,
- aesthetic and ecological habitat benefits at multi-objective sites,
- control of sediment deposition, and
- improved water quality through stormwater filtration.

6.2.3 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.

6.3 Symbols and Definitions

To provide consistency, 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 appendix, the symbol will be defined where it occurs in the text or equations.

Table 6-1 Symbols, Definitions and Units

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.
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, O	Flow or outflow rate	cfs
S, Vs	Storage volume	$ft.^3$, ac.-ft.
t	Routing time period	s.
t_b	Time base on hydrograph	hr.
T_c	Time of concentration	hr.
T_i	Duration of basin inflow	hr.
t_p	Time to peak	hr.
V_S, S	Storage volume	$ft.^3$
W	Width of basin	ft.
z	Side slope factor	—

6.4 Design Criteria

6.4.1 General Criteria

Storage may be concentrated in large basin-wide (or regional) facilities or distributed throughout an urban drainage system. Storage may be developed in depressed areas in parking lots, behind road embankments, freeway interchanges, parks and other recreation areas, and small lakes, ponds and depressions within urban 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 shall 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 reasonably be expected to pass through the storage facility shall be included in the analysis. The design criteria for storage facilities shall include the following:

- release rate,
- storage volume,
- grading and depth requirements,
- safety considerations and landscaping,
- outlet works and location,
- provisions for efficient maintenance.

6.4.2 Release Rates

As a minimum, full build-out peak discharge rates should be computed for the 2-, 10-, and 100-year discharges at the project property line, and in accordance with Section 6.4.6 unless waived by the Director of Public Works. Spillway and drawdown release rates must also comply with applicable permitting and dam safety regulations of the Nebraska Department of Natural Resources (NDNR). If a facility is intended to provide water quality improvement, additional release rate control for small stream events should be included in the design.

6.4.3 Storage

Storage volume shall be adequate to provide attenuation of peak discharge rates consistent with the policies established in the Papillion Creek Watershed Partnership, Watershed Master Plan, 2009. Routing calculations must be used to demonstrate that the storage volume is adequate. Storage volume shall allow for the sediment load anticipated from the contributing watershed. Sediment accumulation design time shall be consistent with the policies established by the Papillion Creek Watershed Partnership, Watershed Master Plan, 2009, or applicable requirements of NDNR dam safety regulations. Proper implementation of site erosion and sediment measures will greatly reduce the sediment load. If sedimentation during construction causes loss of detention volume, or long-term sediment storage volume, design dimensions shall be restored before completion of the project. For storage facilities, temporarily stored runoff should drain down within 72-hrs.

6.4.4 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.

6.4.4.1 General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Dams shall be designed as per the applicable Department of Natural Resources requirements. Specific City of Omaha requirements are that vegetated embankments of storage facilities shall have side slopes no steeper than 4:1 (horizontal to vertical), unless approved in writing by the Director of Public Works; that the top width of any embankment shall be no narrower than 14 ft.; and that traversable vehicular access for maintenance purposes shall be provided from public right-of-way.

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. New development shall be designed so the lowest opening of adjacent new buildings should be is one ft. above the 100-year flood elevation or 1 ft. above the auxiliary spillway elevation, whichever is greater. Inclusion of public safety features and aesthetically pleasing features is also important for storage facilities in urbanizing areas.

6.4.4.2 Detention

Areas above the normal high-water elevations of storage facilities shall slope at a minimum of 2% toward the facilities to provide effective drainage. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities shall be graded toward the outlet to prevent standing water conditions. A minimum 2% bottom slope is required on unpaved areas. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is required to convey low flows, and prevent standing water conditions.

6.4.4.3 Retention

Retention facilities are conducive to establishment of wetland and open water habitats. Site-specific criteria relating to such things as depth, habitat, and bottom and shoreline geometry shall be selected to encourage establishment of desired habitat. Where wetland habitat is desired, vegetative and geometric conditions shall be provided to minimize the propagation of undesired vegetation. Plant and wildlife experts should be contacted for site specific guidance. If the facility provides open water conditions, a depth sufficient to discourage growth of vegetation, except along the shoreline, (without creating undue potential for anaerobic bottom conditions) shall be provided. A depth of 5 to 10 ft. is generally reasonable unless greater depth is needed for fishery purposes. Aeration may be needed in permanent pools to prevent anaerobic conditions. The maximum depth of permanent storage facilities will be determined by site conditions, design constraints, and environmental needs.

6.4.5 Outlet Works

Outlet works selected for storage facilities shall include a principal spillway and an emergency overflow. Principal spillway discharge must be released in a nonerosive manner. Outlet works can be combinations of drop inlets, pipes, weirs, orifices, chutes, and channels. Slotted-riser-pipe outlets are sometimes used, but are prone to clogging problems. Curb openings may be used as outlots for parking lot storage facilities. Storage facilities shall pass all required design storms, for full build-out conditions, using a combination of available storage and outlet works capacity, without allowing flow to enter an auxiliary (or emergency) outlet. Outlet works must operate without requiring attendance or operation. The emergency spillway crest elevation shall be set at or above the maximum water surface elevation for the 100-year design storm. Minimum freeboard of three ft. above the emergency spillway crest elevation will be necessary for embankment structures which are large enough to require review and permitting by NDNR. For storage facilities, selecting a flood magnitude for sizing the emergency outlet shall be consistent with the potential threat to downstream life and property if the facility embankment were to fail. The sizing of all outlet works shall be based on results of hydrologic and hydraulic routing calculations.

Outlet controls such as weirs are preferred to open-end pipes since they can provide control for a range of storm runoff events, including more frequent events for which control can provide water quality improvement benefits.

6.4.6 Location and Stream Analysis

Although storage facilities are designed to control the discharge at the outlet device, the discharge likely will need to be routed downstream to be sure that the downstream drainage system provides an adequate outlet for the discharge without causing drainage or flooding problems. This is particularly important where discharge from the storage facility may exceed the downstream drainage system capacity and overtop roadways, causing a hazard or property damage. 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. Evaluation of potential impacts of a facility to upstream and adjacent properties is also necessary.

If the storage facility being designed is located in a drainage basin that has a master plan, the discharge hydrographs from the outlet works should be routed downstream to the bottom of the master plan subbasin. The resulting design storm flows, with the proposed facility in place, shall be compared to Papillion Creek Watershed Partnership, Watershed Master Plan, 2009 policies to verify the development drainage and storage facility plan is acceptable. If the resulting peak flows are not acceptable, the designs shall be improved to be consistent with the policies of the Papillion Creek Watershed Partnership, Watershed Master Plan, 2009.

Effective off-channel detention may be possible for some locations within floodplains through the use of hydrologic timing. In these locations, the flood peak coming down the stream does not coincide with the peak flow rate from local on-site storm runoff. Allowing the on-site water to pass downstream, before arrival of the stream peak, may be advantageous if appropriate erosion control practices and properly sized conveyance facilities are utilized. Off-channel detention may possibly then be designed and located to remain dry until overflow into them can "skim" the peak from the oncoming stream hydrograph through the use of a side-channel weir or a flow-through depression along the banks.

6.5 Safe Dams Act

6.5.1 Background

National responsibility for the promotion and coordination of dam safety lies with the Federal Emergency Management Agency (FEMA). State responsibility for the provisions of the Federal Dam Safety Act is administered by State of Nebraska Department of Natural Resources (NDNR).

