12 Storage Facilities

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12.1 INTRODUCTION

12.1.1 Overview

Traditional storm drainage systems have been designed to collect and convey stormwater runoff as rapidly as possible to a point where it can be discharged. As areas urbanize, this type of design may result in major drainage and flooding problems downstream.

The temporary storage of storm runoff can decrease downstream peak flows, and often reduce the cost of the downstream conveyance system. Storage facilities can also improve water quality in downstream receiving waters.

Photo 12.1

Detention storage facilities range from small ponds contained on roof tops or in parking lots, to large lakes and reservoirs. This chapter provides general design criteria for sizing detention and retention storage basins, as well as procedures for reservoir-routing calculations. Whenever possible, measures to improve stormwater quality should be incorporated into design of storage facilities. Specific stormwater quality design guidelines are provided in Chapter 16 - Permanent

Water Quality, additional guidelines can be found in the *CDOT Erosion Control and Stormwater Quality Guide* (2014).

The objectives for managing stormwater quantity with storage facilities are typically based on limiting peak-runoff rates to match one or more of the following values:

- Historic rates for specific design conditions (i.e., post-development peak flows equal pre-development peak flows for a particular frequency of occurrence);
- Non-hazardous discharge capacity of the downstream drainage system; and
- A specified value for allowable discharge set by a regulatory jurisdiction.

Photo 12.2

Storage facilities are used to store increases in volume and to reduce discharge rates. They also provide for sediment and debris collection, which improves downstream water quality. Public health and safety benefits may often occur from storage of stormwater runoff.

CDOT is involved in both the design and review of storage facilities. CDOT often designs storage facilities on its projects or in coordination with local municipalities. The Department is responsible for review of developers' drainage plans to ensure runoff rates are detained at historic rates, and that the development will cause no adverse impacts to a State highway or the travelling public. Development adjacent to State highways will often increase runoff volumes and rates. Stormwater detention is an important means to protect the highway and public.

12.1.2 Location Considerations

The location of storage facilities is very important because it affects the ability of these facilities to control downstream flooding. Small facilities will only have minimal flood-control benefits, and these benefits quickly diminish as the flood wave travels downstream.

There are two general classifications of storage facilities based on the relationship to the storm drain system. In line storage facilities are an integral part of the stormwater conveyance system. These include excess capacity in storm drains, detention and retention ponds. Off line storage facilities are located apart from the active stormwater conveyance system. Off line storage fills only when a specific flow level is exceeded within the in line conveyance system, and empties when sufficient conveyance becomes available in the downstream system.

Multiple storage facilities located in the same drainage basin affect the timing of runoff through the conveyance system, which alters flood peaks in different downstream locations. It is important to design a storage facility as a drainage structure that both controls runoff from a defined area, and interacts with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a regional or basin-wide planning basis. CDOT should encourage and participate in such planning.

Storage facilities should be located with particular attention to public safety and community sentiment. Efforts should be made during storage-facility design and operation to minimize risks to the public and adjacent properties. Community and neighborhood involvement in the planning of storage facilities is extremely important in promoting public support of a project.

Photo 12.3

12.1.3 Detention and Retention

Urban stormwater-storage facilities are often referred to as either *detention* or *retention* facilities. The following definitions are used in this chapter:

- Detention facilities are designed to reduce the peak discharge and detain runoff for a short period of time. These facilities are designed to completely drain after the design storm has passed.

- Retention facilities are designed to contain a permanent pool of water, reducing the peak discharge and runoff volumes.

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 both. If special procedures are needed for detention or retention facilities, they 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. In either case, the guidelines in this chapter apply to all storage facilities.

12.1.4 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 reservoir-routing computer programs available. Spreadsheet-based computations are acceptable with thorough documentation of the calculation method and equations used.

Some municipalities utilize empirical equations to size detention volumes and release rates for onsite storage facilities in small developments. These methods are acceptable for developments adjacent to a State highway if the total developed area is less than five acres. All storage facilities for developments larger than 100 acres, and all storage facilities designed by CDOT or its consultant must be analyzed with reservoir-routing techniques. The Federal Aviation Administration (FAA) detention method is acceptable on CDOT projects or developments adjacent to a State highway if a discharge rate which varies with flood stage is used.

When using a computer program, empirical equation, or spreadsheet computation, it should be understood that they are only design tools and not a substitute for sound engineering judgement. For additional information on computer programs pertinent to storage facility design, including programs capable of performing reservoir-routing calculations, see Section 12.15 – Software for Designing Storage Facilities.

12.2 USES

12.2.1 Introduction

The use of storage facilities for stormwater management has increased dramatically in recent years. The benefits of storage facilities can be divided into two major control categories, quality and quantity.

12.2.2 Quality

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

- Decreased downstream channel erosion;
- Control of sediment deposition; and
- Improved water quality through stormwater filtration, and capture of the first flush with detention for 24 hr or more.

Refer to Chapter 16 - Permanent Water Quality for CDOT specifications, policies and design criteria related to permanent water-quality control measures.

12.2.3 Quantity

Controlling the quantity of stormwater with storage facilities provides the following benefits:

- Prevention or reduction of increases in peak-runoff rates 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.

Federal and state regulations have been established which place greater emphasis on improving the quality of stormwater runoff. While this chapter addresses design of storage facilities for reduction of flood volumes and peak discharges, features to enhance water quality can usually be incorporated into detention and retention pond facilities. Design criteria and general guidelines for incorporating water quality features into pond designs are provided in Chapter 16 - Permanent Water Quality, as well as in the *CDOT Erosion Control and Stormwater Quality Guidelines*.

12.2.4 Objectives

Objectives for managing stormwater quantity by storage facilities are typically based on limiting peak-runoff rates to match one or more of the following values:

- Historic rates for specific design conditions (i.e., post-development peak equal to pre-development peak for a particular frequency of occurrence);
- Non-hazardous discharge capacity of the downstream drainage system; and
- A specified value for allowable discharge set by a regulatory jurisdiction.

For a watershed without an adequate outfall, the total volume of runoff is critical and retention storage facilities are used to store increases in volume and control discharge rates.

12.3 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this manual, the symbols in Table 12.1 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.

12.4 DESIGN CRITERIA

12.4.1 General Criteria

Storage may be concentrated in large basin-wide or regional facilities, or distributed throughout an urban drainage system. Dispersed or on site storage may be developed in depressed areas in parking lots, road embankments and freeway interchanges, recreation areas, small lakes, ponds, and depressions within urban developments.

The utility of any storage facility depends on the amount of available 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, the effect of larger flows expected to pass through the storage facility should be included in the analysis. For most cases, ensuring that the 100-year flood is adequately accommodated is sufficient. But, in critical situations where there is the possibility of loss of life or significant property damage, the probable maximum flood (PMF) should be evaluated.

The design criteria for storage facilities should include:

- Release rate;
- Storage volume;
- Grading and depth limitations;
- Outlet works;
- Location and aesthetics; and
- Provisions to ensure public safety.

Table 12.1 Symbols and Definitions

Symbol	Definition	Units
A	Cross-sectional or surface area	ft^2
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
f	Infiltration rate	mm/hr
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Infiltration rate	mm/hr
I	Inflow rate	ft^3/s
L	Length	ft
Q, O	Flow or outflow rate	ft^3/s
S_a	Surface area	acre
S, V_s	Storage volume	ft^3, acre-ft
t	Routing time period	s
t_b	Time base on hydrograph	hr
T_i	Duration of basin inflow	hr
t_p	Time to peak	hr
W	Width of basin	ft
z	Side slope factor	-

12.4.2 Release Rate

Storage facility release rates must ensure that the 100-year post developed flood is detained to the 100-year pre-developed peak runoff rate. Wherever possible, multistage outlet works must be provided to ensure that runoff is detained at historic, predeveloped rates for the conventional design frequency of the downstream drainage system. Some municipalities have specific design storms for which developments must design storage-facility release rates. The local requirements are typically adhered to if the downstream drainage system capacity will not be exceeded.

Emergency overflow capacity is required to handle flows exceeding the post-development, 100-year discharge. Design calculations are required to demonstrate that runoff from the design storms is controlled.

12.4.3 Storage Volume

Storage volume must be adequate to attenuate the post development peak-discharge rates to pre-development discharge rates, both for the 100-year storm, and the conventional design frequency of the downstream drainage system.

Routing calculations should be used to demonstrate that the storage volume is adequate. If sedimentation during or after construction causes loss of detention volume, design dimensions must be restored before completion of the project and by regular maintenance. For detention basins, all detention volume must be drained within 72 hr to prevent excessive saturation of embankment material. Additional detention storage may be needed to meet water-quality requirements of environmental regulations (see Chapter 16 - Permanent Water Quality).

12.4.4 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.

