CHAPTER 8

STORMWATER STORAGE FACILITIES

NOTE: All questions and comments should be directed to the Drainage Specialist, Design Support Area.

Revised January 2006

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8.1 INTRODUCTION/PURPOSE

8.1.1 Storage Overview and Acceptable Drainage Outlets

The traditional purpose of stormwater 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 increased flooding problems downstream. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows. This chapter provides general design criteria for detention/retention storage facilities, as well as procedures for performing basin sizing and reservoir routing calculations. Properly designed storage facilities may provide improved water quality benefits.

Examples of acceptable drainage outlets include, but are not limited to, the following:

- Natural watercourses. For highway purposes, a natural watercourse is defined as one that has not been altered artificially.
- County drains. Plans that involve discharging stormwater to county drain must be submitted to the County Drain Commissioner.
- Storm sewers or ditches owned by other public agencies. Approval to use these facilities must be obtained from the owner. When considering a sewer owned by others, an agreement will be needed. Reference Chapter 2, Legal Policy and Procedure. MDOT will evaluate operation and maintenance costs, outlet costs, and future capacity. (Storm sewers are discussed in Chapter 7, Road Storm Drainage Systems.)

Examples of unacceptable drainage outlets include, but are not limited to, the following:

- Natural depressions.
- Farm tiles unless with a recorded agreement.
- Sanitary sewers.
- Drainage course across private property unless drainage or flowage easements are obtained and recorded.
- Combined sewers, unless existing facilities, drain there by an agreement.

8.1.2 Computer Programs

Routing calculations needed to design storage facilities, although not extremely complex, are time-consuming and repetitive. The Modified Puls Method is one means of performing these calculations (Section 8.4.7). All storage facilities shall be designed and analyzed using reservoir routing calculations. To assist with these calculations, there are many available reservoir routing computer programs (See computer programs mentioned in Chapter 3, Hydrology).

8.2 DEFINITIONS

Following are some definitions of terms used in this chapter.

Auxiliary Spillway - A waterway around a dam, used to supplement the principal spillway in conveying extreme amounts of runoff.

Best Management Practice (BMP) - Structural devices or nonstructural practices (both temporary and permanent) that are designed to prevent pollutants from entering into stormwater flows, to direct the flow of stormwater, or to treat polluted stormwater flows.

Detention Basin - A basin or reservoir incorporated into the watershed to temporarily store runoff, thus reducing the peak of the runoff hydrograph. Water is released and no permanent pool remains.

First Flush Basin - A basin designed to capture the initial 1/2-inch to 1-inch of runoff from a storm.

Infiltration Basins or Trenches - A basin or trench that discharges stored water into the ground.

Retention - The process of collecting and holding surface and stormwater runoff.

Retention Basin - A basin or reservoir wherein water is stored and which regulates a flood. It has a controlled outlet. The stored water is discharged by infiltration, injection or dry wells, or by release to the downstream drainage system during and after a storm event. The release may be through a gate-controlled gravity system or by pumping. The basin maintains a permanent pool elevation.

Storage Facility - A type of drainage control facility designed to hold water for a considerable length of time and then release it.

Structural Control - Use of engineered facilities and outlets to control the rate of stormwater runoff released from a storage facility. This may include using a principal and an auxiliary spillway.

Wetland - (The following definition is taken from Part 303 of NREPA Act 451.) Land characterized by the presence of water at a frequency and duration sufficient to support, and that under normal circumstances does support, wetland vegetation or aquatic life, and is commonly referred to as a bog, swamp, or marsh and which is any of the following:

- Contiguous to the Great Lakes or Lake St. Clair, an inland lake or pond, or a river or stream.