Under state 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 ac.-ft. or more (including surcharge storage). A number of exemptions are allowed from the Safe Dams Act and the appropriate state office should be contacted to resolve questions. Detention or retention storage embankments which fall under the jurisdiction of the Department of Natural Resources must be designed and constructed in accordance with Safe Dam criteria, with review and permitting by NDNR. An owner proposing a detention or retention embankment shall submit to Omaha Department of Public Works, documentation of compliance with NDNR review and permitting requirements, or documentation why the embankment does not fall under NDNR jurisdiction.

6.5.2 Classification

Dams are classified as either new or existing, by hazard potential, and by size. The State of Nebraska Department of Natural Resources classifies dams under the Rules for Surface Water (August 1995), Title 457, Chapter 19 (entitled Dam Hazard Class). These classifications are presented below.

High Hazard Dam – A dam located where failure may cause loss of life, or serious damage to homes, normally occupied industrial and commercial buildings, important public utilities, main highways, or major railroads.

Significant Hazard Dam – A dam located in areas where failure may damage isolated homes, occasionally occupied buildings, main highways, minor railroads or interrupt public utility use or service.

Low Hazard Dam – A dam located in areas where failure may damage normally unoccupied buildings, undeveloped land, or township and county roads.

6.5.3 New Dams

Detailed engineering requirements are given in the regulations for new dams. Regulations that shall be consulted for further details and engineering requirements are State of Nebraska Department of Natural Resources, Surface Water Chapter 46, Article 2 and State of Nebraska Department of Natural Resources Rules for Surface Water (August 1995), Title 457.

6.6 General Hydraulic Design Procedure

6.6.1 Data Needs

The following data will be needed to complete storage design and routing calculations for submittal to the Public Works Department.

- Inflow hydrographs for all design storms.
- Stage-storage curve for the proposed storage facility (see Figure 6-3 for an example). For sizable storage volumes use ac.-ft., cu. ft. or cu. yards may be applicable for small-site storage volumes.
- Stage-discharge curve for all outlet control structures (see Figure 6-4 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 in Section 6.9).

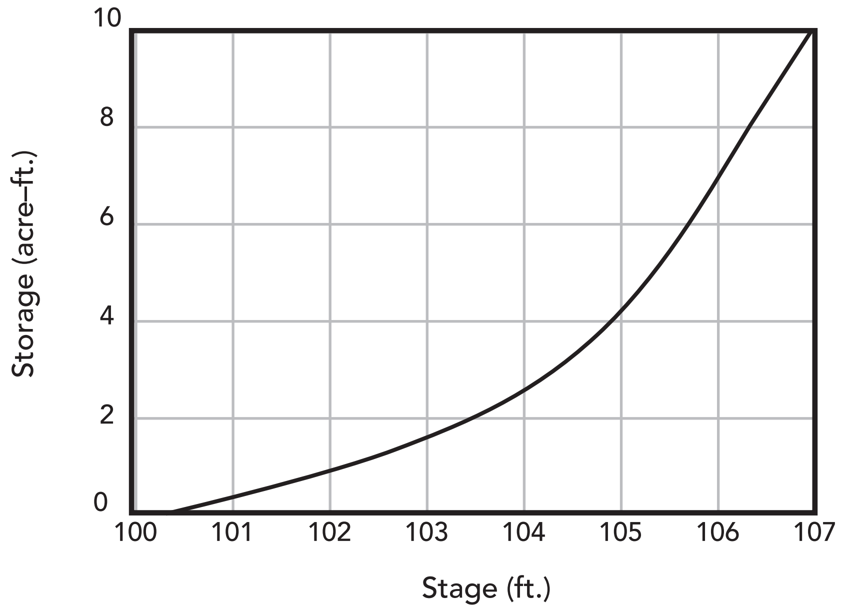


Figure 6-3 Example Stage-Storage Curve

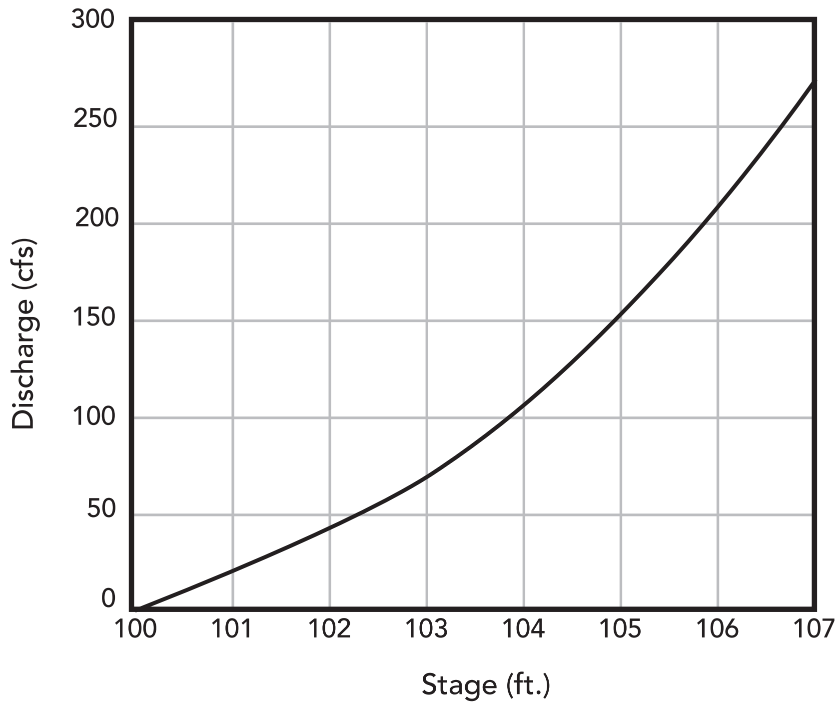


Figure 6-4 Example Stage-Discharge Curve

6.6.2 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 one of the following formulas; the average-end area, frustum (i.e., cross-sectional slice) of a pyramid, or prism. Storage basins are often irregular in shape to blend well with the surrounding terrain and to improve aesthetics. Therefore, the average-end area formula applied to elevational contour slices is usually preferred as the method to be used on non-geometric areas. The average-end area formula is expressed as:

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

Where: $V_{1,2}$ = storage volume between elevations 1 and 2, ft.^{3}
$A_{1,2}$ = 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:

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

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 prism formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)\, ZD^2 + 4/3\, Z^2 D^3 \tag{6.3}$$

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 vertical to horizontal

6.6.3 Stage-discharge Curve

Stage-discharge curves define 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. A pipe culvert, rectangular weir, v-notch weir or other appropriate outlet can be used for the principal spillway or outlet. Tailwater influences and structure losses must be considered when developing discharge curves.

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 must be designed taking into account the potential threat to downstream life and property if the storage facility were to fail. The stage-discharge curve shall take into account the discharge characteristics of both the principal and emergency spillways.

6.6.4 Design Procedure

A general procedure for the design of storage facilities follows.

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

Step 2: Perform preliminary calculations to estimate detention storage requirements for the hydrographs from Step 1 (see Section 6.8). If storage requirements are satisfied for runoff from the 2-, 10-, and 100-year design storms, runoff from intermediate storms is assumed to be controlled.

Step 3: Determine the physical dimensions necessary to hold the estimated storage volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 shall be used.

Step 4: Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure shall 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 the storage routing equations. If any of the routed post-development peak discharges from the 2-, 10- and 100-year design storms exceed the corresponding predevelopment peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise estimated volume and return to Step 3.

Step 6: Design for emergency overflow and established freeboard requirements.

Step 7: Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility shall be routed downstream for a minimum distance as determined by the criteria provided in Section 6.4.6.

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

This procedure can involve a significant number of reservoir routing calculation iterations to obtain appropriate design results.

