Chapter 13

STORAGE FACILITIES

SOUTH DAKOTA DRAINAGE MANUAL

October 2011

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13.1 INTRODUCTION

13.1.1 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 downstream land owners. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and, often, the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This Chapter provides general design criteria for detention/retention storage basins, preliminary and final sizing procedures and reservoir routing calculations. For more detailed information, see HEC 22 (Reference (1)) and HDS No. 2 (Reference (2)). Storage and flood routing associated with culverts is addressed in Chapter 10 "Culverts." The benefits of storage facilities can be divided into two major control categories of quality and quantity.

13.1.1.1 Quality

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

- decrease downstream channel erosion;
- control sediment deposition; and
- improve water quality through:
  - stormwater filtration, and
  - capture of the first flush with detention for 24 hours or more.

13.1.1.2 Quantity

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

- prevention or reduction of peak runoff rate increases caused by urban development,

- mitigation of downstream drainage capacity problems,
- recharge of groundwater resources,
- reduction or elimination of the need for downstream outfall improvements, and
- maintenance of historic low-flow rates by controlled discharge from storage.

13.1.2 Location Considerations

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

13.1.3 Detention and Retention

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For this Chapter, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are only addressed in this *Manual* for the case of land-locked retention (see Section 13.9). Retention facilities are designed to contain a permanent pool of water. Because most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this Chapter to include detention and retention facilities. If special procedures are required for detention or retention facilities, these will be specified.

Storage facilities may be small in terms of storage capacity and dam height where serving a single outfall from a watershed of a few acres, or they may be larger facilities serving as regional stormwater management control. Although the same principles apply to all storage facilities, Sections 13.9 and 13.10 more specifically relate to the smaller installations.

13.1.4 Design Practice

SDDOT practice is to design and analyze storage facilities using reservoir routing calculations. Routing calculations required to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations, there are many available reservoir routing software applications (see Section 18.2.5). SDDOT will consider managing stormwater quantity with storage facilities if limiting peak runoff rates is required to match one or more of the following values:

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

In rare cases, reservoir routing may be used to minimize a drainage structure size where there is considerable natural storage immediately upstream.

13.2 SYMBOLS AND DEFINITIONS

To provide consistency throughout this *Manual*, the symbols in Figure 13.2-A will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this Chapter, the symbol will be defined in the text or equations.

Symbol	Definition	Units
A	Cross sectional or surface area	sq ft
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/sec^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Inflow	cu ft
L	Length	ft
O	Outflow	cu ft
r	Ratio of width to length of basin at base	cfs
Q_i	Peak inflow rate	cfs
Q_o	Peak outflow rate	cfs
t_i	Duration of basin inflow	minute or sec
t_p	Time to peak of the hydrograph	hour or sec
V	Storage volume	cu ft, acre-ft, acre-in
V_s	Storage volume estimate	cu ft
W	Width of basin	ft
z	Side slope factor	H:V

Figure 13.2-A — SYMBOLS AND DEFINITIONS

13.3 DESIGN CRITERIA

13.3.1 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 parking lots, road embankments and freeway interchanges, in parks and other recreational areas, in small lakes and ponds and in 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 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. The design criteria for storage facilities should include:

- release rate,
- storage volume,
- grading and depth requirements,
- outlet works,
- location, and
- construction and maintenance considerations (see Section 13.11).

13.3.2 Release Rate

The release rates of control structures should:

- approximate pre-developed peak runoff for the design storm, and
- provide for emergency overflow of the 100-year discharge.

13.3.3 Storage Volume

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

13.3.4 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.

13.3.4.1 General

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

A minimum freeboard of 1 ft above the 100-year design storm high-water elevation should be provided for impoundment depths of less than 25 ft. Impoundment depths greater than 25 ft or volumes greater than 50 acre-ft are subject to the requirements of the *Safe Dams Act* (see Section 13.4), 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. Pedestrian consideration is addressed in Section 13.11.3.

13.3.4.2 Detention

Slope areas above the normal high-water elevations of storage facilities slope toward the facilities to allow drainage and to prevent standing water. Fine grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2% bottom slope is recommended. A low-flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing water conditions.

13.3.4.3 Retention

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

- depth,
- habitat, and
- bottom and shore geometry.

13.3.5 Outlet Works

Outlet works selected for storage facilities should be able to accomplish the design functions of the facility. Typically, outlet works include a principal outlet and an emergency overflow. Outlet works can take the form of combinations of drop inlets, pipes, weirs and orifices. Slotted riser pipes are discouraged because of clogging problems. The principal outlet 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 fails. The minimum flood to be used to size the emergency outlet is the 100-year flood. The size of a particular outlet works is determined using hydrologic routing calculations. In the unusual location where an emergency overflow is not practical due to a high embankment, evaluate the facility using the maximum measured flood flow (see Section 7.7.4).

13.3.6 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. For all storage facilities, channel routing calculations should proceed downstream to a confluence point where the drainage area being analyzed represents 10% of the total drainage area. At this point, the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph should be assessed for detrimental effects on downstream areas.

13.4 SAFE DAMS ACT

The South Dakota Department of Environment and Natural Resources (DENR) is responsible for the administration of the *Safe Dams Act*. The requirements of the *Act* can be found on the DENR website. DENR should be consulted if a proposed highway facility meets the DENR criteria:

> *"A structure is a dam if the height to the dam crest is greater than or equal to 25 feet and the storage at the dam crest (not at the spillway elevation) is greater than 15 acre feet or if the height to the dam crest is greater than 6 feet and the storage at the dam crest (not at the spillway elevation) is greater than or equal to 50 acre feet. The height of the dam is the difference in elevation between the natural bed of the watercourse or the lowest point on the toe of the dam, whichever is lower, and the crest elevation of the dam."*

Dams or dugouts storing 25 acre-ft or less of water located on a dry draw or on a non-navigable watercourse can be constructed without a water right permit provided that a location notice is filed. However, the dam may not be constructed if it changes the course of the water, interferes with vested rights or wrongfully floods land owned by others. See the DENR website for additional information.

13.5 GENERAL PROCEDURE

13.5.1 Data needs

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

- inflow hydrograph for all selected design storms;
- stage-storage curve for proposed storage facility (see Figure 13.5-A for an example). For large storage volumes (e.g., for reservoirs), use acre-ft; otherwise, use cubic feet; and
- stage-discharge curve for all outlet control structures (see Figure 13.5-B for an example).

Using this data, a design procedure is used to route the inflow hydrograph through the storage facility to establish an outflow hydrograph; see Figure 13.5-C for an example of an inflow and outflow hydrograph. If the desired outflow results are not achieved, basin and outlet geometry are varied to yield new stage-storage and stage-discharge curves and the routing procedure is redone until the desired outflow hydrograph is achieved.