- Not contiguous to the Great Lakes, an inland lake or pond, or a river or stream; and more than 5 acres in size; except this subparagraph shall not be of effect, except for the purpose of inventorying, in counties of less than 100,000 population until the department certifies to the commission it has substantially completed its inventory of wetlands in that county.
- Not contiguous to the Great Lakes, an inland lake or pond, or a river or stream; and 5 acres or less in size if the department determines that protection of the area is essential to the preservation of the natural resources of the state from pollution, impairment, or destruction and the department has so notified the owner; except this subparagraph may be utilized regardless of wetland size in a county in which information given in the bullet above is of no effect; except for the purpose of inventorying, at the time.

A list of symbols and acronyms used in this chapter are given in Appendix 8-A.

8.3 POLICY AND DESIGN CRITERIA

8.3.1 Introduction

The use of storage facilities for stormwater management has increased dramatically in recent years. The benefits of storage facilities are water quality and water quantity control.

When an existing drainage outlet lacks capacity and cannot be economically increased, the discharge may still be adequately handled by designing a detention system that will meter out the discharge at a specified flow rate. Outflow rates may be regulated by local agency ordinances/regulations.

In order to meet National Pollution Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) permit requirements, a storage facility may be necessary. Detention basins are a structural Best Management Practice (BMP) method; see Chapter 9, Stormwater Best Management Practices, for more BMPs and discussion on environmental benefits.

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics. If water is impounded to a depth greater than 2 feet, it should be located outside the clear zone (defined in Section 7.01(DG) of the *MDOT Road Design Manual*). Analysis of storage facilities may 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, 1 percent chance (100-year) flood flow should be routed through the storage facility. The design data for storage facilities should include:

- Stage vs. discharge curves or table,
- Stage vs. storage curves or table,
- Grading and depth requirements, and
- Structural control location.

8.3.2 Water Quality Control (updated January 2004)

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

- Decreases downstream channel erosion.
- Controls sediment deposition and improves water quality through stormwater filtration.

Federal and state regulations have necessitated that MDOT re-evaluate the design practices used to control the pollutants contained in stormwater runoff from state highways. MDEQ, EPA, or FHWA may require the inclusion of stormwater runoff control measures as a mitigation for state and Federal environmental clearance and permits. Environmental regulations have increased emphasis on water quality and nonpoint source pollution. BMPs will be greatly expanded and required on projects. Under Part 91of NREPA, MDOT is an Approved Public Agency (APA), and as part of the NPDES Permit applications required from the Water Quality Act of 1988, it is necessary to provide soil erosion control measures on the design plans. The design engineers must provide copies of the plans with site-specific control measures along with other NPDES construction permit application information to the Geotechnical Services Unit, Construction and Technology Division.

The project manager should contact the Environmental Section of the Project Planning Division for status of environmental clearance in regard to mitigation measures for stormwater (See Chapter 9, Stormwater Best Management Practices [BMPs], Section 9.3.3).

The following general guidelines for controlling the pollution contained in stormwater runoff are applicable to virtually all highway situations. There are a number of low-cost drainage design principles and practices having significant potential for reducing pollutant loads from highway stormwater runoff. These relatively low-cost principles and practices can be incorporated into existing highway design procedures. They are intended to be used wherever practical and without the necessity of identifying a specific highway runoff contaminant problem. The principal concepts to consider when designing highway systems include:

- **Reduce Direct Discharges**: Avoid direct discharges of highway stormwater runoff to receiving waters wherever practical. Highway stormwater runoff should be routed through one, or a combination of, effective stormwater management measures including vegetation, detention, infiltration, or wetland systems prior to discharge to receiving waters.

  When possible, drains discharging directly to open water along bridge decks should be avoided.

- **Reduce Runoff Velocity**: Reducing the runoff velocity to a non-erosive level for the channel type decreases the transport of sediment and encourages sedimentation, especially bedload. The bedload contains larger soil particles that slide, roll, or bounce along the channel bottom. The methods for reducing the runoff velocity include reducing channel slopes, installing energy dissipaters (check dams, drop structures, baffles, and sedimentation basins), and by using vegetative controls (grassed waterways, overland sheet flow, etc.).