General

Grading is important to ensure that adequate storage volume and an aesthetic appearance are provided in the storage facility. But, it is critical to the safety of people who live near the facility or utilize the facility for recreational purposes. As a general rule, slopes should be as flat, and depths as shallow as site conditions and safety considerations allow. If a person were to fall into the facility, slopes should be flat enough that they can easily climb out. Slope terracing is sometimes effective. Slope stabilization should be accomplished with vegetation or other materials which are traversable when wet. The following is a discussion of the general grading and depth criteria for storage facilities:

- The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume.
- Vegetated embankments must be less than 20 ft high and have side slopes no steeper than 3:1 (horizontal to vertical) and follow federal/state dam safety regulations. Slopes flatter than 4:1 are preferred.
- Riprap-protected embankments must be no steeper than 2:1. Geotechnical slope-stability analysis is recommended for embankments greater than 10 ft high, and is mandatory for embankment slopes steeper than those given above.
- A minimum freeboard of one foot above the 100-year design storm high-water elevation must be provided for impoundment depths of less than 25 ft.
- Impoundment depths greater than 25 ft or volumes greater than 50 acre-ft are subject to the requirements of the National Dam Safety Program (see Section 12.5), 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.

Detention Pond Depth and Grading

Detention facilities should be provided only where they are shown to be beneficial by hydrologic, hydraulic, and cost analysis. Areas above the normal high-water elevations of storage facilities should be sloped a minimum of 2% toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. A small paved apron should be provided around the outlet works to allow maintenance access, and prevent vegetation from clogging the release structure. A low-flow or trickle channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing-water conditions.

Retention Pond Depth and Grading

The maximum depth of permanent storage facilities is determined by site conditions, design constraints, and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds without creating increased potential for anaerobic bottom conditions should be considered. A depth of 5 to 10 ft is generally reasonable unless fishery requirements dictate otherwise. Aeration may be required in permanent pools to prevent anaerobic conditions. Where aquatic habitat is required, Colorado Parks and Wildlife should be contacted for site-specific criteria, relating to such things as depth, habitat, and bottom and shore geometry.

12.4.5 Outlet Works

Outlet works selected for storage facilities typically include a principal spillway and an emergency spillway. Outlet works can take the form of combinations of drop inlets, culvert pipes, overflow weirs, stand pipes and orifices.

Perforated riser pipes are discouraged for use as principal spillways because of clogging problems. Curb openings may be used for parking lot storage. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet.

For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood used to size an emergency outlet is the 100-year flood. The sizing of a particular outlet work must be based on results of reservoir-routing calculations.

12.4.6 Location

In addition to controlling the peak discharge from the outlet works, storage facilities 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 has on combined hydrographs in downstream locations. For all storage facilities, channel-routing calculations must 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 must be assessed for detrimental effects on downstream areas.

12.5 NATIONAL DAM SAFETY PROGRAM

National responsibility for the promotion and coordination of dam safety lies with the Federal Emergency Management Agency (FEMA). Responsibility for administration of the provisions of the NDSP is given to the states. Rules and regulations relating to applicable dams are promulgated by the Colorado Division of Water Resources.

The State of Colorado has had an active dam-safety program since 1881, and since then has maintained a degree of jurisdiction over the construction and safety of all dams within the state, including federal dams.

Colorado law provides that a notice of construction is required for all storage structures, and that all jurisdictional dams must have plans and specifications approved by the State Engineer prior to construction. A jurisdictional dam is defined as a dam which:

- Impounds water above the elevation of the natural surface of the ground, creating a reservoir with a capacity of more than 100 acre-ft; or
- Creates a reservoir with a surface area in excess of 20 acres at the high water line; or
- Exceeds 10 ft in height, measured vertically from the elevation of the lowest point of the natural surface of the ground, along the longitudinal centerline of the dam, to the flowline crest of the emergency spillway.

For reservoirs created by excavation, the vertical height must be measured from the invert of the outlet works. The State Engineer has final authority over determination of the vertical height. If there is any doubt or uncertainty as to whether a dam is jurisdictional, or what design requirements are, the State Engineer should be consulted.

Information concerning the construction of a dam or highway work affecting the safety and operation of a reservoir dam is available at the Office of the State Engineer, Colorado Division of Water Resources.

12.6 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

12.6.1 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. Facilities should be designed to minimize the following maintenance problems typical of urban detention facilities:

- Weed growth;
- Grass and vegitation maintenance;
- Sediment 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. Some methods include:

- 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.
- 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 diameter 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.

12.6.2 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.

12.6.3 Design Considerations for Pedestrians

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 hr, or the area is permanently wet and has side slopes steeper than 1V:4H.
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 3 ft, or a flow velocity greater than 3 ft/s.
- Side slopes equal or exceed 1V:2H.

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 dry-retention areas with design depths in excess of 3 ft for 24 hr, unless the area is within a fenced, limited-access facility.

12.7 GENERAL DESIGN PROCEDURE

12.7.1 Data Needs

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

- Inflow hydrograph for all selected design storms;

- Stage-storage curve for a proposed storage facility (see Figure 12.1 for an example). For large storage volumes (e.g., for reservoirs), use acre-ft. Otherwise, use ft^3; and
- Stage-discharge curve for all outlet control structures (see Figure 12.2 for an example).

This data is used to route the inflow hydrograph through the storage facility to establish an outflow hydrograph (see Figure 12.3). Different basin and outlet geometries can be analyzed and selected based on the desired outflow hydrograph.