13.5.2 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve is usually developed using a topographic map and one of the following formulas — the average-end area, frustum of a pyramid or prismoidal formulas. Storage basins are often irregular in shape to blend well with the surrounding terrain and to improve aesthetics. Therefore, the average-end area formula is usually preferred as the method to use on non-geometric areas. The average-end area formula is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad \text{(Equation 13.1)}$$

where:

$V_{1,2}$ = storage volume between Elevations 1 and 2, cu ft
$A_{1,2}$ = surface area at Elevations 1 and 2 respectively, sq ft
d = change in elevation between Points 1 and 2, ft

The frustum of a pyramid is expressed as:

$$V = d\,[A_1 + (A_1A_2)^{0.5} + A_2]/3 \qquad \text{(Equation 13.2)}$$

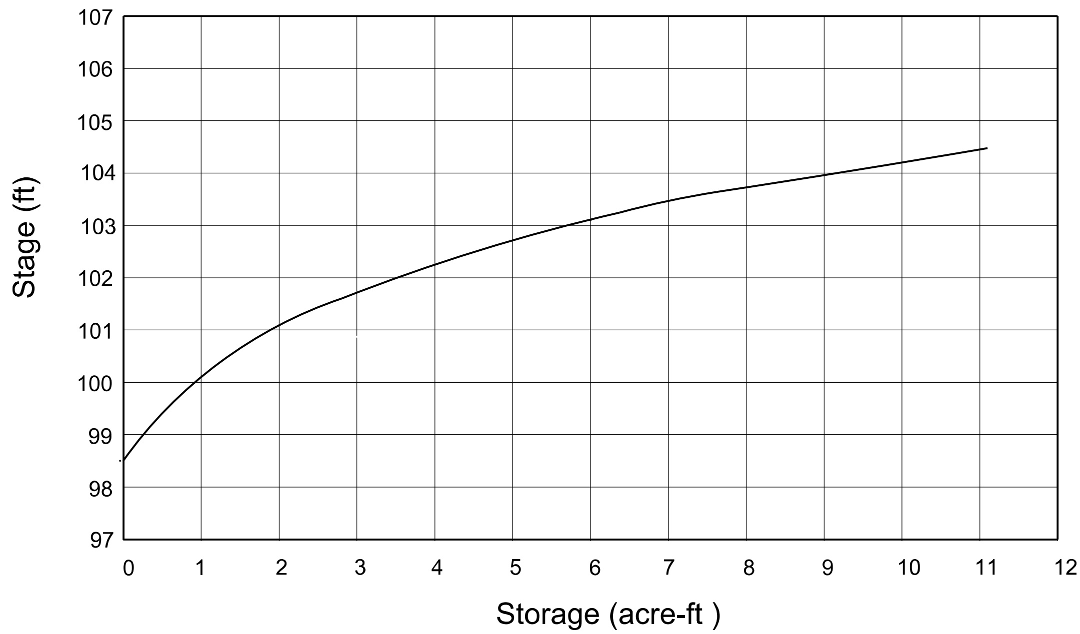


Figure 13.5-A — EXAMPLE STAGE-STORAGE CURVE

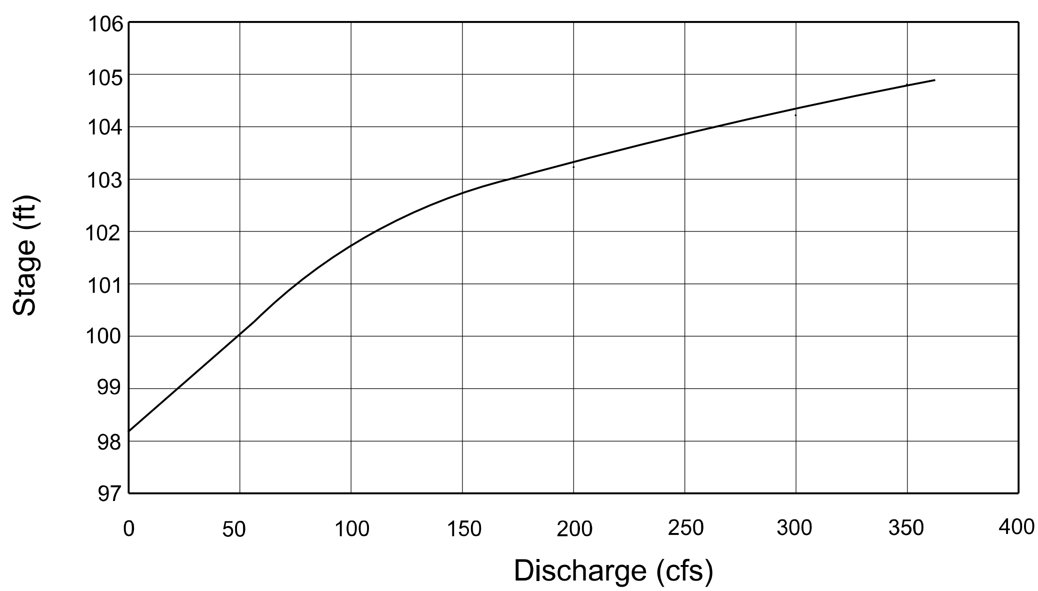


Figure 13.5-B — EXAMPLE STAGE-DISCHARGE CURVE

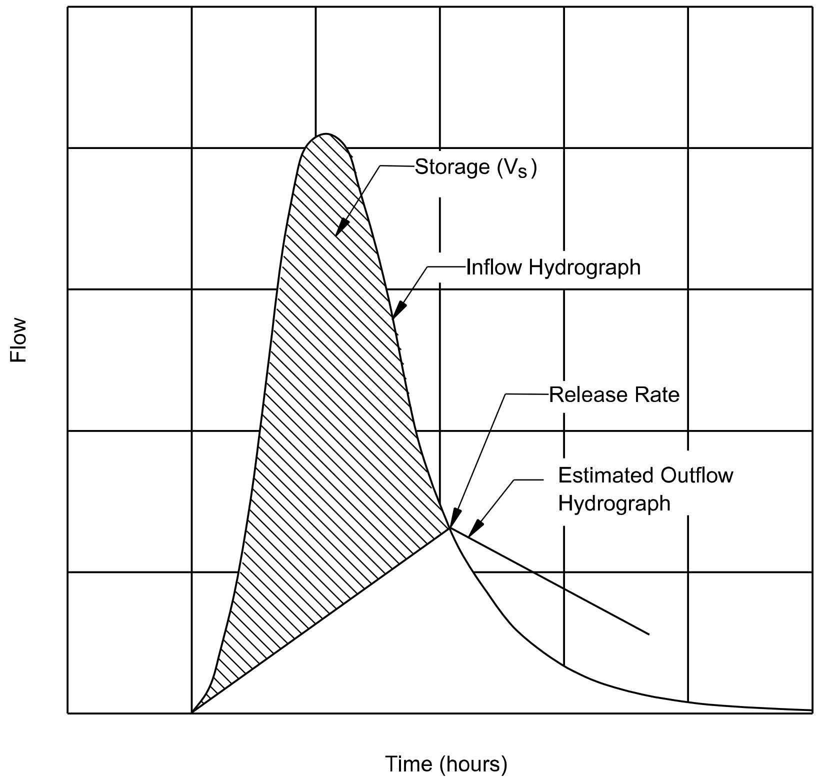


Figure 13.5-C — INFLOW AND OUTFLOW HYDROGRAPHS FROM A STREAM REACH

where:

V	=	volume of frustum of a pyramid, cu ft
$A_{1,2}$	=	surface area at Elevations 1 and 2 respectively, sq ft
d	=	change in elevation between Points 1 and 2, ft

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)zD^2 + (4/3)z^2D^3 \qquad \text{(Equation 13.3)}$$

where:

V	=	volume of trapezoidal basin, cu ft
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 components of side slope (H:V)
r	=	ratio of width to length of basin at the base

Estimating the trial dimensions of a basin for a given basin storage volume can be accomplished by rearranging Equation 13.3 as shown in Equation 13.4:

$$L = \{ -zD (r + 1) + [(zD)^2 (r + 1)^2 - 5.33 (zD)^2 r +((4r V)/D)]^{0.5}\} / 2r \quad \text{(Equation 13.4)}$$

13.5.3 Stage-Discharge Curve

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

A typical outlet structure may include culverts, weirs, orifices or a combination of all three. Because culverts are typically used as the primary outlet, the procedures presented in Section 10.4 should be used to develop stage-discharge data. When analyzing release rates, the tailwater influence of the culvert should be considered on the emergency control structure (orifice and/or weirs) to determine the effective head on each opening. Avoid slotted riser pipe outlet facilities due to debris-plugging potential. For design information on weirs and orifices, see HEC 22 (Reference (1)).

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

The stage-discharge curve should reflect the discharge characteristics of both the principal and emergency outlets. Develop a composite stage-discharge curve, which combines the discharge rating curve for all components of the outlet control structure. Figure 13.5-D illustrates an example composite stage-discharge curve.

13.5.4 Procedure

The most commonly used method for routing an inflow hydrograph through a detention pond is the Storage Indication or Modified Puls Method. This Method begins with the continuity equation that states that the inflow minus the outflow equals the change in storage (I – O = S). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by Equation 13.5. This relationship is illustrated graphically in Figure 13.5-E:

$$\frac{\Delta S}{\Delta t} = \frac{I_2 + I_2}{2} - \frac{O_1 + O_2}{2} \quad \text{(Equation 13.5)}$$