Typically, the sediment pollutant load is transported along pavement, curbs, and shoulders as suspended solids or attached to suspended solids in stormwater runoff. Therefore, pollutant reduction measures are usually intended to reduce the volume of particulates available for transport by runoff or to filter and settle out suspended solids.

8.3.3 Water Quantity Control

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

- 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.
- Maintenance of historic flow rates by controlled discharge from storage.

The 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:

- Existing rates for specific design conditions (i.e., post-development peak restricted to pre-development peak for a particular frequency of occurrence).
- Discharge capacity of the downstream drainage system.
- A specified value for allowable discharge set by local regulations.

Design storage volumes and discharges are outlined for each type of storage facility in Table 8-1.

Table 8-1 Storage Volume and Outlet Design Criteria Overview

<table>
<tr><th>Facility</th><th>Available Outlets</th><th>Design Peak Outlet Rate</th><th>Design Storage Elevation</th><th>Water Quantity or Quality Benefits</th><th>Water Quantity or Quality Limitations</th></tr>
<tr><td>Detention Basin (preferred)</td><td>Surface water or storm drain</td><td rowspan="2">Principal Spillway or Control Structure:
• 10% (10-year) Existing peak for enclosed systems and 4% (25-year) existing peak for open channel systems

Auxiliary and Principal Spillway (Emergency Overflow):
• 1% (100-year), routed post-development flow contained with 1.5 feet freeboard and within MDOT R.O.W.</td><td rowspan="2">10% - 4% (10- to 25-year) routed post-development flow contained below auxiliary spillway elevation</td><td>• Reduced peak flow released
• Some water quality benefit</td><td>• Less water quality benefit than retention (wet) pond</td></tr>
<tr><td>Retention Basin</td><td>Surface water or storm drain</td><td>• Reduced peak flow released
• Enhanced water quality benefit</td><td>• Higher liability than dry pond
• More volume required than dry pond</td></tr>
<tr><td>First Flush Basin</td><td>Surface water or storm drain</td><td>Principal Spillway or Control Structure:
• Release storage volume over time via a low level orifice (24 hours is the minimum). Maximum outflow depends upon receiving system.
Auxiliary and Principal Spillway (Emergency Spillway):
• Pass the 1 percent (100-year) with 1.5 feet of freeboard.</td><td>First 1/2 inch of runoff from unpaved area and the first inch of runoff from paved area below the auxiliary spillway.</td><td>• Water quality benefits (removal of silt and debris)</td><td>• Limited water quantity control</td></tr>
</table>

Table 8-1 Storage Volume and Outlet Design Criteria Overview (continued)

<table>
<tr><th>Facility</th><th>Available Outlets</th><th>Design Peak Outlet Rate</th><th>Design Storage Elevation</th><th>Water Quantity or Quality Benefits</th><th>Water Quantity or Quality Limitations</th></tr>
<tr><td>Infiltration Basin</td><td>Groundwater</td><td rowspan="2">Dependent on soil conditions (Basin should drain within 24-72 hours)</td><td rowspan="2">10% (10-year), 24-hour storm, basins are to be designed with 1.5 feet of freeboard. 1% (100-year), 24-hour storm, contained within MDOT R.O.W.</td><td>• Useable where limited surface water outlets are available
• No off-site release</td><td>• Should only be used where native soils are highly permeable
• Large amount of land needed
• High liability
• High amount of maintenance required</td></tr>
<tr><td>Infiltration Trench with Perforated Pipe</td><td>Groundwater</td><td>• Useable where limited surface water outlets are available
• No off-site release
• Small amount of space needed</td><td>• Should be used only where native soils have high permeability
• Appropriate only for MDOT R.O.W.
• High amount of maintenance required</td></tr>
</table>

8.3.4 Fencing

Fencing may be required to prevent entry to facilities. Retention basins expected to have 2.0 feet or more of water depth should be fenced with a 6-foot chain link fence, unless ownership is to be transferred to private interests or another public agency that does not wish to use a fence. The fence should be located at least 10 to 15 feet from the high water's edge to enable a vehicle to drive between the pond and the fence. A vehicular gate should be provided for maintenance.