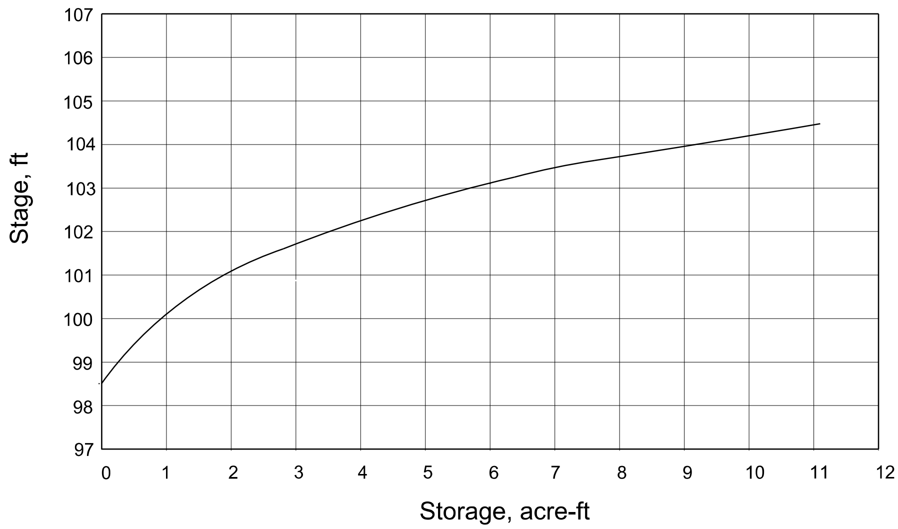

Figure 12.1 Example Stage-Storage Curve

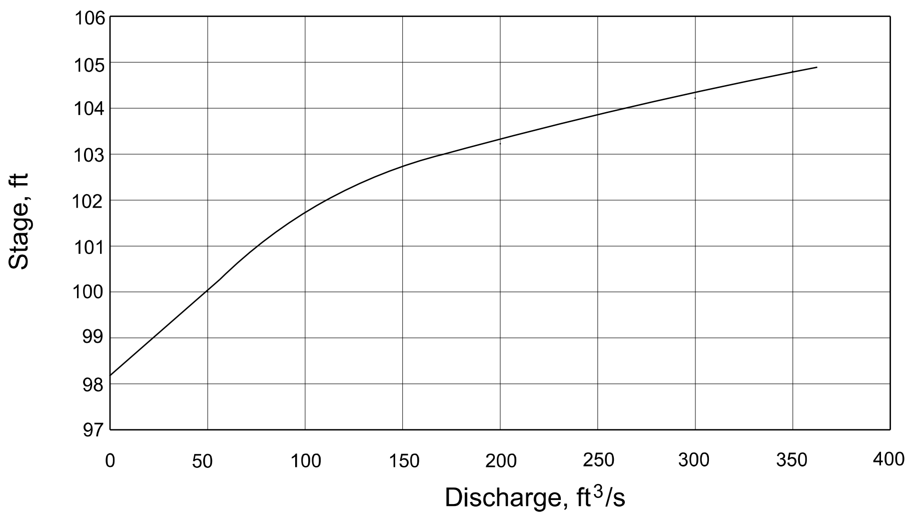

Figure 12.2 Example Stage-Discharge Curve

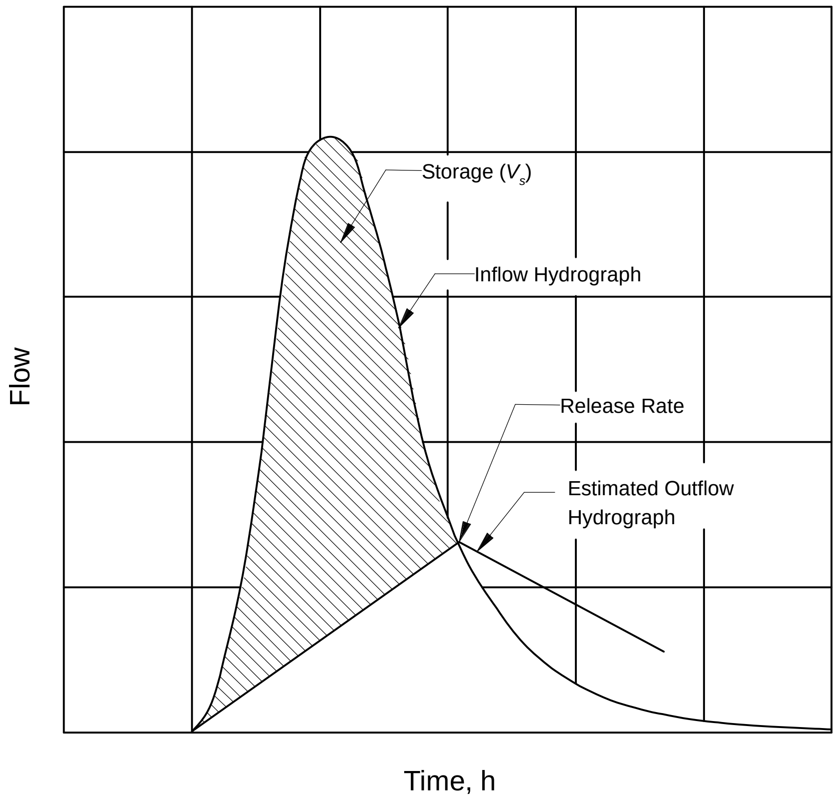

Figure 12.3 Inflow and Outflow Hydrographs from a Stream Reach

12.7.2 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 or grading plan, 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 be used on non-geometric areas. The average-end area formula is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \tag{12.1}$$

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:

$$V = d/3\ [A_1 + (A_1A_2)^{0.5} + A_2] \tag{12.2}$$

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

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)ZD^2 + 4/3\ Z^2D^3 \tag{12.3}$$

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.

Estimating the trial dimensions of a basin for a given basin storage volume can be accomplished by rearranging Equation 12.3 as shown in Equation 12.4:

$$L = \left\{-ZD(r+1) + \left\{(ZD)^2(r+1)^2 - 5.33(ZD)^2 r + \left[(4r\ V)/D\right]\right\}^{0.5}\right\}/2r \qquad (12.1)$$

12.7.3 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two spillways, principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design flood, without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet. 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 the FHWA's *Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual,* (HEC 22) and *Hydraulic Design Series (*HDS*) 2, Highway Hydrology*.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway should be designed considering the potential threat to downstream life and property if the storage facility fails.

The stage-discharge curve should reflect the discharge characteristics of both the principal and emergency spillways. Develop a composite stage-discharge curve, which combines the discharge rating curve for all components of the outlet control structure. Figure 12.4 illustrates an example composite stage-discharge curve.

12.7.4 Routing Procedure

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

$$\frac{\Delta S}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \qquad (12.5)$$

where: ΔS = change in storage, ft^3; Δt = time interval, min; I = inflow, ft^3; and O = outflow, ft^3. This relationship is illustrated graphically in Figure 12.5.

In Equation 12.5, subscript 1 refers to the beginning and subscript 2 refers to the end of the time interval. Equation 12.5 can be rearranged so that all known values are on the left side of the equation and all unknown values are located on the right-hand side of the equation, as shown in Equation 12.6. Now the equation with two unknowns, S_2 and O_2, can be solved with one equation:

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) \qquad (12\text{-}2)$$

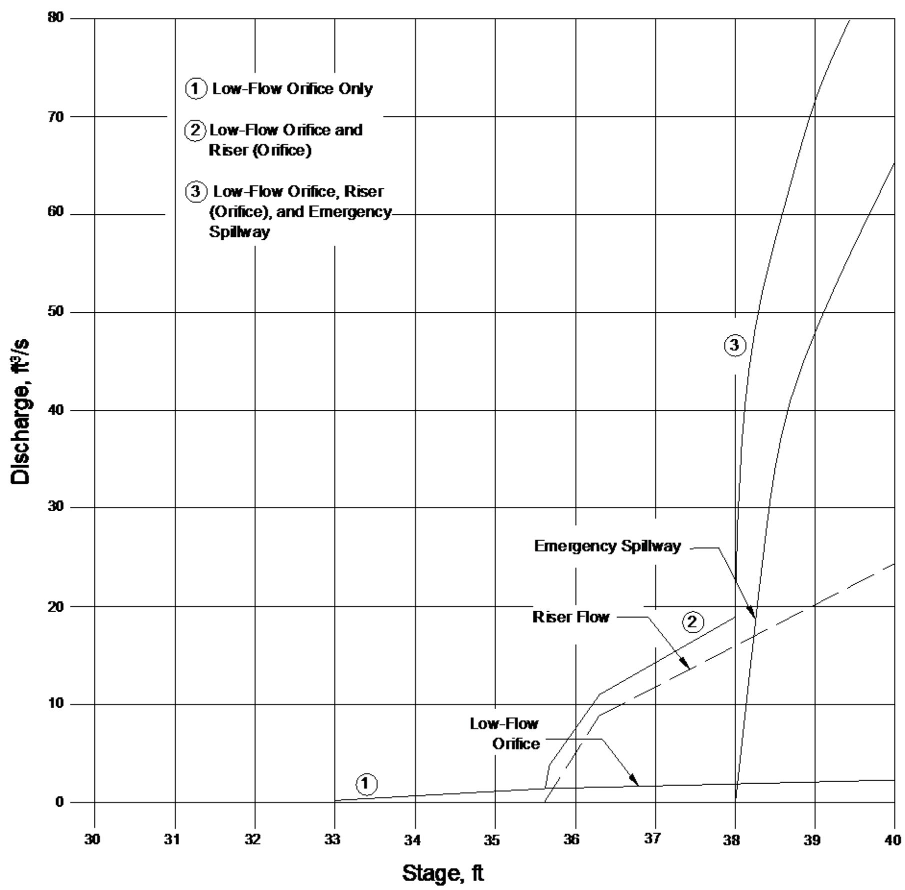

Figure12.4 Combined Stage-Discharge Relationship

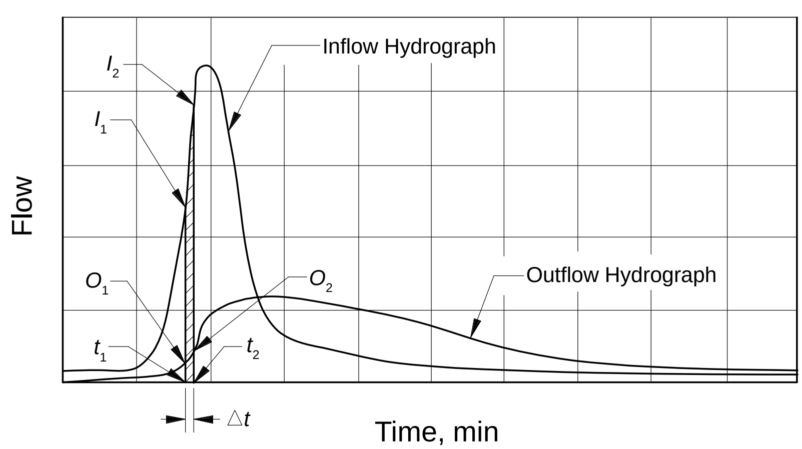

Figure 12.5 Routing Hydrograph Schematic

12.7.5 Preliminary Detention Calculations

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 12.6.

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

$$V_S = 0.5T_i\,(Q_i - Q_o) \tag{12.7}$$

where: V_S = storage volume estimate, ft^3; Q_i = peak inflow rate, ft^3/s; Q_o = peak outflow rate, ft^3/s; Ti = duration of basin inflow, s.

Any consistent units may be used for Equation 12.7.

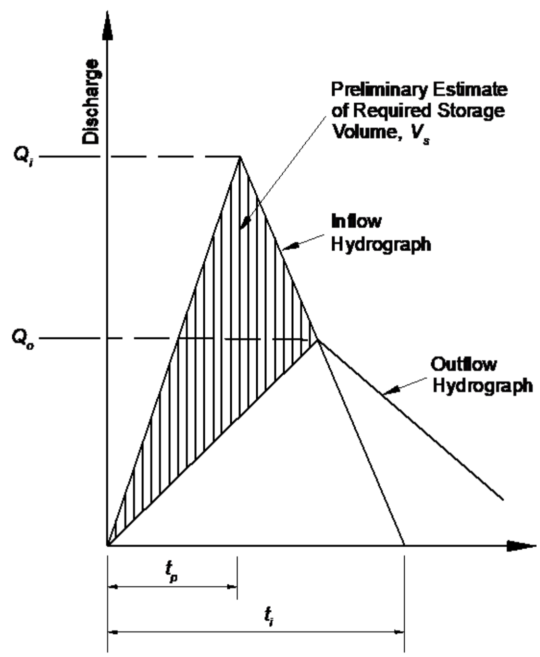

Figure 12.6 Triangular Shaped Hydrographs (for preliminary estimate of required storage volume)

Preliminary Basin Dimensions

Use the following procedure to develop the preliminary basin dimensions:

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

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

12.7.6 Step-by-Step Routing Procedure

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

Step 1

Compute an inflow hydrographs for runoff from the design storms using the procedures outlined in Chapter 7 - Hydrology. Both pre- and post-development hydrographs are required for the design storms. Only the post-development hydrograph is required for runoff from the 100-year storm. The probable-maximum flood should also be determined if failure of the storage facility could lead to loss of life or significant damage to downstream property.