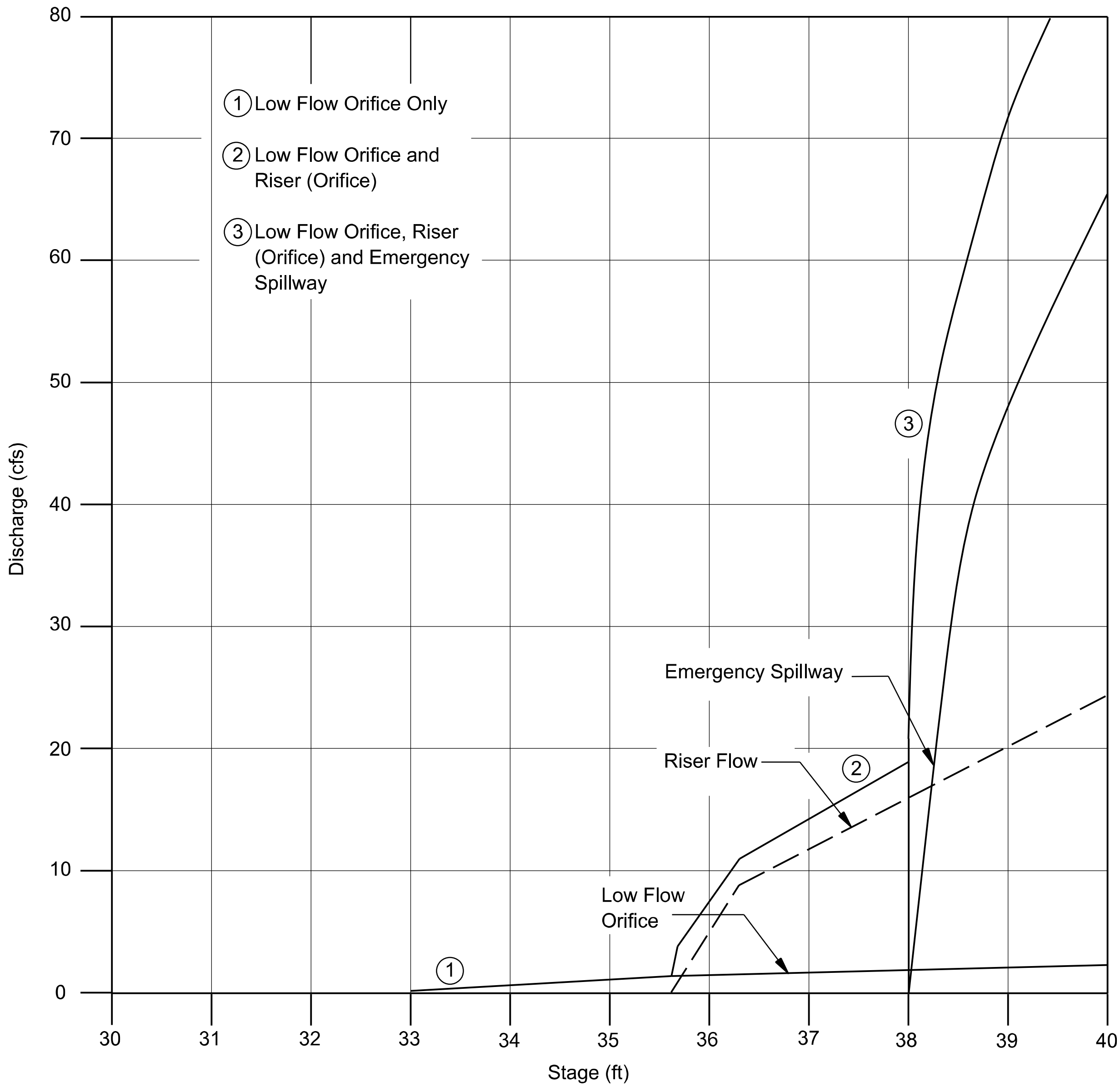


Figure 13.5-D — COMBINED STAGE-DISCHARGE RELATIONSHIP

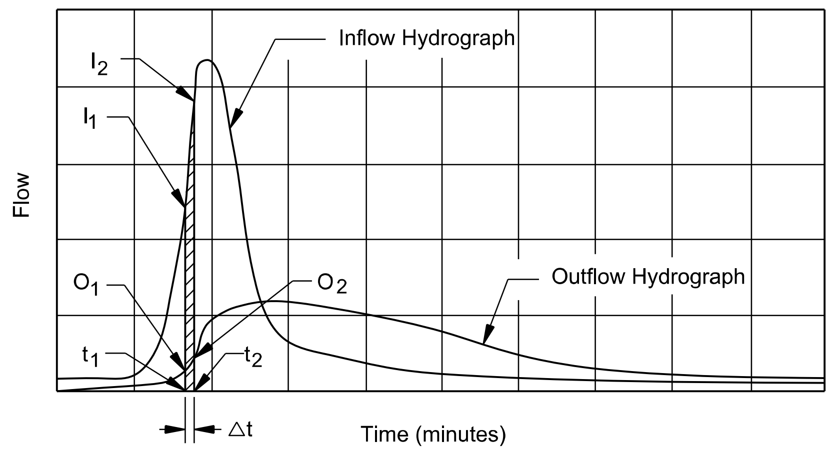


Figure 13.5-E — ROUTING HYDROGRAPH SCHEMATIC

where:

ΔS	=	change in storage, cu ft
Δt	=	time interval, minutes
I	=	inflow, cu ft
O	=	outflow, cu ft

In Equation 13.5, subscript 1 refers to the beginning and subscript 2 refers to the end of the time interval.

Equation 13.5 can be rearranged so that all known values are on the left side of the equation and all unknown values are located on the right-hand side of the equation, as shown in Equation 13.6. Now the equation with two unknowns, S_2 and O_2, can be solved with one equation:

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) \qquad \text{(Equation 13.6)}$$

The following procedure can be used to perform routing through a reservoir or storage facility using Equation 13.6:

Step 1 Compute inflow hydrograph for runoff from the design and 100-year storms using the procedures outlined in Section 7.15. Both pre- and post-development hydrographs are required for the design storm. Only the post-development hydrograph is required for runoff from the 100-year storm.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Section 13.6). If storage requirements are satisfied for runoff from the design storm, runoff from intermediate storms is assumed to be controlled.

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

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