6.7 Outlet Hydraulics

6.7.1 Outlets

Outlet control structures combine the hydraulic function of one or more individual control elements such as, sharp-crested weirs, broad-crested weirs, v-notch weirs, orifices, and pipes to produce integrated stage-discharge behavior required to meet the discharge limits for developments. Typical weir and orifice design equations are presented below. The designer may utilize other equations or coefficients if appropriate for a specific design.

If culverts are used as outlets works, procedures presented in the NDOR Roadway Design Manual should be used to develop stage-discharge data. When analyzing release rates, the tailwater influence of the principal spillway conduit on the control structure (orifice and/or weirs) must be considered to determine the effective head on each opening. Slotted riser pipe facilities, as a sole or primary storage outlet, shall be avoided.

6.7.2 Sharp-crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure 6-5. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [3.27 + 0.4(H/H_C)]\ LH^{1.5}} \qquad \mathbf{(6.4)}$$

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

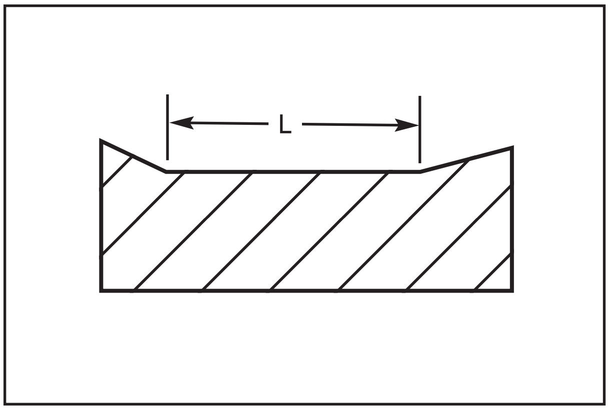


Figure 6-5 Sharp-crested Weir (No End Contractions)

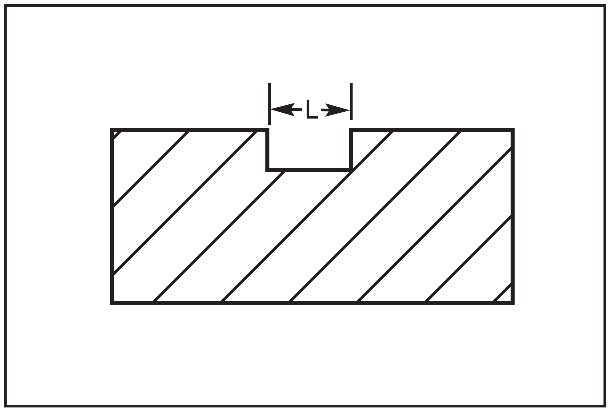


Figure 6-6 Sharp-crested Weir (Two End Contractions)

A sharp-crested weir with two end contractions is illustrated in Figure 6-6. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [3.27 + 0.4(H/H_C)]\ (L - 0.2H)\ H^{1.5}} \qquad \mathbf{(6.5)}$$

Where: Variables are the same as equation 6.4.

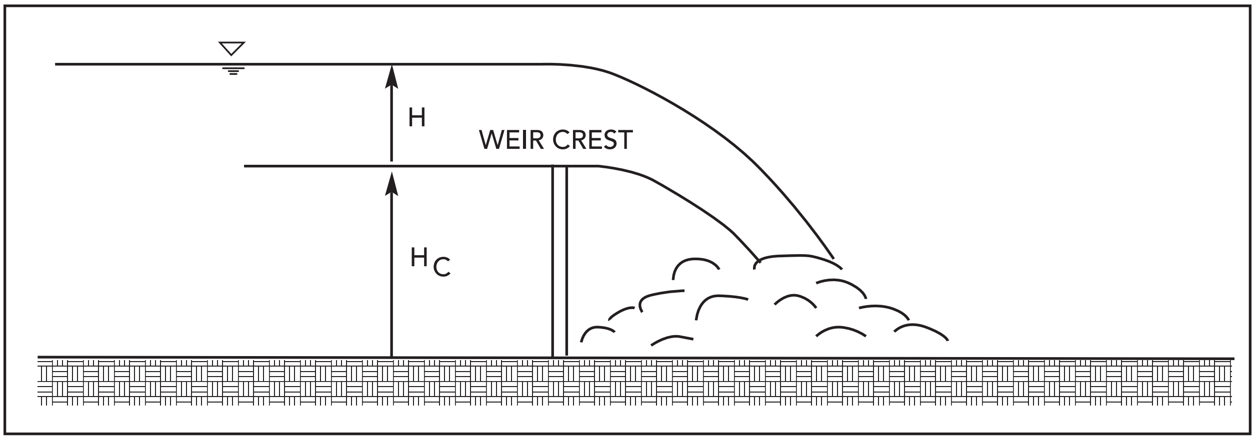


Figure 6-7 Sharp-crested Weir And Head

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation (see Figure 6-7). 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):

$$Q_s = Q_f(1 - (H_2/H_1)^{1.5})^{0.385} \tag{6.6}$$

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.

6.7.3 Broad-crested Weirs

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

$$Q = CLH^{1.5} \tag{6.7}$$

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

Information on C values as a function of weir crest breadth and head is given in Table 6-2.

6.7.4 V-Notch Weirs

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

$$Q = 2.5 \tan(\theta/2)H^{2.5} \tag{6.8}$$

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

6.7.5 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 (example shown in Figure 6-8). Refer to “Proportional Weirs for Stormwater Pond Outlets” (Sandvik, 1985) for a full discussion.

Design equations for a proportional weir, as developed by Sandvik, 1985, are :