Step 2

Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1. 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. The maximum storage requirement calculated from Step 2 should be used.

Step 4

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

Step 5

Perform routing calculations using inflow hydrographs from Step 1 and Equation 12.6 to check the preliminary design. If the developed peak discharge from the design storms exceeds the historic peak discharge, or if the peak stage varies significantly from the estimated peak stage from Step 4, revise the estimated volume and return to Step 3.

Step 6

Consider emergency overflow from runoff due to the 100-year or larger post development storm and established freeboard requirements.

Step 7

Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph will not cause downstream flooding problems or property damage. 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 percent 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 involves iterative reservoir routing calculations to obtain a final facility configuration and design of outlet works. HEC-22, Example 8.9 uses the USGS Nationwide urban hydrograph to compute runoff for existing and proposed conditions. HEC-22, Example 8.3 provides step-by-step hand calculations.

12.8 OUTLET WORK HYDRAULICS

Different structures are used at the outlet of detention facilities. Various weirs, orifices, grated-inlet structures and culvert pipes are used as spillways. The principal spillway is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway.

An emergency spillway is required on all detention ponds. It is sized to provide conveyance for floodwater during a flood that exceeds the design capacity of the principal spillway. The emergency spillway should minimize damage to storage facility embankment and downstream facilities during flooding. Design of this spillway should consider the potential threat to downstream life and property if the storage facility were to fail.

The principal spillway can be an orifice, sharp-crested weir, or a culvert pipe.

Orifice

The orifice may be a small plate used to reduce the opening into a culvert or inlet. Flow through an orifice can be determined using the basic orifice equation:

$$Q = C_d A\,[2g\,(h + k\,d/2)]^{0.5} \tag{12.8}$$

where Q = discharge through the outlet, cfs; C_d = discharge coefficient for the orifice; A = area of the orifice opening, ft^2; g = acceleration due to gravity, ft/s^2; h = water depth above the invert of the orifice, ft; k = -1.0 for vertical and +1.0 for horizontal orientation; d = diameter (or its equivalent) of the orifice, ft. Equation 12.8 is limited to conditions where the water depth above the orifice invert h is greater than twice the equivalent orifice diameter.

A pipe smaller than 12 inches may be analyzed as a submerged orifice if H/D is greater than 1.5, where D = diameter of pipe, ft; and H = head on pipe, from the center of pipe to the water surface. For square-edged entrance conditions:

$$Q = 0.6A(2gH)^{0.5} \tag{12.9}$$

Weirs

Various types of weirs shown in Figures 12.7 through 12.10 are also used as outlet structures. Sharp-crested weirs are sometimes used as principal spillways, and broad-crested weirs are frequently used as emergency spillways. Orifice openings operate as sharp-crested weirs under shallow-water conditions. There are several different equations for determining flow rates through weir structures depending on weir shape, and downstream flow conditions. Discussion of these equations and their application can be found in the AASHTO *Drainage Manual* or other hydraulics texts.

Discussion of culvert hydraulics is provided in Chapter 9 - Culverts, of this manual. Information on discharge characteristics of grated inlets is provided in FHWA's *Hydraulic Engineering Circular No. 12, Drainage of Highway Pavements,* (HEC-12), or Chapter 13 - Storm Drains.

Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure 12.7. The discharge equation for this configuration is:

$$Q = [(1.805 + 0.221(H/H_c)]\ L\ H^{1.5} \tag{12.10}$$

where Q = discharge, ft^3/s; H = head above weir crest, excluding velocity head, ft; H_c = height of weir crest above channel bottom, ft; L = horizontal weir length, ft.

A sharp-crested weir with two end contractions is illustrated in Figures 12.4 and 12.5. The discharge equation for this configuration is:

$$Q = [(1.805 + 0.221(H/H_c)]\ (L - 0.2H)\ H^{1.5} \tag{12.11}$$

A sharp-crested weir will be affected by submergence when tailwater rises above the weir-crest elevation. As a result, discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is:

$$Q_S = Q_f\ (1 - (H_2 / H_1)^{1.5}\ )^{0.385} \tag{12.12}$$

where Q_S = submergence flow, ft^3/s; Q_f = free flow, ft^3/s; H_1 = upstream head above crest, ft; and H_2 = downstream head above crest, ft.

Broad-Crested Weirs

The equation generally used for the broad-crested weir is:

$$Q = C\ L\ H^{1.5} \tag{12.13}$$

where: Q = discharge, ft^3/s; C = broad-crested weir coefficient; L = broad-crested weir length, ft; and H = head above weir crest, ft.

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction, and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest. This gives the maximum C value of 3.087. For sharp corners on a broad-crested weir, a minimum C value of 2.52 should be used. Additional information on C values as a function of weir-crest breadth and head is given in Table 12-3.

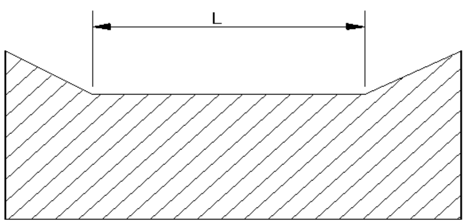

Figure 12.7 Sharp-Crested Weir
(No End Contractions)

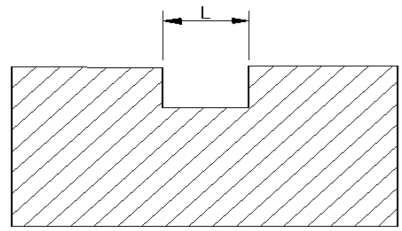

Figure 12.8 Sharp-Crested Weir
(Two End Contractions)

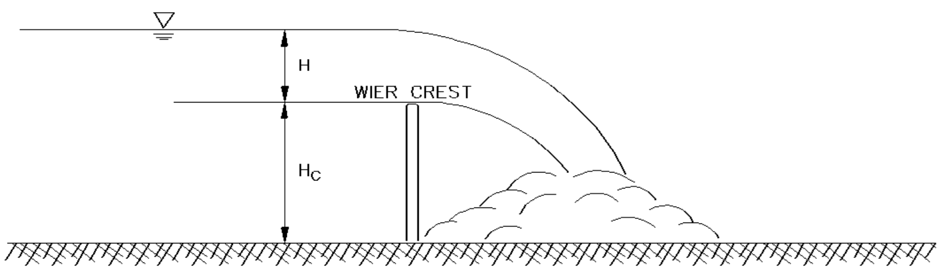

Figure 12.9 Sharp-Crested Weir and Head

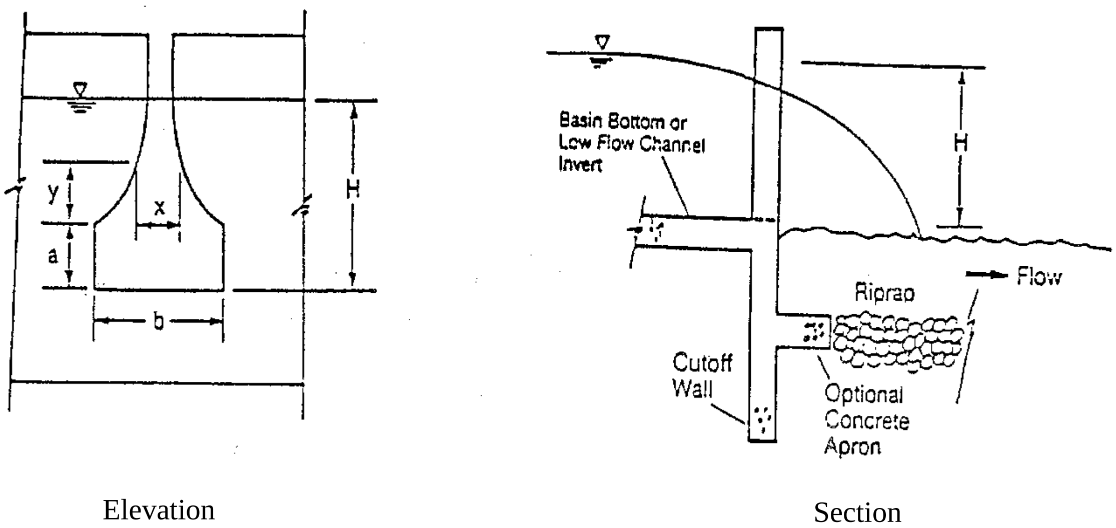

Figure 12.10 Proportional Weir Dimensions

Table 12.2 Broad-Crested Weir Coefficient *C* Values as a Function of Weir Crest Breadth and Head

Measured Head, *H*[1] (ft)	**Breadth of the Crest of Weir (ft)**										
	0.5	**0.75**	**1**	**1.5**	**2**	**2.5**	**3**	**4**	**5**	**10**	**15**
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 2.5*H* upstream of the weir.