Step 5 Perform routing calculations using inflow hydrographs from Step 1 and Equation 13.6 to check the preliminary design. If the routed post-development peak discharge from the design storm exceeds the pre-development peak discharge, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

Step 6 Consider emergency overflow from runoff due to the 100-year or larger design storm and established freeboard requirements.

Step 7 Assess the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems. If a sensitive area exists downstream, route the exit hydrograph from the storage facility though the downstream channel system to the area of interest or until a confluence point is reached where the drainage area being analyzed represents 10% of the total drainage area.

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

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results. HEC 22 (Reference (1)), Example 8.9 uses the USGS Nationwide urban hydrograph to compute runoff for existing and proposed conditions. Example 8.3 provides step-by-step hand calculations.

13.5.5 <u>Example Problem</u>

The procedure in Section 13.5.4 is illustrated in the following example, which uses the FHWA Hydraulic Toolbox.

Given: The 25-year hydrograph that was produced in Section 7.15.2.4.

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

Step 1 Compute inflow hydrograph; see Figure 13.5-F.

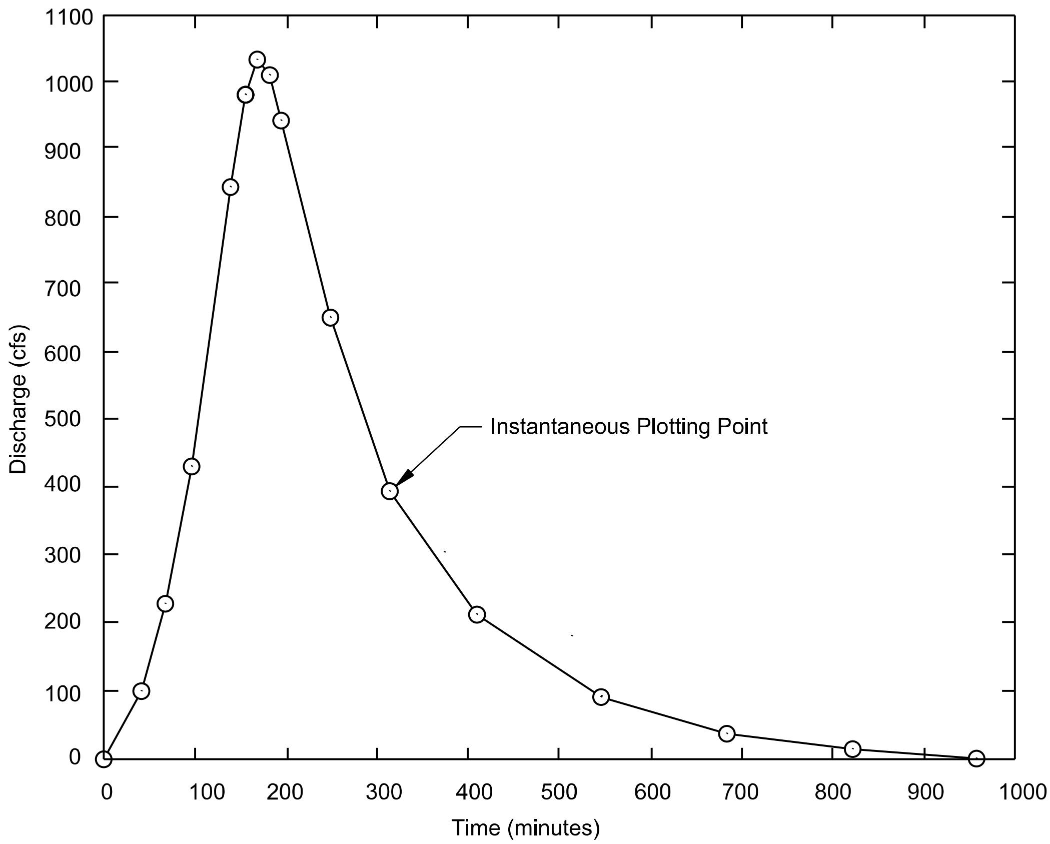


Figure 13.5-F — 25-YEAR HYDROGRAPH
(from Section 7.15.2.4)

The points are provided in the following table:

Minutes	Cubic Feet Per Second
0	0
41	96
68	223
96	429
137	841
151	979
164	1030
178	1010
192	944
246	652
315	395
411	206
548	89
685	34
821	8.6
958	0

Step 2 Perform preliminary calculations to evaluate detention storage.