$$\mathbf{Q = 4.97\, a^{0.5}\, b(H - a/3)} \qquad \mathbf{(6.9)}$$

$$\mathbf{x/b = 1 - (1/3.17)\,(\arctan\,(y/a)^{0.5})} \qquad \mathbf{(6.10)}$$

Where: Q = discharge, cfs

Dimensions a, b, h, x and y are shown below

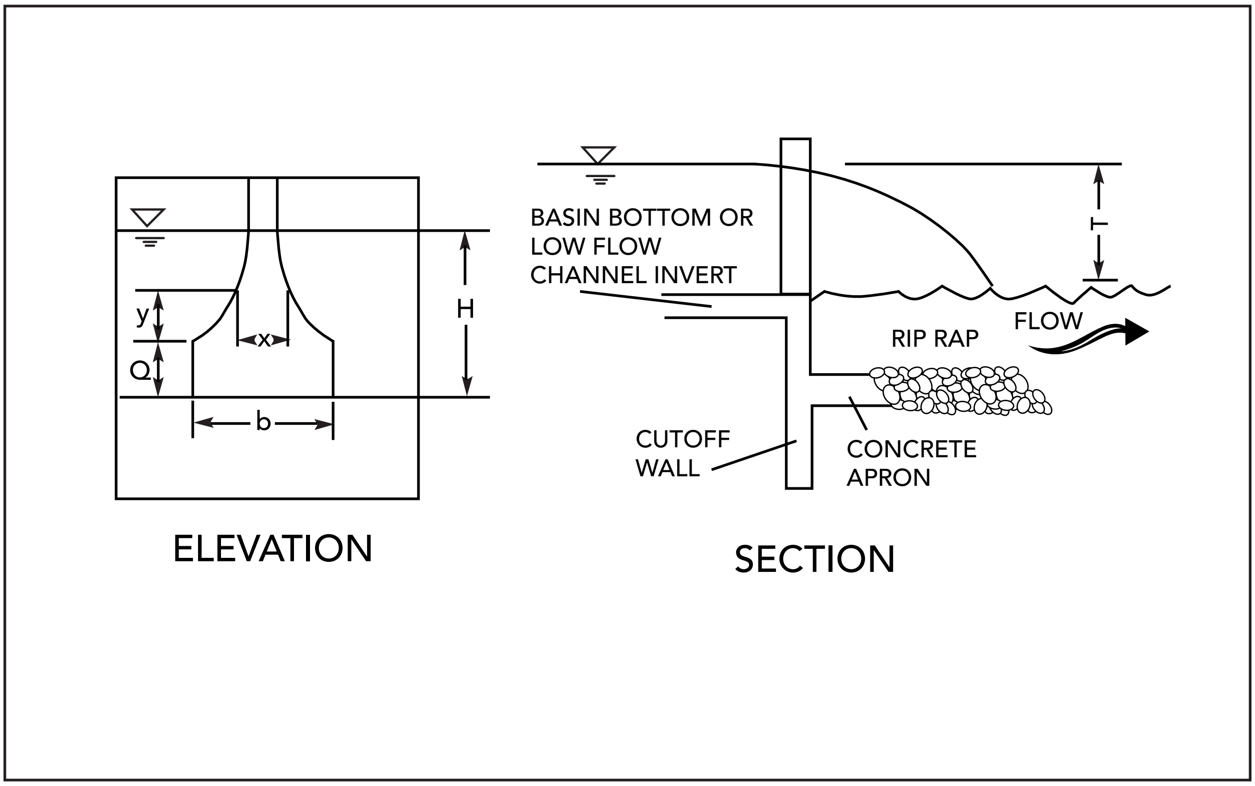


Figure 6-8 Proportional Weir Dimensions

6.7.6 Orifices

Pipes smaller than 12 in. may be analyzed as a submerged orifice if H/D is greater than 1.5.

$$Q = 0.6A(2gH)^{0.5} = 3.78D^2H^{0.5} \quad \text{(typical for square-edged entrance conditions)} \qquad (6.11)$$

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, ft. *

* In cases where the tailwater is higher than the center of the opening, the head is calculated as the difference in water surface elevations.

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

Measured Head, H[1]	Breadth Of Crest Of Weir (ft.)										
(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	2.72	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).

6.8 Preliminary Detention Calculations

6.8.1 Estimating Storage Volume

When a detention basin is installed, hydrologic routing procedures can be used to estimate the effect on hydrographs. HEC-HMS, DAMS2 (SCS 1982), and SITES (USDA 2002) computer programs provide accurate methods of analysis. It is recommended that one of these programs be used for routing hydrographs through storage facilities.

This chapter contains a manual method for quick estimates of the effects of temporary detention on peak discharges. The method is based on average storage and routing effects for many structures. Figure 6-9a relates two ratios: peak outflow to peak inflow discharge (q_o/q_i) and storage volume runoff volume (Vs/Vr) for all rainfall distributions.

The relationships in Figure 6-9a were determined on the basis of single stage outflow devices. Some were controlled by pipe flow, others by weir flow. Verification runs were made using multiple stage outflow devices, and the variance was similar to that in the base data. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that (1) each stage requires a design storm and a computation of the storage required for it and (2) the discharge of the upper stage(s) includes the discharge of the lower stage(s).

Use Figure 6-9a to estimate storage volume (Vs) required or peak outflow discharge (q_o). The most frequent application is to estimate Vs, for which the required inputs are runoff volume (Vr), q_o, and peak inflow discharge (q). To estimate q_o, the required inputs are Vr, Vs, and q_i.

Estimating Vs

Use the worksheet shown on Figure 6-9b to estimate Vs, storage volume required, by the following procedure.

1. Determine q_o. Many factors may dictate the selection of peak outflow discharge. The most common is to limit downstream discharges to a desired level, such as predevelopment discharge.

2. Estimate q_i by procedures in Chapter 2, Section 2.6. Do not use peak discharges developed by other procedures.

3. Compute q_o/q_i and determine Vs/Vr from Figure 6-9a.

4. Q (in inches) was determined when computing qi in step 2, but now it must be converted to the units in which Vs is to be expressed-most likely, ac.-ft.

5. Use the results of steps 3 to 4 to compute Vs: Vs = Vr(Vs/Vr) where Vs = storage volume required (ac.-ft.).

6. The stage in the detention basin corresponding to Vs must be equal to the stage used to generate q_o. In most situations a minor modification of the outflow device can be made. If the device has been preselected. repeat the calculations with a modified q_o value.

Limitations

This routing method is less accurate as the q_o/q_i ration approaches the limits shown in Figure 6-9a. The curve in Figure 6-9a depends on the relationship between available storage, outflow device, inflow volume, and shape of the inflow hydrograph. When storage volume (Vs) required is small, the shape of the outflow hydrograph is sensitive to the rate of the inflow hydrograph. Conversely, when Vs is large, the inflow hydrograph shape has little effect on the outflow hydrograph. In such instances, the outflow hydrograph is controlled by the hydraulics of the outflow device and the procedure therefore yields consistent results. When the peak outflow discharge (q_o) approaches the peak flow discharge (q_i), parameters that affect the rate of rise of a hydrograph, such as rainfall volume, curve number, and time of concentration, become especially significant. The procedure should not be used to perform final design if an error in storage of 25 percent cannot be tolerated. It is adequate, however, for final design of small detention basins. Figure 6-9a is biased to prevent under sizing of outflow devices, but it may significantly overestimate the required storage capacity. More detailed hydrograph development and routing will often pay for itself through reduced construction costs.

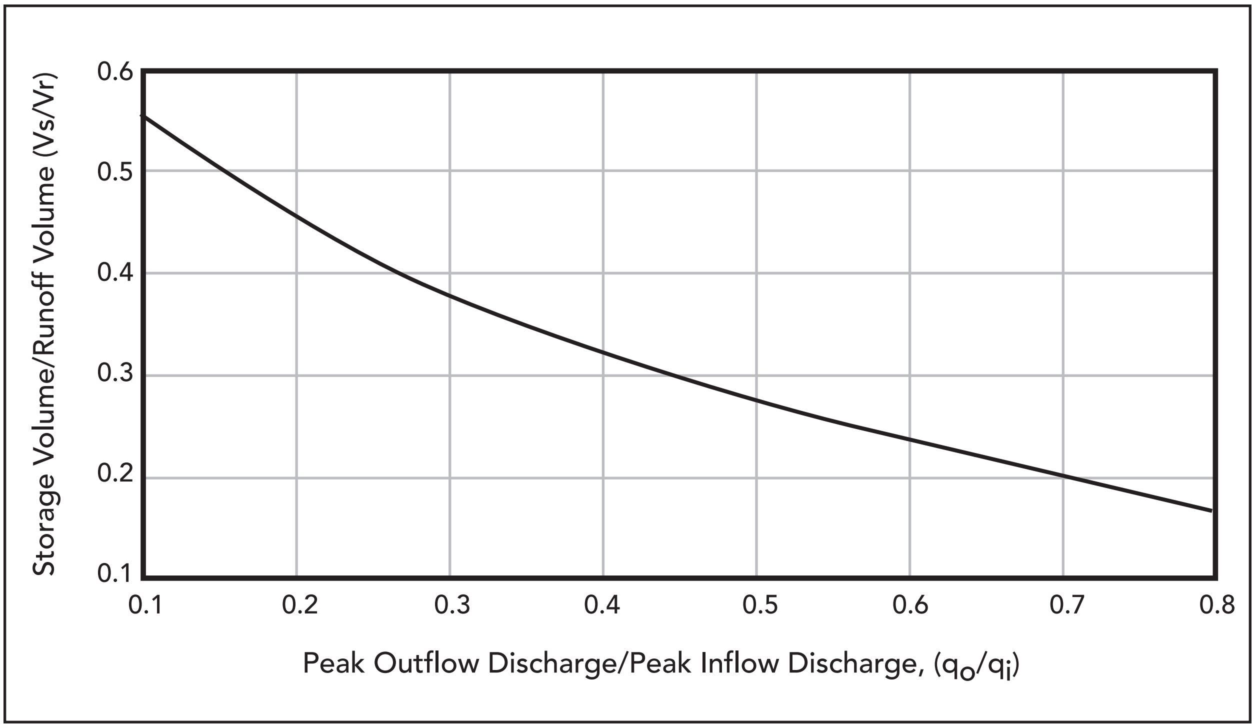


Figure 6-9a Approximate Stormwater Storage Basin Routing for Type II Rainfall Source 210-VI-TR-55, Second Ed., June 1986A