12.9 PRELIMINARY DETENTION CALCULATIONS

12.9.1 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak-flow reduction can be obtained by the following regression-equation procedure:

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

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

$$V_S / V_r = [1.291(1 - Q_o / Q_i)^{0.753}] / [(t_b / t_p)^{0.411}] \qquad (12.14)$$

where: V_S = volume of storage, ft^3; V_r = volume of runoff, ft^3; Q_o = outflow peak flow, ft^3/s; Q_i = inflow peak flow, ft^3/s; t_b = time base of the inflow hydrograph, *hr* (determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak); t_p = time to peak of the inflow hydrograph, *hr*.

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

12.9.2 Peak-Flow Reduction

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

1. Determine the following:
   - Volume of runoff, V_r
   - Peak-flow rate of the inflow hydrograph, Q_i
   - Time base of the inflow hydrograph, t_b
   - Time to peak of the inflow hydrograph, t_p
   - Storage volume, V_S

2. Calculate a preliminary estimate of the potential peak-flow reduction for the selected storage volume using the following equation:

$$Q_o / Q_i = 1 - 0.712(V_S / V_r)^{1.328}(t_b / t_p)^{0.546} \qquad (12.15)$$

where: Q_o = outflow peak flow, ft^3/s; Q_i = inflow peak flow, ft^3/s; V_S = volume of storage, ft^3; V_r = volume of runoff, ft^3; t_b = time base of the inflow hydrograph, *hr* (determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak); t_p = time to peak of the inflow hydrograph, *hr*.

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

12.10 ROUTING CALCULATIONS

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

The Puls Method below is used to perform routing through a reservoir or storage facility, but other methods of reservoir routing are acceptable.

Step 1

Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility. An example stage-storage curve is shown in Figure 12.1, and a stage-discharge curve is shown in Figure 12.2.

Step 2

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

Step 3

Use the storage-discharge data from Step 1 to develop storage-characteristic curves that provide values of $S \pm (O/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 12.3.

Table 12.3 Storage Characteristics

(1) Stage (ft) L__= L__	(2) Storage[1] (acre-ft)	(3) Discharge[2] (cfs)	(4) Discharge (acre-ft/hr)	(5) S-(O/2) Δt (acre-ft)	(6) S+(O/2) Δt (acre-ft)

[1] Obtained from the Stage Storage Curve.
[2] Obtained from the Stage Discharge Curve

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 12.1).

Step 5

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

$$S_2 + (O_2/2)\Delta t = [S_1 - (O_1/2)\Delta t] + [(I_1 + I_2)/2]\Delta t \tag{12.16}$$

where: S_2 = storage volume at Time 2, ft^3; O_2 = outflow rate at Time 2, ft^3/s; Δt = routing time period, s; S_1 = storage volume at Time 1, ft^3; O_1 = outflow rate at Time 1, ft^3/s; I_1 = inflow rate at Time 1, ft^3/s; I_2 = inflow rate at Time 2, ft^3/s. Other consistent units are equally appropriate.

Step 6

Enter the storage characteristic curve at the calculated value of $S_2 + (O_2/2)\Delta t$ determined in Step 5 and read 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.

12.11 DRY POND (DETENTION BASIN)

Detention basins are depressed areas that store runoff during wet weather, but are dry the remainder of the time. They are very popular because of their comparatively low cost, few design limitations, ability to serve large and small watersheds, and ability to be incorporated into other uses (e.g., recreational areas). Table 12.10 summarizes considerations for a dry pond.

Detailed design procedures can be found in the Chapter 16 - Permanent Water Quality, Volume 2, Chapter 12 of the Denver Regional Council of Governments, *Urban Drainage and Flood Control District Criteria Manual*, HEC-22 or the AASHTO *Drainage Manual*.

Table 12.4 Summary of Considerations for a Dry Pond

Quality	Detain WQV for 30 hr (minimum 3-inch orifice)
Quantity	Control 2- and 10-yr peak flows and maintain non-erosive velocity
Shape	3:1 length-to-width ratio; 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 (1V:3H); 2% bottom slope to prevent ponding; sediment forebay to reduce maintenance-safety requirements (depth and perimeter ledges)
Pollutant Removal	Moderate

12.12 WET POND (RETENTION BASIN)

A wet pond is very similar to a dry detention basin in that it detains stormwater, but it is different in that it maintains a permanent pool during dry weather. Wet ponds are usually more expensive than dry detention basins and usually serve large watersheds. Because of their permanent pool, they may also have recreational benefits. Table 12.11 summarizes considerations for a wet pond.

Detailed design procedures can be found in the Chapter 16 - Permanent Water Quality, Volume 2, Chapter 12 of the Denver Regional Council of Governments, *Urban Drainage and Flood Control District Criteria Manual*, HEC-22 or the AASHTO *Drainage Manual*.

Table 12.5 Summary of Considerations for a Wet Pond

Quantity	Control 2- and 10-yr peak flows
Shape	3:1 length-to-width ratio; wedge shaped (wider at outlet); permanent pool depth from 5 ft - 10 ft; perimeter ledges
Maintenance	Inspect once a year, preferably during wet weather; mow at least twice a year; remove sediment every 5 to 10 years
Safety	Fence around pond; provide shallow 2-ft deep safety ledge around pond; post signs
Other considerations	Side slopes provide easy maintenance access, 1V:3H; perimeter vegetation; sediment forebay; provide valve to drain pond for maintenance
Pollutant Removal	Moderate to high

12.13 INFILTRATION CONTROLS

12.13.1 Introduction

Infiltration controls are best management practices (BMPs) where the primary discharge of stormwater is to the groundwater table. These include infiltration trenches, infiltration basins, vegetated filter strips, grassed swales, wetlands and porous pavement. In some cases, the stormwater is intercepted after it has infiltrated a few feet by an underdrain and is discharged to a storm sewer or surface water. One of the primary concerns with the use of infiltration BMPs is the risk of groundwater contamination. That is why there should be at least 2 to 5 ft between the bottom of the facility and the seasonable high-water table, and 5 ft to the underlying bedrock. Another factor is the residence time in the facility. It is recommended that the first-flush stormwater be infiltrated within 24 to 72 hr. The infiltration rate is directly related to the soil type and disposition. A soil investigation should be performed at all facility locations prior to construction. Table 12.12 provides some considerations for evaluating an infiltration control.

Detailed design procedures can be found in Chapter 14 of the AASHTO *Drainage Manual* and HEC-22. In addition, design procedures for several of these infiltration controls can be found in Chapter 16 -Permanent Water Quality.

Table 12.6 Summary of Considerations for an Infiltration Facility

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

12.13.2 Infiltration Trench

An infiltration trench (see Figure 12.11) is a facility where a trench is excavated and filled with a porous medium. Stormwater is stored in the voids of the fill material until it can infiltrate. In a variation of this design, the stormwater is collected by an underdrain pipe after the stormwater has been detained and filtered by the trench. Infiltration trenches can be used in median strips or adjacent to parking lots. The bottom of the infiltration trench should be below the frost line and should be 3–5 ft above bedrock and the seasonally high groundwater table.

12.13.3 Infiltration Basin

An infiltration basin looks very similar to a dry pond (see Figure 12.13). Stormwater from smaller, more-frequent storms infiltrates through the bottom of the basin. Larger storms can be controlled through infiltration and/or by a "peak-shaving" outlet. The most important consideration for an infiltration basin is keeping the bottom from clogging with sediment. The clogging of basins, along with the overestimating of their infiltration rates, has led to the failure of many infiltration basins.