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

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

Step 5 Perform routing calculations:

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

The diameter of the culvert and the size of the basin can be varied until the target elevation is achieved. In this example, a 7-ft diameter culvert with a trapezoidal-shaped basin that is 500-ft wide by 2100-ft long will store 256 acre-ft of water at the peak elevation of 109.90 ft. See Figure 13.5-G for the storage and discharge table and routed hydrograph for the final alternative.

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

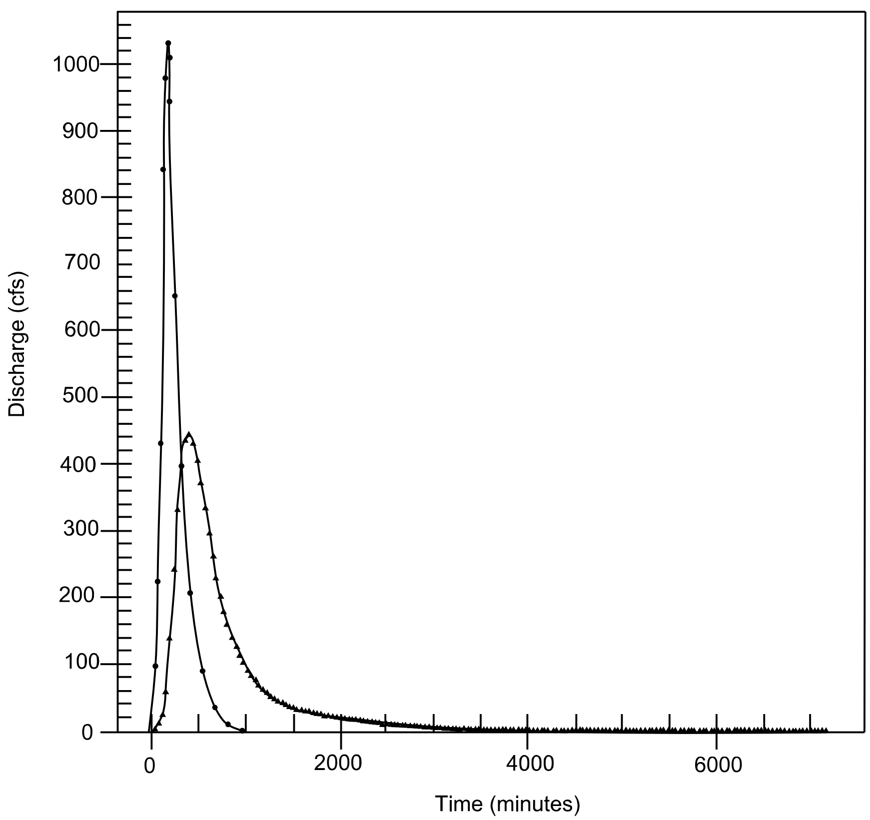


Figure 13.5-G — INFLOW AND OUTFLOW HYDROGRAPHS FOR STORAGE EXAMPLE

13.6 PRELIMINARY DETENTION CALCULATIONS

13.6.1 Storage Volume

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 13.6-A.

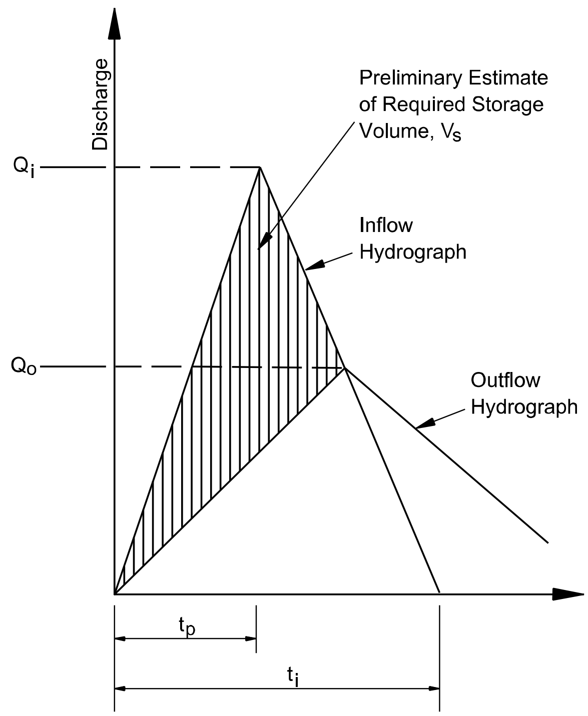


Figure 13.6-A — TRIANGULAR-SHAPED HYDROGRAPHS (For Preliminary Estimate of Required Storage Volume)

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

$$V_s = 0.5t_i(Q_i - Q_o) \qquad \text{(Equation 13.7)}$$

where:

V_s = storage volume estimate, cu ft
Q_i = peak inflow rate, cfs
Q_o = peak outflow rate, cfs
t_i = duration of basin inflow, seconds

Any consistent units may be used for Equation 13.7.

13.6.2 Preliminary Basin Dimensions

Use the following procedure to develop the preliminary basin dimensions:

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

13.7 DRY POND (Detention Basin)

13.7.1 Introduction

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

13.7.2 Design

For quantity purposes, design the pond to reduce the post-construction peak flow from the design storm to the preconstruction level. The basin should be able to pass a 100-year storm safely. To control these storms, the basin storage should be equal to the area between the pre- and post-construction hydrographs.

The design storm should be routed through the facility to ensure that the peak flows from the post-construction watershed are not greater than the corresponding preconstruction peak flows. Finally, a 100-year storm should be routed through the facility to ensure that the embankment will not be damaged or fail during the passage of that storm. Several outlets may be used to control different storms (see Figure 13.7-A) and an emergency spillway to control anything larger, including the 100-year storm. To improve the efficiency of the outlet, it may be necessary to include an antivortex device (see Figure 13.7-B).