8.3.5 Release Rate and Storage

Release rates for each type of storage facility are outlined in Table 8-1. These rates will generally be limited to peak flows from pre-construction conditions, but may be regulated by a local agency or physical constraints. The design storm for the pre-construction conditions will vary depending on the type of receiving water. Storage requirements for each type of storage facility are presented in Table 8-1. Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project.

8.3.6 Grading and Depth

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Embankments shall have a side slope of 1V:3H or flatter. The crest of the embankment should have a minimum width of 10 feet.

A minimum freeboard of 1.5 feet above the routed 1 percent chance (100-year) design storm high water elevation shall be provided for basins.

Other considerations for design pond depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, and maintenance requirements. Aesthetically pleasing features may be important in urbanized areas.

The bottom of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 0.3 percent bottom slope is recommended.

8.3.7 Structural Controls

Structural controls selected for storage facilities typically include a principal spillway and an auxiliary spillway, and must be able to pass the design peak outflow rate. Outlet works can take the form of combinations of a riser pipe connected to an outlet pipe, weirs, and orifices. A perforated riser pipe is discouraged because of clogging problems, unless an appropriate gravel filter is placed around the riser. (These are used primarily for temporary soil erosion control. See MDOT *Soil Erosion and Sedimentation Control Manual*).

Both the principal and auxiliary spillways are intended to convey the design storm flows. The minimum flood to be used to size these spillways is the 1 percent chance (100-year) flood. The sizing of a particular outlet works shall be based on results of flood-routing calculations.

8.3.8 Location

The location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding and potential impacts of flooding above the facility. 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 watershed or drainage area may affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Thus, it is important for the engineer to design storage facilities that both control runoff from a defined area and evaluate the downstream impact to other drainage structures within the watershed or drainage area. Effective stormwater management must be coordinated on a regional or basin-wide planning basis. MDOT encourages and participates in such planning. The Sargent Creek Watershed report (Sorrell, 1987) discusses the importance of a designer to consider watershed effects.

8.3.9 Regulatory Concerns

Under some circumstances, storage facilities may be classified as a dam. NREPA governs dam safety. A detention or retention basin may be classified as a dam. Part 315 defines a dam as an impoundment at least 6 feet in height and impounds a surface area of 5 acres or more at design flood elevation.

8.4 DESIGN GUIDANCE AND PROCEDURES

Storage Facilities discussed in this section include Detention, Retention, Infiltration, and First Flush Basins. Table 8-1 gives an overview of these facilities' features.

8.4.1 Detention Basins

Dry pond detention basins are depressed areas that store runoff during wet weather and are dry the rest of the time. They have comparatively low cost, have few design limitations, can serve large as well as small watersheds, and can be incorporated into other uses, such as recreational areas. Table 8-2 summarizes considerations for a detention basin.

Detention basins can be utilized in areas where:

- Suitable depressions occur or can be constructed, and where acceptable inflow and outflow conditions can be achieved. Also from a cost standpoint, the basin should be placed where the least amount of earth moving is required to obtain the necessary basin volume.
- Soils are able to provide a stable embankment.
- Storage volume and the surface area for pollution abatement are adequate to detain the runoff from a specified storm event.

8.4.1.1 Detention Basin Design

Design considerations for a detention basin from FHWA's "*Retention, Detention, and Overland Flow for Pollutant Removal from Highway Storm Water Runoff: Interim Guidelines for Management Measures*," dated March 1988, and the Michigan Department of Natural Resources' *Stormwater Management Guidebook*, dated March 1992, are noted below.