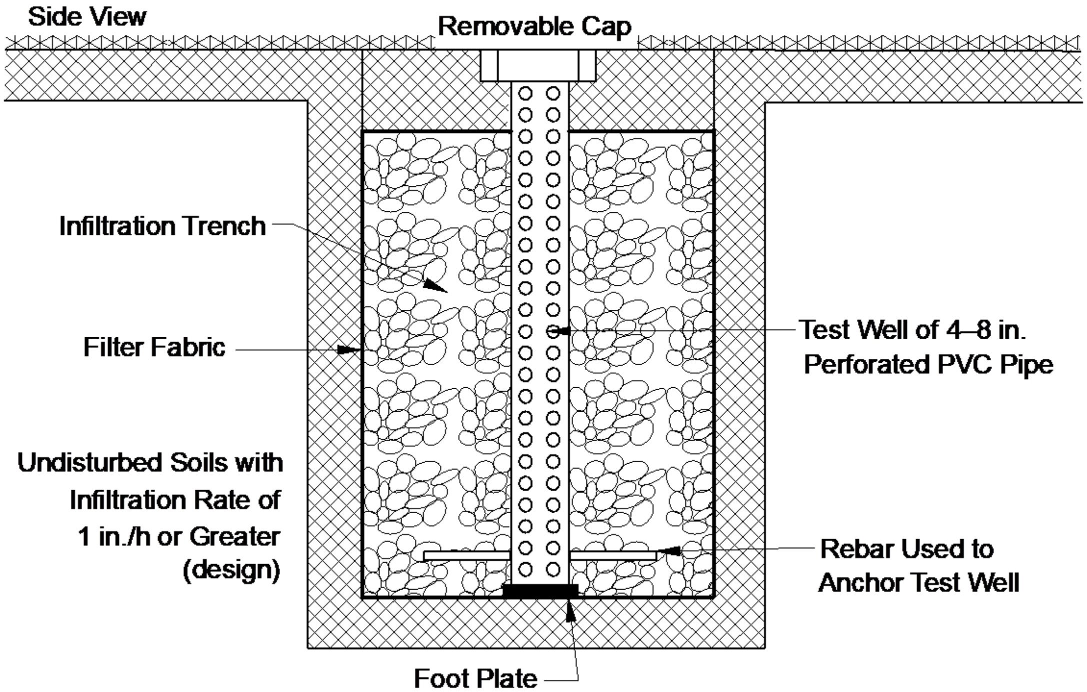

Figure 12.11 Infiltration Trench with Observation Well

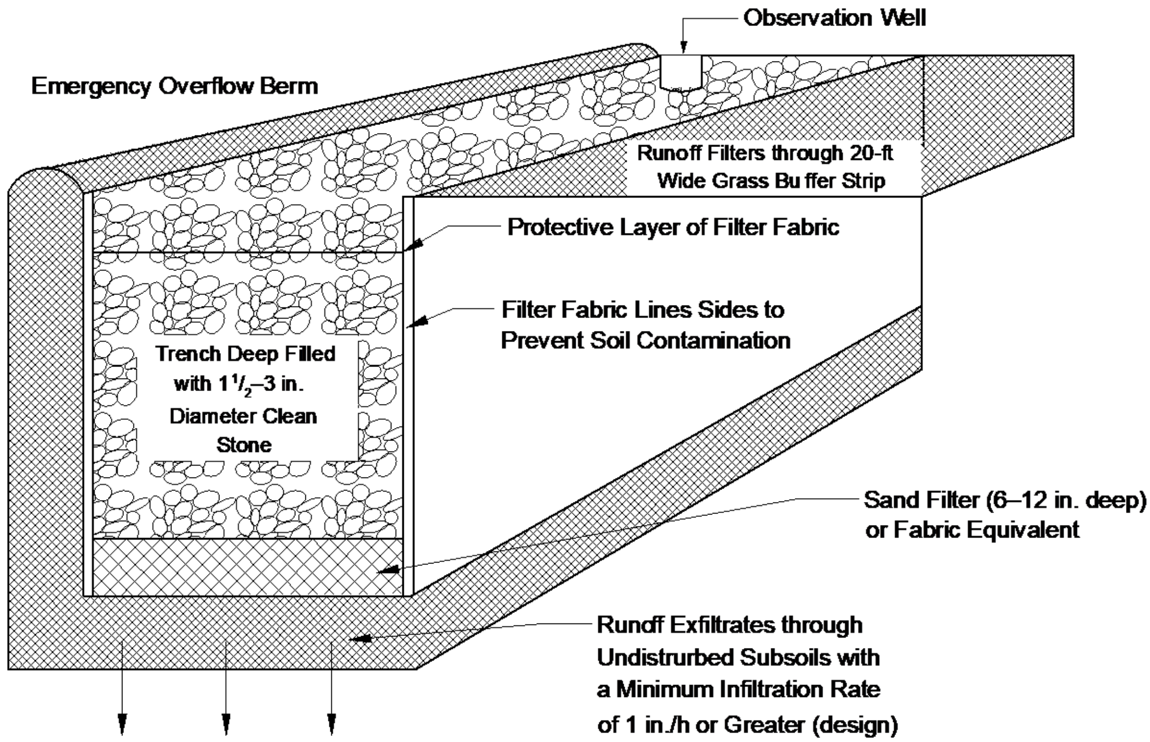

Figure 12.12 Infiltration Trench

12.13.4 Vegetated Filter Strip

A filter strip is a vegetated area designed to accept sheet flow. While flowing over the strip, stormwater is filtered by the vegetation, infiltrated, and detained. The most common cause for failure of filter strips is runoff bypassing the strip through eroded channels. If the stormwater is not evenly distributed over the entire strip, a channel could form, causing the strip to lose effectiveness. To prevent this channelization, a level spreader can be used, as shown in Figure 12.14.

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

Filter strips should be constructed of dense, soil-binding, deep-rooted, water-resistant plants. They are usually constructed of grass, but forested strips are also feasible (they can have higher pollutant removal rates, but should be longer because of their lack of cover and susceptibility to erosion). For the filter strips to be effective, their slope should be no more than 5%, and their length should be at least 20 ft. Figure 12.15 was developed by Wong and McCuen for determining the required length of a grassed filter strip. If the slope of the strip, roughness coefficient (Manning's *n*) and desired trap efficiency are known, the length required can be found by using Figure 12.15.