13.7.2.1 Quantity and Quality Combined

Several design variations may be considered to enhance the capabilities of the facility. One consideration is to shape the basin to improve its pollutant-removal capabilities. The length-to-width ratio should be at least three, and a wedge-shaped basin (wider at the outlet) can also improve pollutant removal. The inlet, outlet and side slopes should be stabilized with riprap and/or vegetation to prevent erosion. The basin floor should also be vegetated to stabilize the soil and increase biological uptake. The pond floor should be sloped no less than 2% to prevent the ponding of stormwater. The side slopes should allow for easy maintenance access. A marsh or wetland can be established on the pond floor to increase biological uptake. A sediment forebay (a small sediment trap at the inlet of the basin, either a depressed area or a shallow area with a very flat slope where sediment is easily deposited) can be used to catch the sediment before it fills the basin (see Figure 13.7-C).

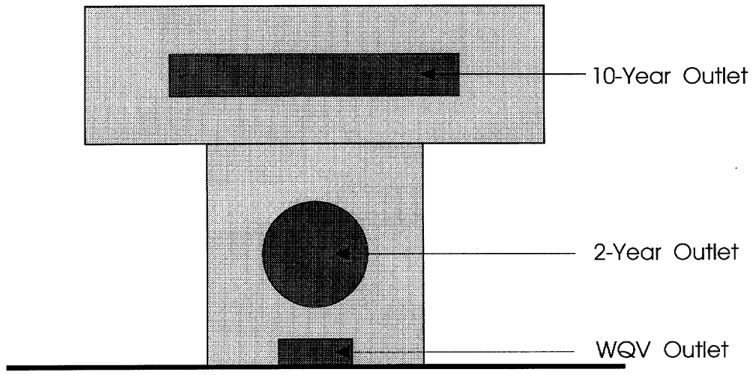


Figure 13.7-A — CONCRETE RISER

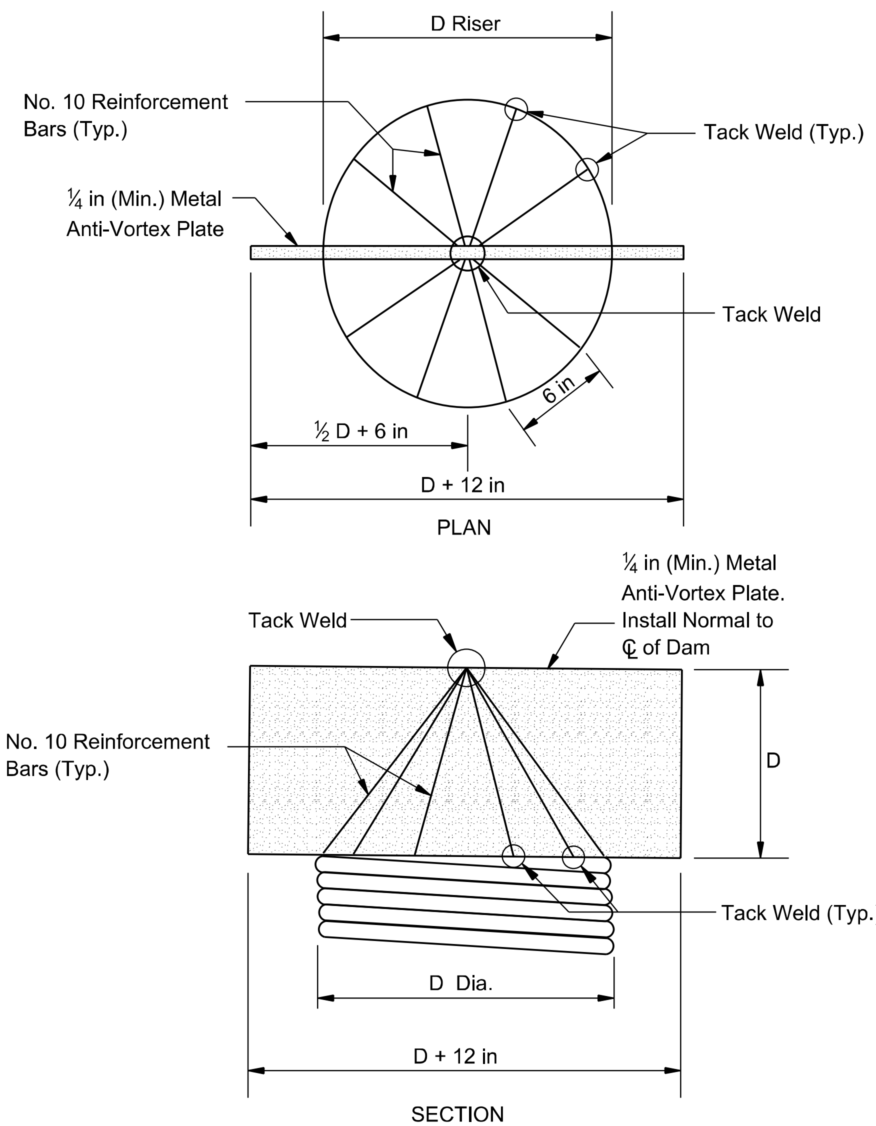


Figure 13.7-B — ANTIVORTEX PLATE AND TRASH RACK

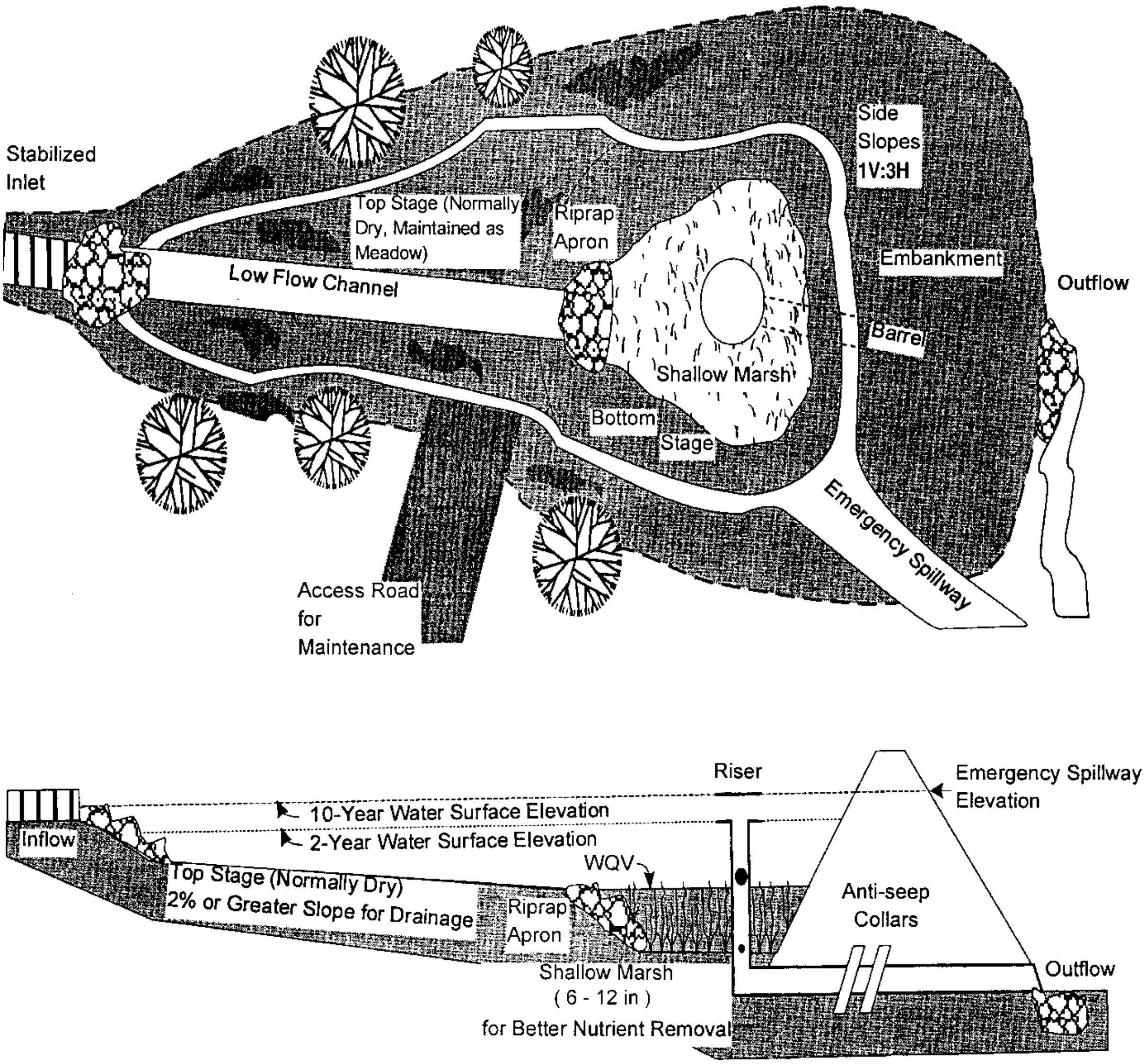


Figure 13.7-C — DRY POND (after Reference (3))

The basin can also have two stages (see Figure 13.7-C) — a lower stage to hold the smaller storms and a top stage, which is rarely inundated and can be used for other purposes, to help store the larger storms. Safety considerations include reducing the chance of drowning by fencing the basin (see Section 13.11.3), reducing the maximum depth and/or including ledges and mild slopes to prevent people from falling in and to facilitate their escape from the basin.

13.7.2.2 Outlets

Outlets for dry basins can be designed in a wide variety of configurations. Most outlets use riser pipes of concrete or corrugated metal. These risers can be designed to control different storms through the use of several orifices on the riser; for example, a small diameter to control the water quality volume (WQV), an orifice to control a 2-year storm and a larger orifice to control a 10-year storm. This larger flow is usually controlled by stormwater flowing in through the top of the riser, using the entire riser diameter. In the latter case, an antivortex design may be necessary. Larger flows are usually handled by an emergency spillway. Because the WQV outlet should be small to detain the WQV for a sufficient length of time, it can be easily clogged; thus, a minimum size of 3 in should be used. To prevent clogging, a trash rack may be included in the design to cover the orifices.

13.7.2.3 Dry Pond Summary

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