- Storage volume and surface area available for detention must be determined. Additional R.O.W. may be required based on the surface area inundated during the 1 percent chance (100-year) flood.
- Stability of the detention basin embankments and natural slopes requires the following:
  a. Soil boring(s) for embankment basin area and outlet structure.
  b. Determination of groundwater elevation.
  c. Embankment slopes at 1:3 or flatter.
  d. Grassed slopes (maintained).
  e. Auxiliary spillway should be graded through natural ground, <u>not</u> over constructed embankment.

Principal spillway can be either a mechanical spillway (CSP riser and outlet tube) or permanent steel sheet piling weir (preferred based on constructability, maintenance, and safety considerations; weir outlet may require the use of a low-flow orifice to completely drain the basin). An example weir structure is shown in Figure 8-1, Example Weir Structure. However, orifices may also be installed in the stop planks to release low flows. Detail plans depicted in Figure 8-1, can be obtained from the Design Engineer - Hydraulics/Hydrology.

- Methods other than weirs are generally discouraged by MDOT.
- Highway safety and aesthetics:
    a. AASHTO recommends protective treatments (see Section 8.3.4) when a body of water more than 2 feet deep is located in the clear zone.
    b. Gentle slopes for maintenance access, measures to prevent public access to outlet structures. Local zoning regulations may exist for stormwater detention/retention facilities and fencing.

In urban environments, land available for detention is limited. MDOT will consider providing detention in buried, oversized storm sewers. The required storage volume is calculated the same as for basins on the surface. Gravity discharge of the pipes is preferred; however, because of their depth, a pumped outlet may be required (See Chapter 10, Pump Stations).