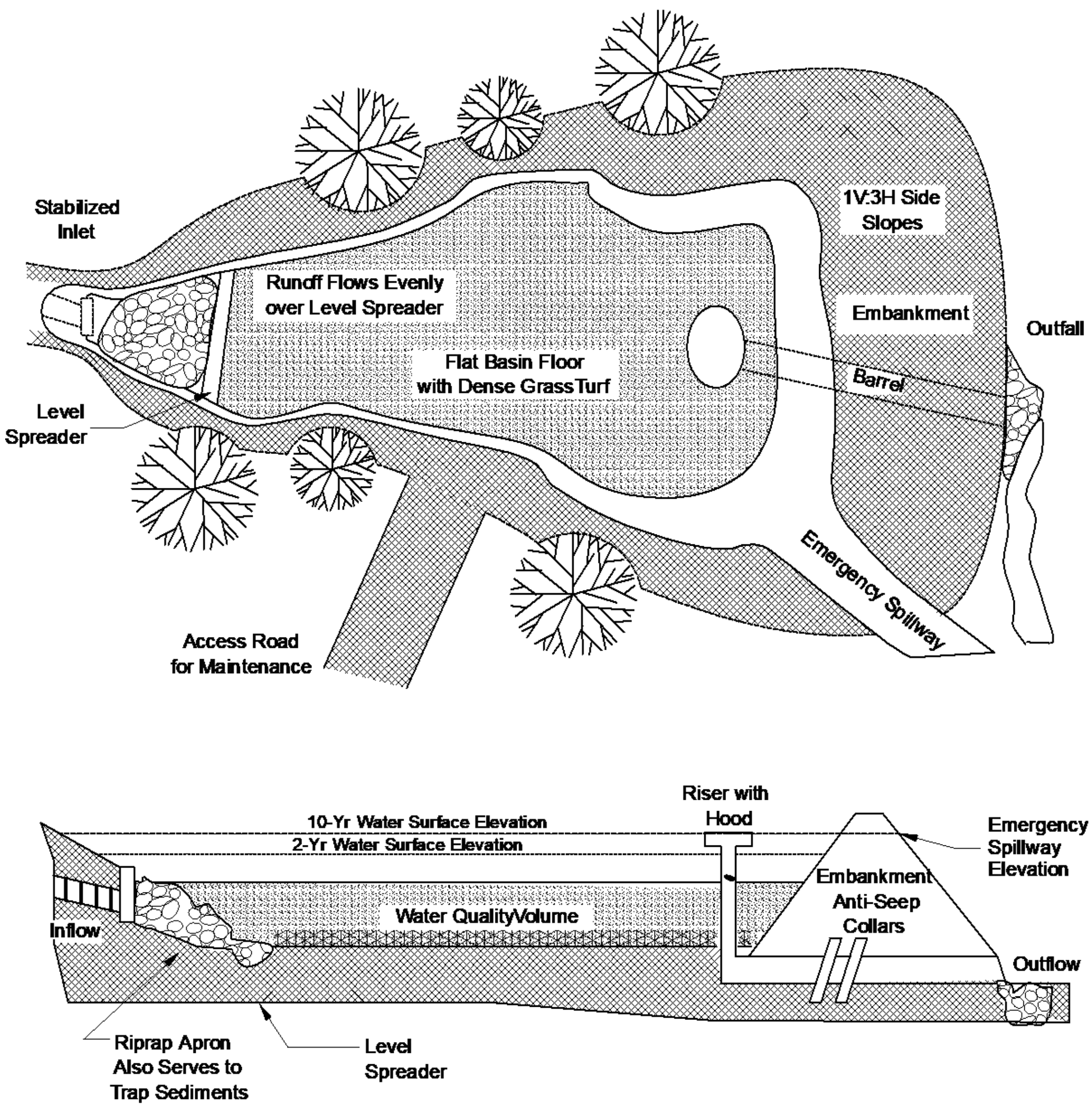

Figure 12.13 Infiltration Basin, source*:* Schueler

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

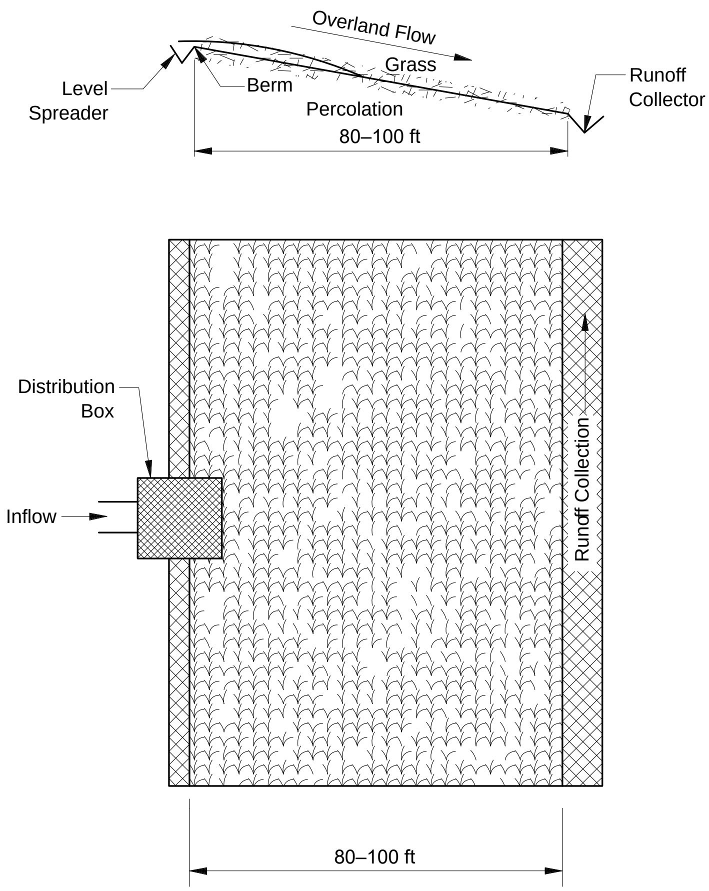

Figure 12.14 Level Spreader System

12.13.5 Grassed Swale

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

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

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

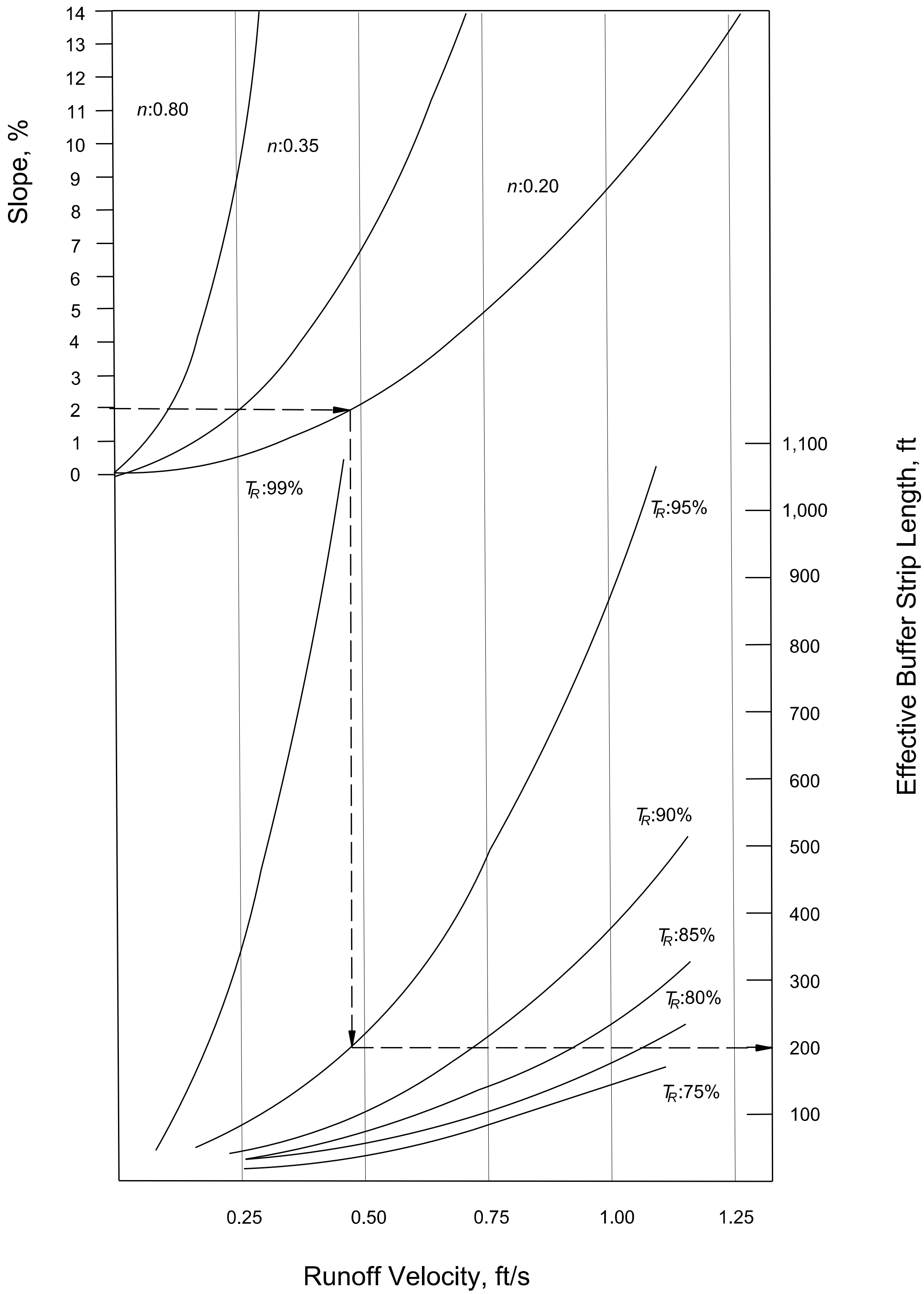

Figure 12.15 Removal Rates (R_R) for Buffer Strips, source: Wong

12.13.6 Wetlands

Wetlands have the ability to remove many pollutants and detain stormwater. However, the processes that occur in wetlands are not fully understood, and the amount of wetland area required to treat stormwater can be very large. It is recommended that wetlands and marshes be used in conjunction with other BMPs, such as on the bottom of dry ponds and on the fringes of wet ponds. Although a substantial amount of information is available on using wetlands as a final treatment process of wastewater, very little is known about using wetlands for treating stormwater.

A report by Marble provided guidelines for designing replacement wetlands. With regard to using wetlands for stormwater management, Marble reported that urban runoff is a good source of nutrients for the development of wetlands, and that wetlands downstream of an impoundment may have reduced aquatic diversity because of reductions in the outflow detritus. Marble further stated that wetlands have the ability to remove sediments and toxins through sedimentation. However, the loadings of toxins and sediments should be low-to-moderate, and the ratio of wetland area to watershed area should be kept high. The functions of wetlands with regard to water quality are very complex. Hemond and Benoit noted the following:

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

The very limited number of studies undertaken on the use of wetlands for stormwater management indicate a wide disparity in the efficiency of wetlands to remove pollutants. A study by Martin suggested that wetlands, when used in conjunction with another BMP (e.g., a wet detention pond), can be quite effective in treating highway-stormwater runoff. Because CDOT is required to replace wetlands, the idea of using these constructed wetlands for stormwater management appears prudent. However, more field test and monitoring data needs to be collected and analyzed before appropriate design guidelines can be developed.

12.13.7 Porous Pavement

Porous pavement is an infiltration practice in which a stone "reservoir" is placed under a layer of open-graded asphalt pavement that contains no fines, yielding a pavement with approximately 16% voids that allows water to infiltrate. Under the asphalt is a stone reservoir that stores stormwater. This type of facility is not to be confused with an open-graded, bituminous-concrete surface course used to reduce water filming on highway surfaces. Figure 12.16 shows a cross section of a typical design. Porous pavement is generally not recommended for highway uses, but is more appropriate for parking lots and other low-traffic areas. There have also been structural problems and clogging problems in some applications. Because the stone reservoir is located under the asphalt layer, maintenance can be difficult and costly. Winter salts and other abrasives should not be applied to the facility because they may cause clogging. Vacuuming on a regular basis is recommended.