Figure 13.7-D — SUMMARY OF CONSIDERATIONS FOR A DRY POND

13.8 WET POND (Extended Detention Basin)

A wet pond is very similar to a dry detention basin in that it detains stormwater, but is different in that it maintains a permanent pool during dry weather. Wet ponds are usually more expensive than dry detention basins and usually serve large watersheds. Because of this permanent pool, wet ponds may have recreational benefits. SDDOT rarely uses these types of facilities. HEC 22 (Reference (1)) contains an illustration of a wet pond and discussion of wet ponds used to enhance urban water quality. Additional guidance is found in the EPA Storm Water Technology Fact Sheet Wet Detention Ponds (Reference (4)).

13.9 LAND-LOCKED RETENTION

Watershed areas that drain to a central depression with no positive outlet (playa lakes) are typical of many topographic areas including karst topography. They can be evaluated using a water budget (see Section 13.10) or flow routing procedure to estimate flood elevations. The water budget procedure can be used to determine if a permanent pond will exist. Flood routing can be used to determine what frequency storm will be stored in the depressed area. If the depression represents less than 10% of a highway drainage basin, the attenuation effects of the depression do not have to be considered and will represent additional safety for the highway. Larger areas should be evaluated, but can also be discounted if they do not store the design storm. HEC 22 (Reference (1)) contains guidance for enhancing infiltration.

13.10 WATER BUDGET FOR LAND-LOCKED FACILITIES

13.10.1 Objectives

Water budget calculations can be used to determine if a permanent pool will be maintained or if a land-locked facility will perform for average annual conditions. The water budget should consider all significant inflows and outflows, including but not limited to:

- rainfall,
- runoff,
- infiltration,
- exfiltration,
- evaporation, and
- outflow.

Average annual runoff may be computed using a weighted runoff coefficient for the tributary drainage area multiplied by the average annual rainfall volume. Infiltration and exfiltration should be based on site-specific soils testing data. Evaporation may be approximated using the mean monthly pan evaporation or free-water surface evaporation data given below from NOAA Technical Report 33 (Reference (5)); see Figure 13.10-A.

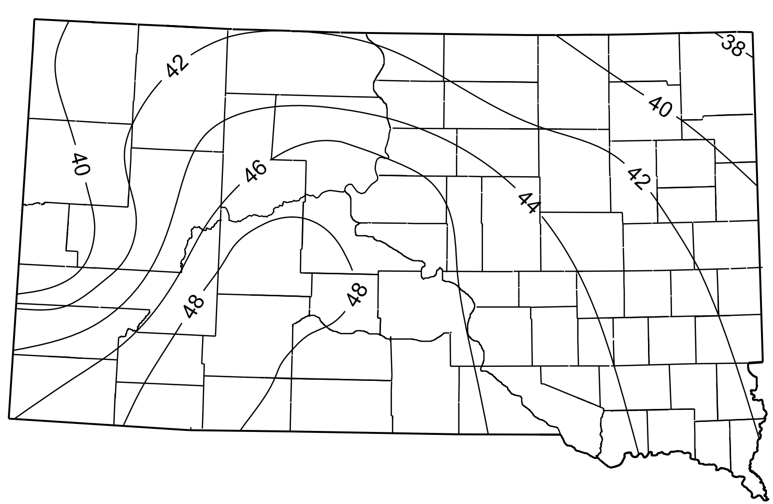


Figure 13.10-A — SOUTH DAKOTA AVERAGE ANNUAL FREE WATER SURFACE EVAPORATION IN INCHES FROM NOAA TECHNICAL REPORT 33 (Reference (5))

13.10.2 Example Problem

13.10.2.1 Given

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

13.10.2.2 Solution

Step 1 From rainfall records, the average annual rainfall is approximately 50 in.