<u>Water Quality Considerations</u>

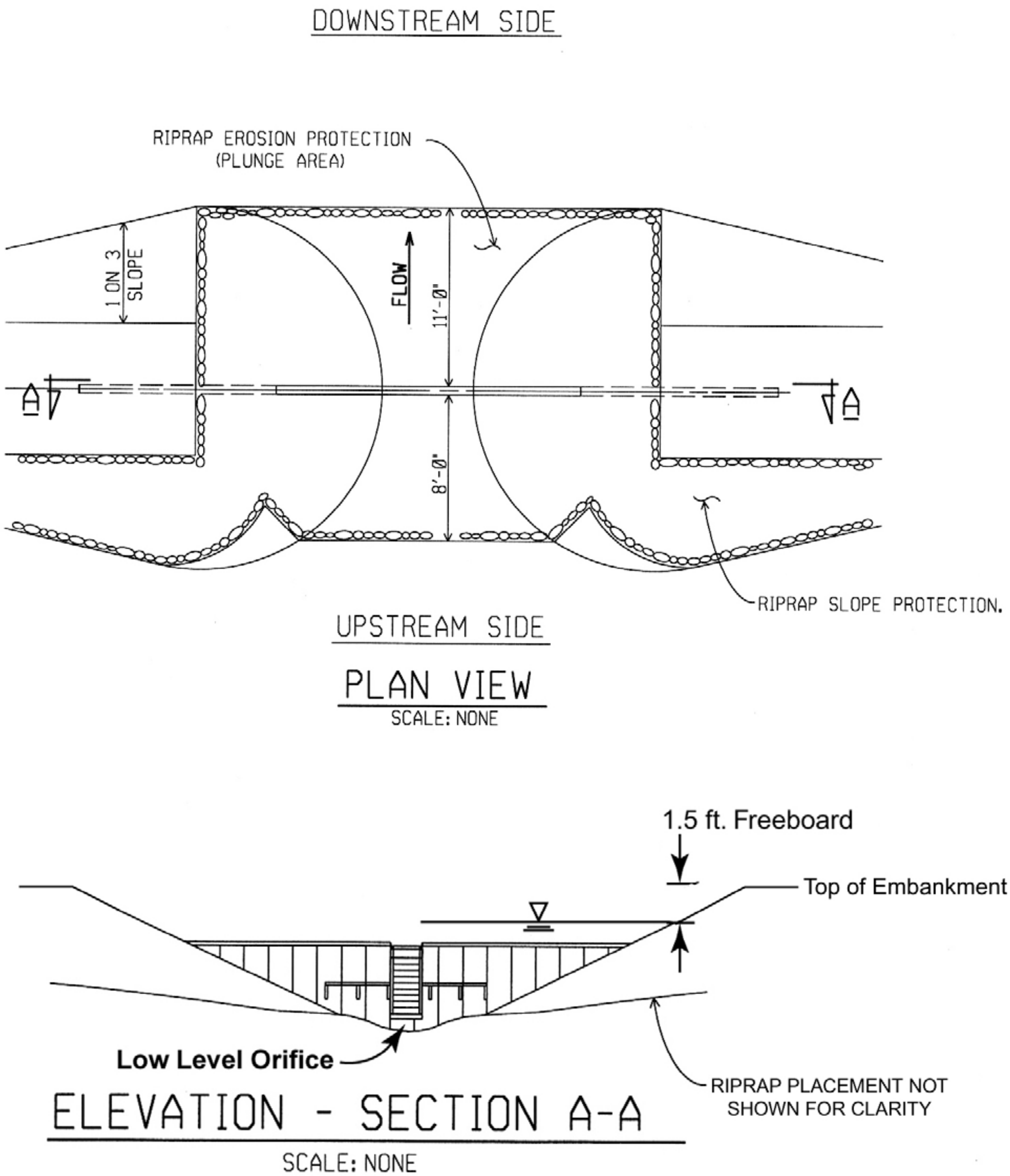
One consideration to help with water quality is shaping the basin to improve its pollutant-removal capabilities. The length-to-width ratio should be at least 3:1, and a wedge-shaped basin (wider at the outlet) can also improve pollutant removal. Figure 8-3, Methods of Increasing the Length-to-Width Ratio, shows pond configurations that may be used to increase the length to width ratio and allow for maximum flow path length. 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 0.3 percent to prevent the ponding of stormwater, and the side slopes should allow for easy maintenance access (see Figure 8-2, Detention and First Flush Basin). An additional consideration is the release from the basin. It is recommended that the volume be released over 24 hours or more (probably through a low-flow orifice below the weir).

Safety considerations include reducing the chance of drowning by fencing the basin, reducing the maximum depth and/or including ledges and mild slopes to prevent people from falling in and facilitate their escape from the basin.

<u>Outlets</u>: See Section 8.4.5 for more information on outlets. Weirs are the preferred method for discharge from storage facilities. Auxiliary spillways should be capable of passing the peak from a 1 percent chance (100-year) storm.

Table 8-2 Summary of Considerations for a Detention Basin

Quality	Release over 24 hours or more. Pollutant removal is moderate.
Quantity	Store volume from a 10 to 4 percent (10- to 25-year) routed post-development flow. Release at a rate equal to or less than the 10 percent (10-year) existing peak for enclosed systems, and the 4 percent (25-year) existing peak for open channel systems. The water surface elevation during the 1 percent chance (100-year) post-development storm should be 1.5 feet below the basin embankment and within MDOT R.O.W.
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). Inspections should also be called for after major flood events. Remove sediment every 5 to 10 years.
Safety	Fence around pond; provide 10-foot vegetative and maintenance ledge around pool; post signs
Other	Side slopes provide easy maintenance access (1V:3H); 0.3 percent bottom slope to prevent ponding. Local community may dictate other basin design criteria.



Note: A low level orifice can be used in the bottom of stop planks to empty basin.