12.14 LAND-LOCKED RETENTION

12.14.1 Introduction

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

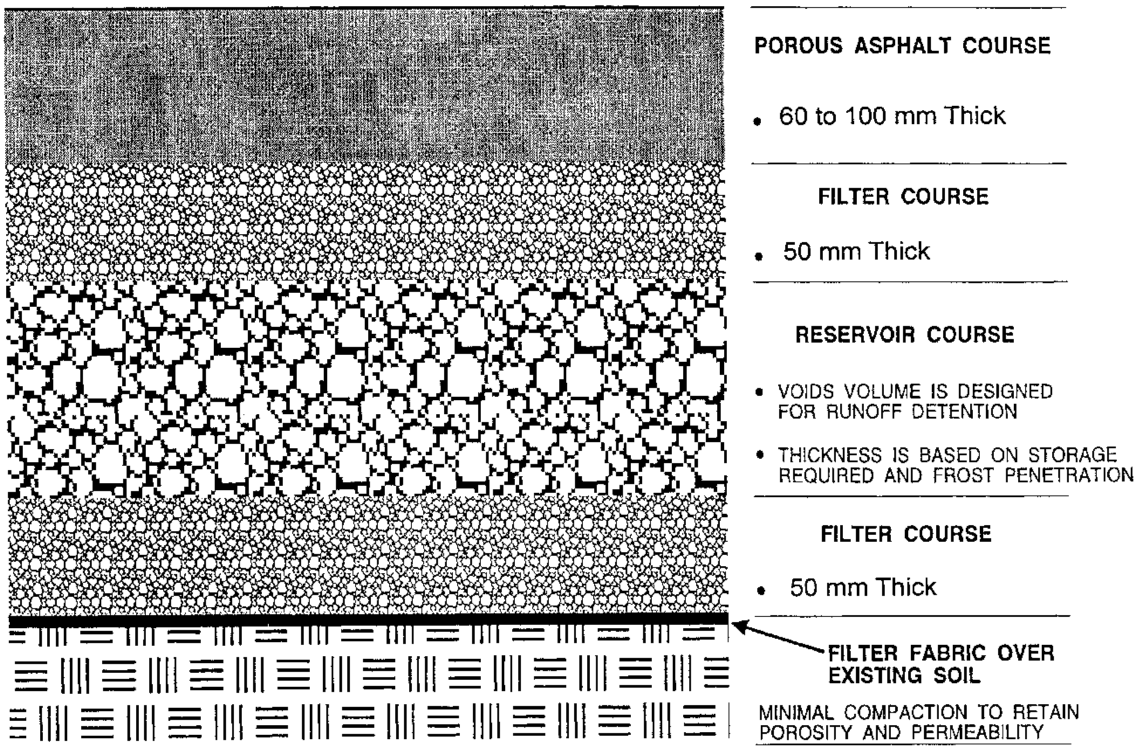

Figure 12.16 Porous Pavement

12.14.2 Mass Routing

The steps presented below illustrate a mass-routing procedure:

A mass flow routing procedure for the analysis of land-locked retention areas is illustrated in Figure 12.17. The step-by-step procedure follows:

Step 1 Obtain cumulative rainfall data for the design storm. If no local criteria are available, a 100-year, 10-day storm is suggested (HEC-22).

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

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

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

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

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

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

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

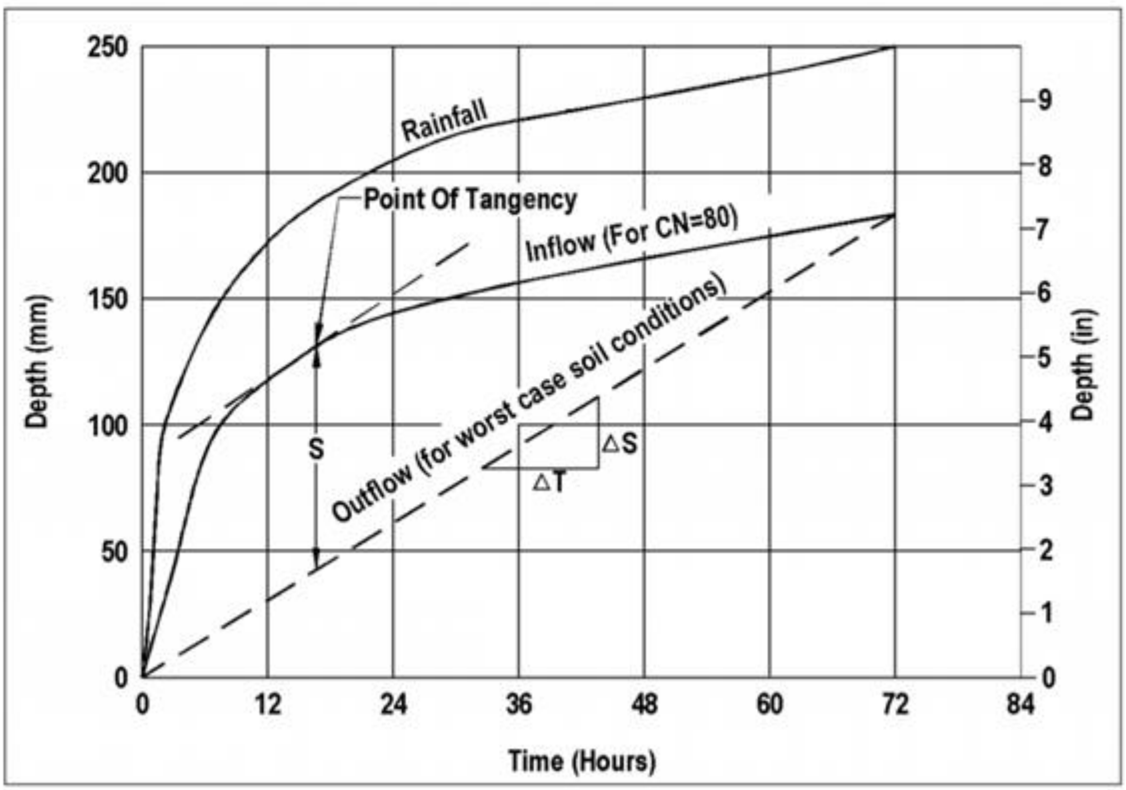

Figure 12.17 Mass Routing Procedure

12.14.3 Water Budget

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

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

12.15 SOFTWARE FOR DESIGNING STORAGE FACILITIES

Current software for designing storage facilities is listed in the following table. The software listed is public domain software, or software CDOT has purchased.

Table 12.7 Software for Reservoir Routing

Software Name	Features	Source
FHWA Hydraulic Toolbox 4.20	The FHWA Hydraulic Toolbox Program is a stand-alone suite of calculators that perform routine hydrologic and hydraulic computations (see the software section of Chapter 8 - Channels). The Detention Basin Analysis option uses procedures from HEC-22. Hydrograph coordinates, stage-storage information, and culvert performance curves can be input by hand or copied from a spreadsheet. The output is a routed hydrograph.	FHWA website
WMS 10.1	Routing is discussed in the help topic "Storage," which focuses on including storage in a HEC-1 run. A similar process is followed as that for including storage in a TR-20 run. After the Hydrology Module has been selected, an outlet point is established in the model. If the reservoir-routing option is specified, then a method for volume and a method for outflow must be defined.	Aquaveo website
WMS 10.1, Detention Basin Hydrograph Routing	The Detention Basin Hydrograph Routing Calculator is a feature of the WMS Hydrologic Modeling Module (see WMS discussion in the software section of Chapter 7 - Hydrology). After an outlet has been established, the calculator can be chosen. Select "Calculators" on the menu tool bar to display available calculators. The calculator performs a reservoir routing after a user-defined volume/elevation curve for storage and user-defined headwater-elevation curve for the outlet have been entered.	Aquaveo website
HEC-HMS 4.2.1	See the Hydrologic Modeling System (HEC-HMS) summary in the software section of Chapter 7 - Hydrology. Chapter 6 of the User's Manual discusses how to model a reservoir.	HEC website
PondPack V8i	PondPack is commercially-available software for the design of storage facilities. Rainfall or hydrograph information can be used for input. The storage facility can be specified, or designed. The outlet culvert hydraulics are calculated using HDS-5 procedures.	Bentley.com

The effects of existing storage facilities on flood peaks can be evaluated using the FHWA Hydraulic Toolbox or WMS Detention Basin Analysis. If the inflow hydrograph and proposed storage-facility size is known, the WMS Detention Basin Hydrograph Routing Calculator can be used to determine, by trial-and-error, the sizes and types of outlets needed. If a culvert outlet is entered, HY-8 can be used to determine a headwater-elevation curve that can be imported into the calculator to fine-tune the storage facility.

The software versions shown in Table 12.7 are the most recent when this manual was prepared. For current versions of software and documentation, the hydraulic engineer should consult the software source.

Available user and reference manuals are listed in the references. However, many software developers provide extensive help within the software in place of a user's manual.

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