Step 2 The mean annual evaporation is 35 in

Step 3 The average annual runoff is estimated as:

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

Step 4 The average annual evaporation is estimated as:

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

Step 5 The average annual infiltration is estimated as:

Infiltration = (0.1 in/hour)(24 hours/day)(365 days/year)(2 acres)
Infiltration = 1752 acre-in

Step 6 Neglecting basin outflow and assuming no change in storage, the runoff (or inflow) less evaporation and infiltration losses is:

Net Budget = 1500 – 105 – 1752 = -357 acre-in.

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

Step 7 Revise pool design as follows:

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

Step 8 Recompute the evaporation and infiltration:

Evaporation = (35 in)(2 acres) = 70 acre-in
Infiltration = (0.1 in/hour)(24 hours/day)(365 days/year)(1 acre) = 876 acre-in

Step 9 The revised runoff less evaporation and infiltration losses is:

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

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

13.11 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

13.11.1 General

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

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

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

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

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

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

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

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

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

13.11.2 Sediment Basins

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

The following is an example methodology that could be used to develop a sediment basin maintenance schedule. Figure 13.11-A can be used to estimate sediment-trap efficiency for sediment basins. The procedure for using this figure is as follows:

Step 1 Establish sediment-generation criteria (e.g., 1786 cu ft of silt per disturbed acre draining to the sediment basin). The amount of sediment from a given area is estimated using the Revised Universal Soil Loss Equation (RUSLE). Guidance for applying this equation is found in Section 5.4.1 of the SDDOT *Water Quality Enhancement Program Design Manual*.

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

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

Step 4 Trap efficiency can be estimated from Figure 13.11-A as follows:

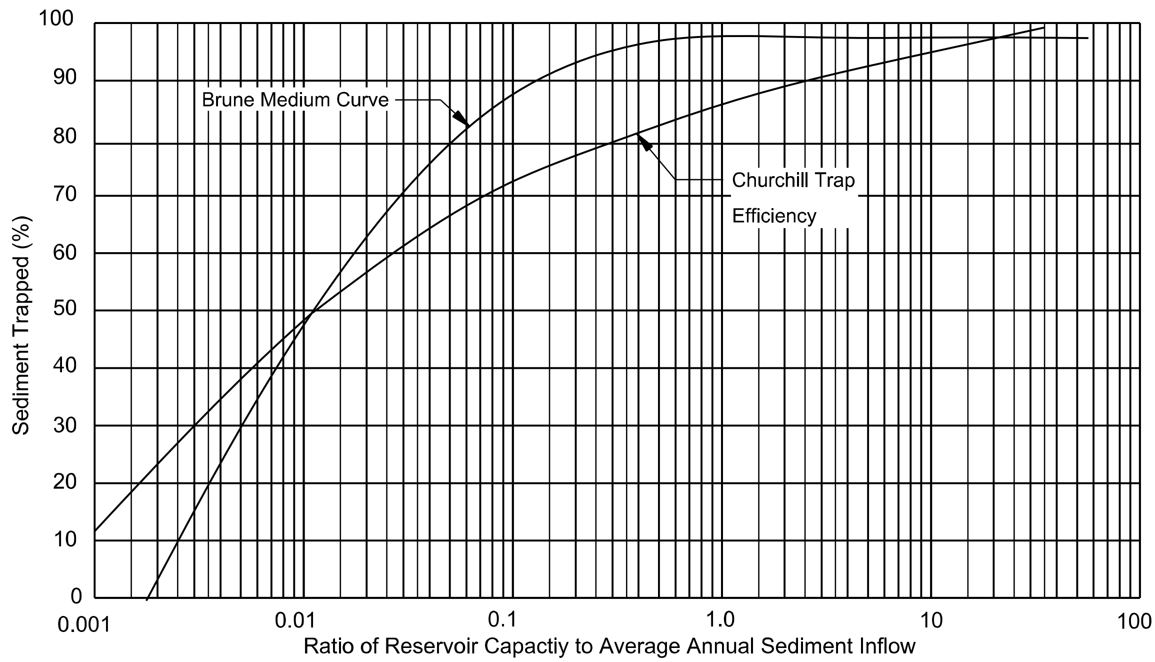
- (19,421 cu ft available storage/17,646 cu ft) is near 1.1.
- From Figure 13.11-A, Trap Efficiency = 90% (Churchill) and 98% (Brune).

Step 5 Determine how frequently the sediment basin should be cleaned:

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

13.11.3 Design Considerations for Pedestrians

Drainage features adjacent to schools, recreational areas or urban areas subject to frequent visits by the public may need to be fenced.



Notes:

1. *Capacity is total sediment basin volume up to emergency spillway crest. From Reference (6).*

2. *A general guideline is to use the Brune method for large storage or normal ponded reservoirs and the Churchill curve for settling basins, small reservoirs, flood-retarding structures, semi-dry reservoirs or reservoirs that are continuously sluiced.*

Figure 13.11-A — EFFICIENCY OF SEDIMENT BASINS

13.12 REFERENCES

(1) Federal Highway Administration, *Urban Drainage Design Manual, Third Edition*, Hydraulic Engineering Circular No. 22, FHWA-NHI-10-009, US Department of Transportation, Washington DC, 2009.

(2) Federal Highway Administration, *Highway Hydrology,* Hydraulic Design Series No. 2 (HDS 2)*,* FHWA-NHI-02-001, 2002.

(3) Schueler, T. R., *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs,* Metropolitan Washington Council of Governments, Washington DC, 1987.

(4) Environmental Protection Agency, *Storm Water Technology Fact Sheet Wet Detention Ponds*, EPA 832-F-99-048, September 1999.

(5) NOAA Technical Report 33, *Evaporation Atlas for the Contiguous 48 United States*, US Department of Commerce, Washington DC, 1982.

(6) US Bureau of Reclamation, *Erosion and Sedimentation Manual,* November 2006.