Example Estimating Vs for Two-stage structure

	2 -year		100-year	
Given:	predevelopment	= 50 cfs	predevelopment	= 180 cfs
	post development	= 91 cfs	post development	= 360 cfs
	post development runoff volume	= 1.5 in.	post development runoff volume	= 3.4 in.

A rectangular concrete weir outflow device was selected; the device could have been another type, but it is important to remember that the flows through the first stage are part of the total discharge of the higher stage.

Figure 6-9b shows how the worksheet is used to compute the Vs of 2.4 ac.-ft. and Emax of 103.6 for the stage. Emax of 103.6 is the weir crest elevation for the second stage. Equation 6.7 is used to compute Lw for the first stage. The weir crest elevation for the first stage is 100.00 ft. and q_o = 50 cfs. The first-stage computations for Hw and Lw are:

Hw = Emax - weir crest elevation = 103.6 - 100.0 = 3.6 ft.;

and, from equation 6.7, $$Lw = \frac{50}{3.2(3.6)^{1.5}} = 2.3 \text{ ft.}$$

The second stage is then proportioned to discharge correct amount at 105.7 ft.

Compute the discharge through the first stage for elevation 105.7 ft. using:

L = 2.3 ft. (first stage) and Hw = 105.7 - 100.0 = 5.7 ft.

By substituting these values in equation 6.7, discharge (q_o) through the first stage at 105.7 ft. is calculated:
$q_o = 3.2(2.3(5.7)^{1.5}) = 100$ cfs

Now compute the required weir crest length (Lw) for the second stage, using equation 6.7. Since the second stage crest elevation is 103.6 ft.,

Hw = Emax - weir crest elevation = 105.7 - 103.6 = 2.1 ft.;

and, since q_o for the second stage equals the total discharge minus discharge through the first stage,

q_o = 180 -100 = 80 cfs

Finally, substituting these Hw and q_o values in Equation 6.7 results in

$$Lw = \frac{80}{3.2(2.1)^{1.5}}$$

In summary, the outlet structure is a two-stage rectangular weir with first stage crest length of 2.3 ft. at elevation 100.0, and second stage crest length of 8.2 ft. at elevation 103.6 ft.

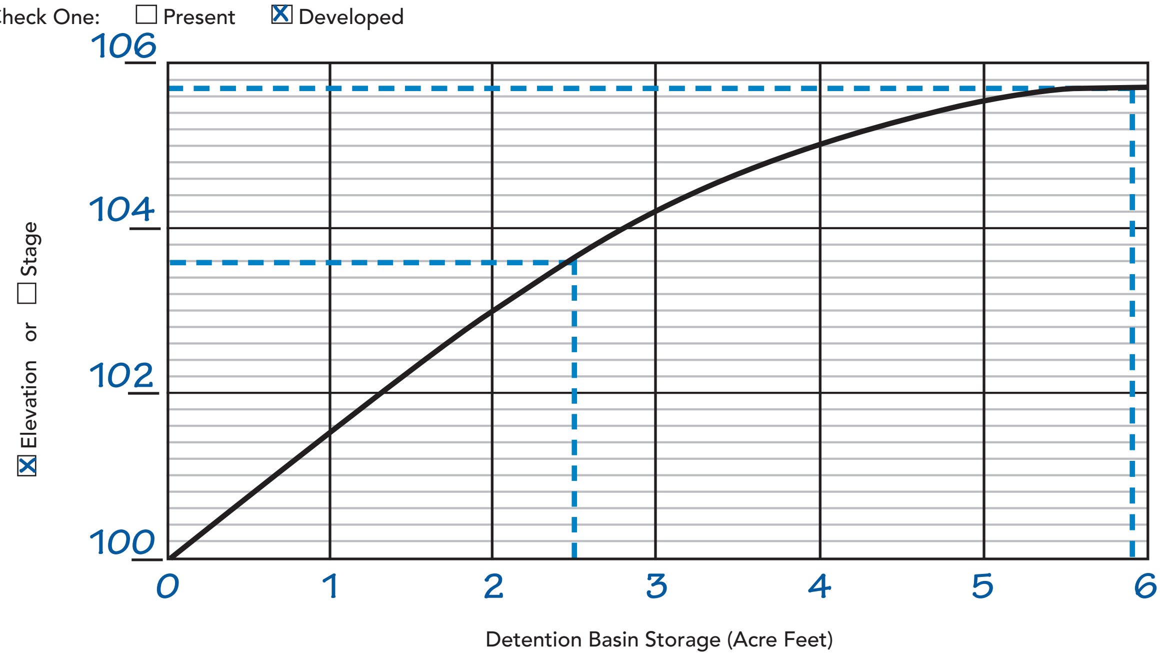
The weir equation used is probably less accurate for the two-stage example than for the single-stage example. The actual second-stage discharge will be slightly more that the one computed. This example is presented only to illustrate the interrelationship of outflow discharges and storage volume and to show how to develop preliminary estimates of storage requirements for two-the stage outlet structures.

6.8.2 Preliminary Basin Dimensions

- Plot the control structure location on a contour map.
- Select desired depth(s) of ponding for the design storm(s).
- Divide the estimated storage volume needed by the desired depth to estimate the required surface area of the reservoir.
- Based on site conditions and contours, estimate the geometric shape(s) required to provide the estimated reservoir surface area.

Project Two-Stage Structure	By ENGINEER	Date 02/22/2000
Location	Checked ENGINEER2	Date 02/22/2000

Check One: ☐ Present ☒ Developed



1. Data:
   Drainage Area............... A_m = 0.117
   Rainfall Distribution
   Type (I, IA, II, III) = II

	1st Stage	2nd Stage
2. Frequency .... yr	2	100
3. Peak Inflow Discharge q_i................. ft³/s (From Worksheet 4 or 5b)	91	360
4. Peak Outflow Discharge q_u .... ft³/s	50	180*
5. Compute $\frac{q_o}{q_i}$	0.55	0.50
6. $\frac{V_s}{V_r}$ .... (Use $\frac{q_o}{q_i}$ with Figure 6-1)	0.26	0.28
7. Runoff, Q .... in (From Worksheet 2)	1.5	3.4
8. Runoff Volume............. ac-ft ($V_r = QA_m$ 53.33)	9.4	21.2
9. Storage Volume V_s ....ac-ft ($V_s = V_s(\frac{V_s}{V_r})$)	2.4	5.9
10. Maximum Storage E_{max} (from Plot)	103.9	105.7

2nd stage q_o includes 1st stage q_o

Figure 6-9b Stormwater Storage Facility Basin Storage Volume, Peak Outflow Discharge (q_o) Known

Source 210-VI-TR-55, Second Ed., June 1986A

6.9 Routing Calculations

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

Step 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 as Figure 6-10 and Figure 6-11.

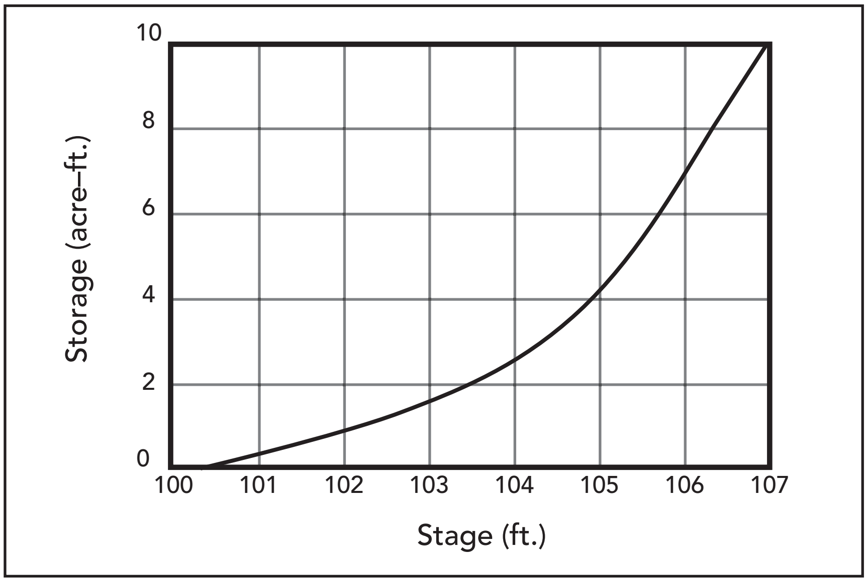


Figure 6-10 Example Stage-Storage Curve

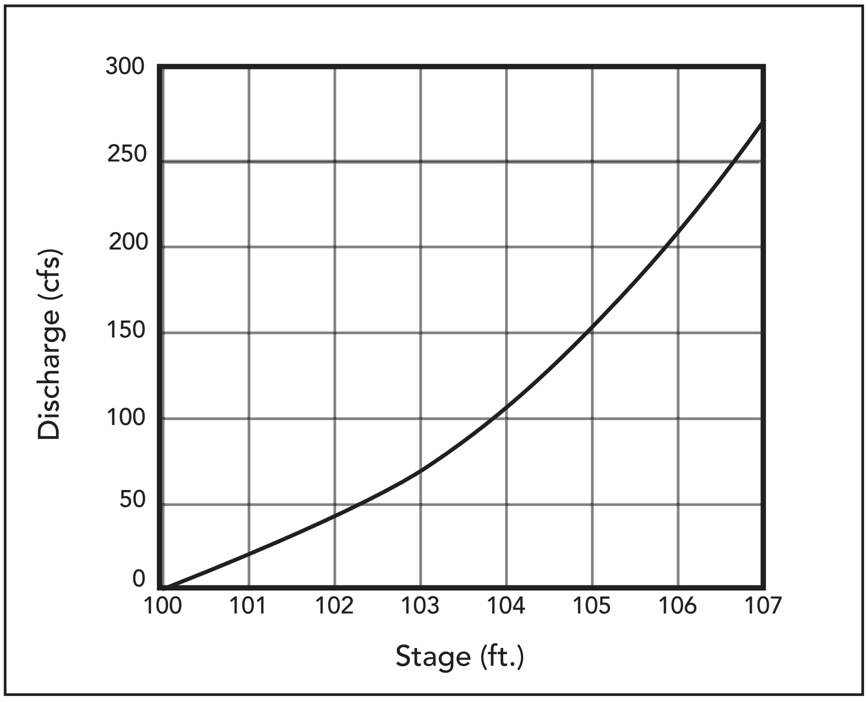


Figure 6-11 Example Stage-Discharge Curve

Step 2: Select a routing time period, dt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < Tc/5$).

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

Table 6-3 Storage Characteristics

(1) Stage (H_1) (ft.)	(2) Storage[1] (ft.³)	(cfs)	(3) Discharge (ac.-ft./hr.)	(4) S-(O/2) Δt (ft.³)	(5) S+(O/2) Δt (ft.³)
100	0.05	0	0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	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: For this example, Δt = 10 min = 0.167 hours and 1 cfs = 0.0826 ac.-ft./hr.

Step 4: 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, $S_1-(O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (Figure 6-12).

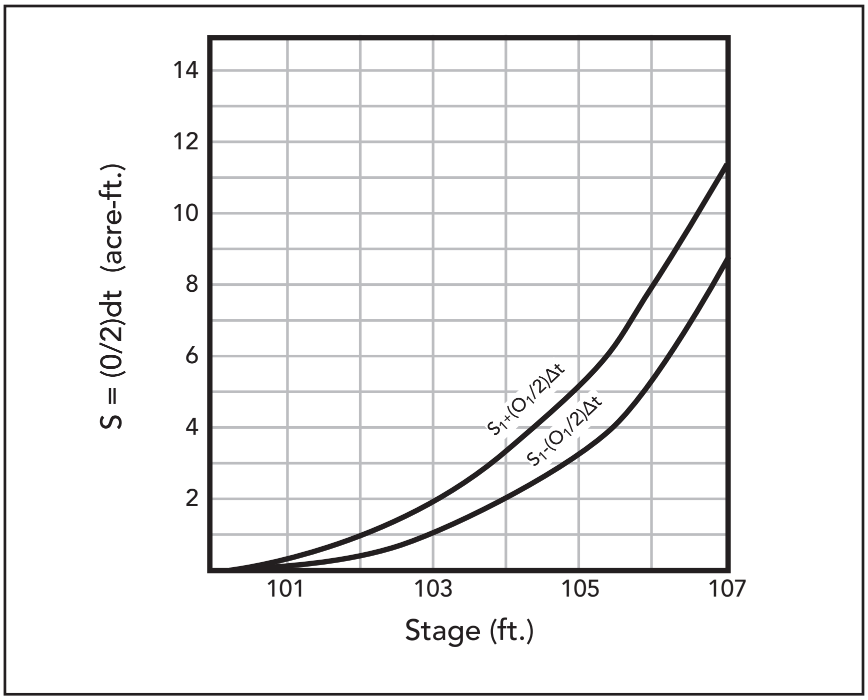


Figure 6-12 Storage Characteristic Curve

Step 5: Determine the value of $S_2 + (O_2/2)\ \Delta t$ from the following equation:

$$\mathbf{S_2 + (O_2/2)\ \Delta t = [S_1 - (O_1/2)\ \Delta t] + [(I_1 + I_2)/2]\Delta t} \qquad \mathbf{(6.15)}$$

Where:
- S_2 = storage volume at time 2, ft.^{3}
- O_2 = outflow rate at time 2, cfs
- Δt = routing time period, s.
- 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.)

Step 6: Enter the storage characteristics curve at the calculated value of $S_2+(O_2/_2)\Delta t$ determined in Step 5 and read off a new depth of water, H_2.

Step 7: Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

Step 8: Repeat Steps 1 through 7 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 has been routed through the storage basin.

There are numerous proprietary and non-proprietary (HEC-HMS, DAMS2) computer models that perform reservoir routing. However, the designer must still have a thorough understanding of hydrology and hydraulics design procedures to properly interpret the results.

6.10 Example Problem

6.10.1 Example

This example demonstrates 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 provided in SCS TR-55.

6.10.2 Design Discharge And Hydrographs

Storage facilities are to be designed for runoff from the 2-, 10- and 100-year design storms. The following example only shows the calculations for the 2- and 10-year storms. Actual designs would also include a similar analysis for the 100-year storm. Example peak discharges from the 2- and 10-year design storm events are as follows:

- Pre-development 2-year peak discharge = 150 cfs
- Pre-development 10-year peak discharge = 200 cfs
- Post-development 2-year peak discharge = 190 cfs
- Post-development 10-year 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 2- and 10-year storms, respectively.

Example runoff hydrographs are shown in Table 6-4. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hrs., respectively, for runoff from the 2- and 10-year storms.

Table 6-4 Example Runoff Hydrographs

Pre-Development Runoff			Post-Development Runoff	
(1) Time (Hrs.)	(2) 2-Year (cfs)	(3) 10-Year (cfs)	(4) 2-Year (cfs)	(5) 10-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

6.10.3 Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in Section 6.8. For runoff from the 2- and 10-year storms, the required storage volumes, V_S, are computed using equation 6.12:

$$V_S = 0.5T_i(Q_i - Q_0)$$

2-year storm: V_S = [0.5(1.2 x 3600)(190 - 150)]/43560 cu.-ft./ac.-ft. = 1.98 ac.-ft.
10-year storm: V_S = [0.5(1.25 x 3600)(250 - 200)]/43560 cu.-ft./ac.-ft. = 2.58 ac.-ft.

6.10.4 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 2- and 10-year design storms are presented in Table 6-5, below. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 2- and 10-year design storms to be provided when the corresponding allowable peak discharges occurred. Discharge 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 ft. and no tailwater submergence.

Table 6-5 Stage-Discharge-Storage Data

(1) Stage (ft.)	(2) Q (cfs)	(3) S (ac.-ft.)	(4) $S_1+(O/2)\Delta t$ (ac.-ft.)	(5) $S_1-(O/2)\Delta t$ (ac.-ft.)
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 2- and 10-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the Stage-Discharge-Storage data on Table 6-5 and the Storage Characteristics Curves given on Figures 6-10, 6-11, and 6-12, and 0.1-hr. time steps are shown in Tables 6-6 and 6-7 below for runoff from the 2- and 10-year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-year design storms.

Table 6-6 Storage Routing for the 2-Year Storm

(1) Time (Hrs.)	(2) Inflow (cfs)	(3) $[(I_1 + I_2)]/2\Delta t$ (ac.-ft.)	(4) H_1 (ft.)	(5) $S_1-(O_1/2)\Delta t$ (ac.-ft.)	(6) $S_2+(O_2/2)\Delta t$ (ac.-ft.) (3)+(5)	(7) H_2 (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
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	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.02	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.42	0.47	1.03	18
1.1	2	0.02	1.30	0.32	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.00	0.70	0.15	0.15	0.40	3

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

Table 6-7 Storage Routing for the 10-Year Storm

(1) Time (Hrs.)	(2) Inflow (cfs)	(3) $[(I_1 + I_2)]/2\Delta t$ (ac.-ft.)	(4) H_1 (ft.)	(5) S_1-$(O_1/2)\Delta t$ (ac.-ft.)	(6) S_2+$(O_2/2)\Delta t$ (ac.-ft.) (3)+(5)	(7) H_2 (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	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173<200
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	103
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 100-year storm must be routed through the storage facility to determine if the routed peak discharge is lower than the maximum allowable peak discharge. In addition, the preliminary design provides hydraulic details only. Final design shall consider site constraints such as depth to groundwater, side slope stability and maintenance, grading to prevent standing water and provisions for public safety.

6.11 Retention Storage Facilities

6.11.1 Introduction

Design of retention storage facilities must allow for performance of maintenance activities. The owners capability for performing required maintenance shall be considered. Provisions for weed control and potential future aeration for prevention of anaerobic conditions shall also be considered.

6.11.2 Water Budget

Water budget calculations are required for all permanent pool facilities and shall consider performance for average annual conditions to demonstrate that adequate runoff is available for maintenance of a permanent pool. The water budget shall 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 shall be based on site-specific soils testing data. Evaporation may be approximated using the mean monthly pan evaporation or free water surface evaporation data.

6.12 Example Problem

A shallow basin with an average surface area of 3 ac. and a bottom area of 2 ac. is planned for construction at the outlet of a 10-ac. 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 about 0.1 in./hr. Determine for average annual conditions if the facility will function as a retention facility with a permanent pool.

Solution

1. From rainfall records, the average annual rainfall is about 30 in.
2. The mean annual evaporation is 19 in.
3. The average annual runoff is estimated as:
   Runoff = (0.3) (30 in.) (100 ac.) = 900 ac.-in.
4. The average annual evaporation is estimated as:
   Evaporation = (19 in.) (3 ac.) = 57 ac.-in.
5. The average annual infiltration is estimated as:
   Infiltration = (0.1 in./hr.) (24 hrs./day) (365 days/yr.) (2 ac.)
   Infiltration = 1,752 ac.-in.
6. Neglecting basin outflow and assuming no change in storage, the runoff (or inflow) less evaporation and infiltration losses is:
   Net Budget = 900 - 57 - 1,752 = -909 ac.-in.
   Thus, the proposed facility will not function as a retention facility with a permanent pool.
7. Revise pool design as follows:
   Average surface area = 1.7 ac. and bottom area = 0.8 ac.
8. Recompute the evaporation and infiltration
   Evaporation = (19) (1.7) = 32 ac.-in.
   Infiltration = (0.1) (24) (365) (0.8) = 700 ac.-in.
9. The revised runoff less evaporation and infiltration losses is:
   Net Budget = 900 - 30 - 700 = + 162 ac.-in.

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

6.13 Construction and Maintenance Considerations

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage facilities. To provide for acceptable performance and function, storage facilities that require extensive maintenance are discouraged. However, the following maintenance considerations should be viewed generally and should not limit efforts in the creation or enhancement of wetlands, open water habitats, plantings, or other natural/conservation design techniques that can contribute positively to the aesthetic or environmental elements of storage areas, particularly retention basins. In general, facilities shall be designed to minimize maintenance problems typical of urban detention facilities such as:

- 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 focuses on elimination or reduction of maintenance requirements by addressing the potential for problems to develop.

- Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers.
- Sedimentation shall be controlled by constructing traps to contain sediment for easy removal or low-flow channels to reduce erosion and sediment transport.
- Bank deterioration can be controlled with protective lining or by limiting bank slopes.
- Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or by constructing underdrain facilities to lower water tables.
- In general, when the above problems are addressed, mosquito control will not be a major problem.
- Outlet structures shall be selected to minimize the possibility of blockage (i.e., very small pipes tend to block quite easily and shall be avoided).
- One way to deal with the maintenance associated with litter and damage to fences and perimeter plantings is to locate the facility for easy access so this maintenance can be conducted on a regular basis.
- Access easements shall be provided for heavy equipment when facilities do not abut public right-of-way Access for vehicular maintenance shall be provided to the control structure, along side(s) of the storage pond as necessary (15-ft. minimum width), and to the basin bottom for facilities with bottom widths greater than 15 ft. When a facility abuts a City right-of-way such as a local or arterial street, maintenance access from the abutting City right-of-way is an option which may be acceptable if it will not result in an unsafe or otherwise unworkable conditions.
- Retention storage, which proposes a permanent pool in addition to detention, shall be constructed to facilitate silt removal and disposal.
- An outlet shall be provided that will allow the retention facilities to be substantially drained for silt removal, maintenance, or inspection.
- Provisions shall be made for the deposit of silt removed from the stilling basin and/or the main pool.

6.14 Protective Treatment

Protective treatment may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences may be required for detention areas where one or more of the following conditions exist:

- Rapid stage increases would make escape practically impossible where small children frequent the area.
- Water depths either exceed 2.5 ft. for more than 24 hrs. or are permanently wet.
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 5 ft. or a flow velocity greater than 5 ft./s.

Guards or grates may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

Fencing should be considered for normally dry storage facilities with design depths in excess of 2.5 ft. for 24 hrs., unless the area is within a fenced, limited access facility.

6.15 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 a possibility to climb to safety.

Well-designed 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 3:1 to 5:1 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 6-13 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in areas with larger debris (e.g. a wooded area) that may require more opening space.

The bar opening space for small pipes shall be less than the pipe diameter. 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 shall not shift to the grate. Nor shall the grate cause the headwater to rise above planned levels. Therefore, headlosses through the grate shall be calculated. A number of empirical loss equations exist, though many have difficult-to-estimate variables. For a discussion of headloss related to grates with example empirical loss equations, see Debo & Reese, 1994.

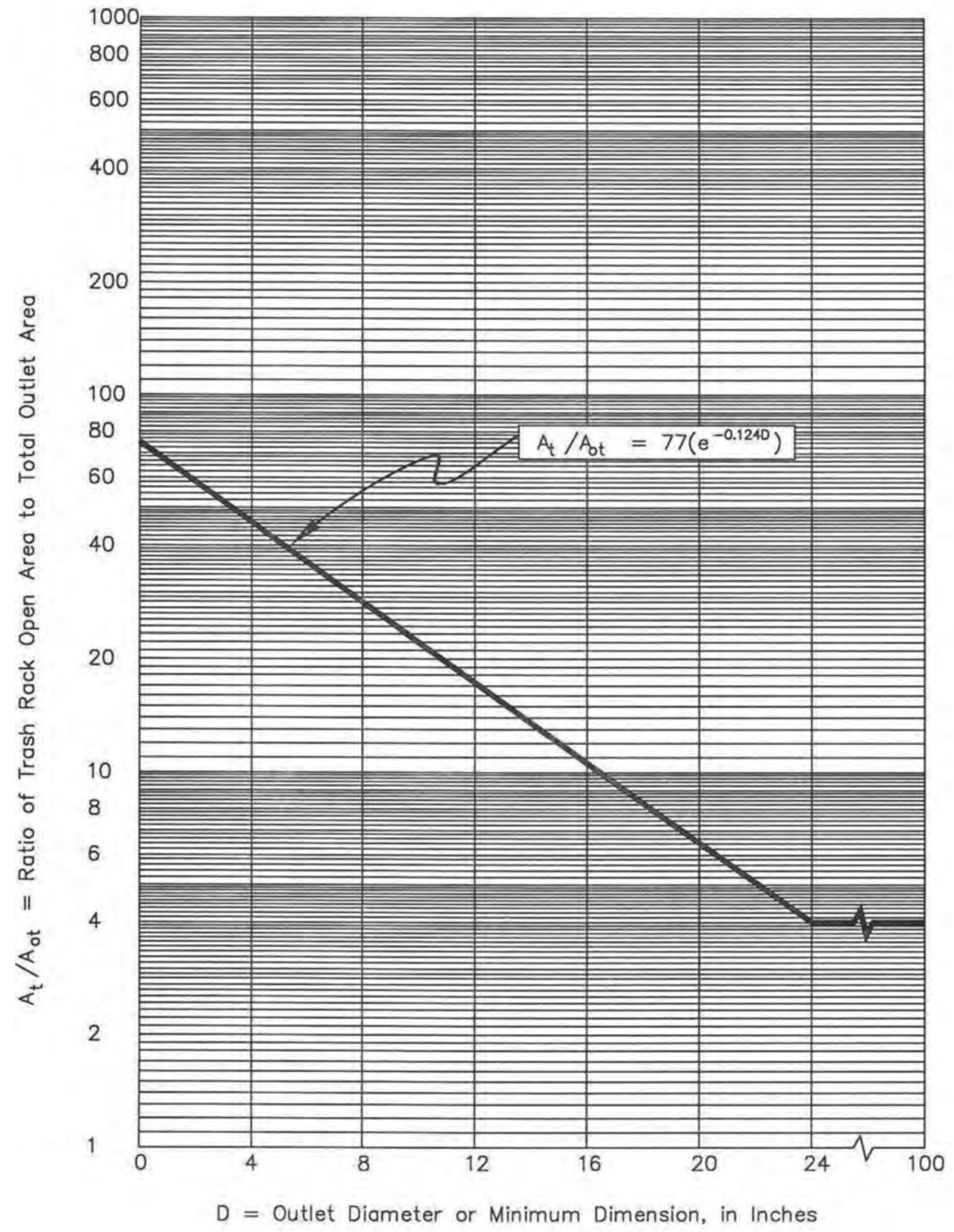


Figure 6-13 Minimum Rack Size vs. Outlet Diameter

Source: Urban Drainage and Flood Control District, 2001 (Denver, CO)

(This page was intentionally left blank)

Appendix 6-A

References

(This page was intentionally left blank)

Brater, E.F., and H.W. King. Handbook of Hydraulics. 6th ed. New York: McGraw Hill Book Company. 1976.

Chow, C.N. Open Channel Hydraulics. New York: McGraw Hill Book Company. 1959

City of Lincoln, Nebraska Drainage Criteria Manual. February 2000, Revision – May 2004.

Debo, T.N. and A.J. Reese. Municipal Storm Water Management. Boca Raton, Ann Arbor, London, Tokyo: Lewis Publishers. 1994.

McBridge, M.C., and Y.M. Sternberg. Stormwater Management Infiltration Structures. Publication No. FHWA-MS-83/04. Baltimore, Md. 1983.

Sandvik, A. Proportional Weirs for Stormwater Pond Outlets. Civil Engineering, ASCE, pp. 54-56. March, 1985.

Schu¬eler, T.R. Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs. Washington, D.C.: Metropolitan Washington Council of Governments. 1987.

Sowers, G.B., and G.F. Sowers. Introductory Soil Mechanics and Foundations. 3rd ed. New York: MacMillan Publishing Company. 1970.

Spangler, M.G., and R. L. Handy. Soil Engineering. 4th ed. New York: Harper & Row. 1982.

State of Nebraska Department of Water Resources. Rules for Surface Water. August, 1995.

State of Nebraska Department of Water Resources. Surface Water Chapter 46.

Stormwater Management Manual C Volume 2 Procedures. Metropolitan Government of Nashville and Davidson County. The EDGE Group, Inc., and CH2M Hill. July, 1988.

Urban Hydrology for Small Watersheds (TR-55)

Wong, S.L., and R.H. McCuen, R. H. The design of vegetative buffer strips for runoff and sediment control in stormwater management in coastal areas. Annapolis: Maryland Department of Natural Resources. 1982.

Wycuff, R.L., and U. P. Singh. Preliminary Hydrologic Design of Small Flood Detention Reservoirs. Water Resources Bulletin. Vol. 12, No. 2, pp 337-49. 1976.

Yim, C.S., and U.M. Sternberg. Development and testing of granular filter design criteria for stormwater management infil¬tration structures. Baltimore: Maryland Department of Transportation. 1987.

Yu, S.L., Norris, W.K., and D.C. Wyant. Urban BMP demonstration project of the Albemarle/Charlottesville area. Report No. UDA/530358/CZ88/102. Charlottesville: University of Virginia. 1987.