Figure 8-1 Example Weir Structure

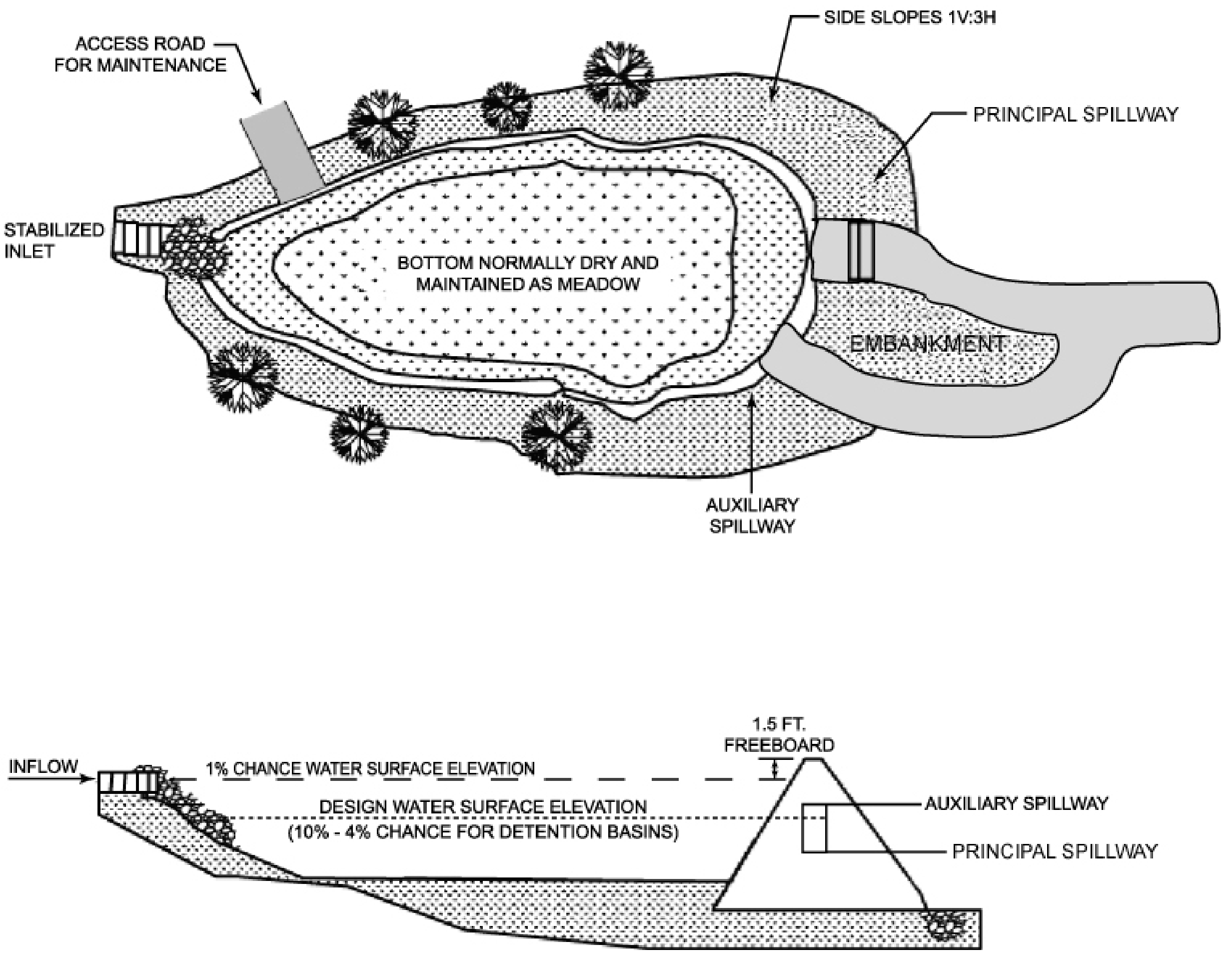


Figure 8-2 Detention and First Flush Basin

Note: 1 percent chance (100-year) water surface elevation may inundate areas outside of the basin within the MDOT R.O.W.

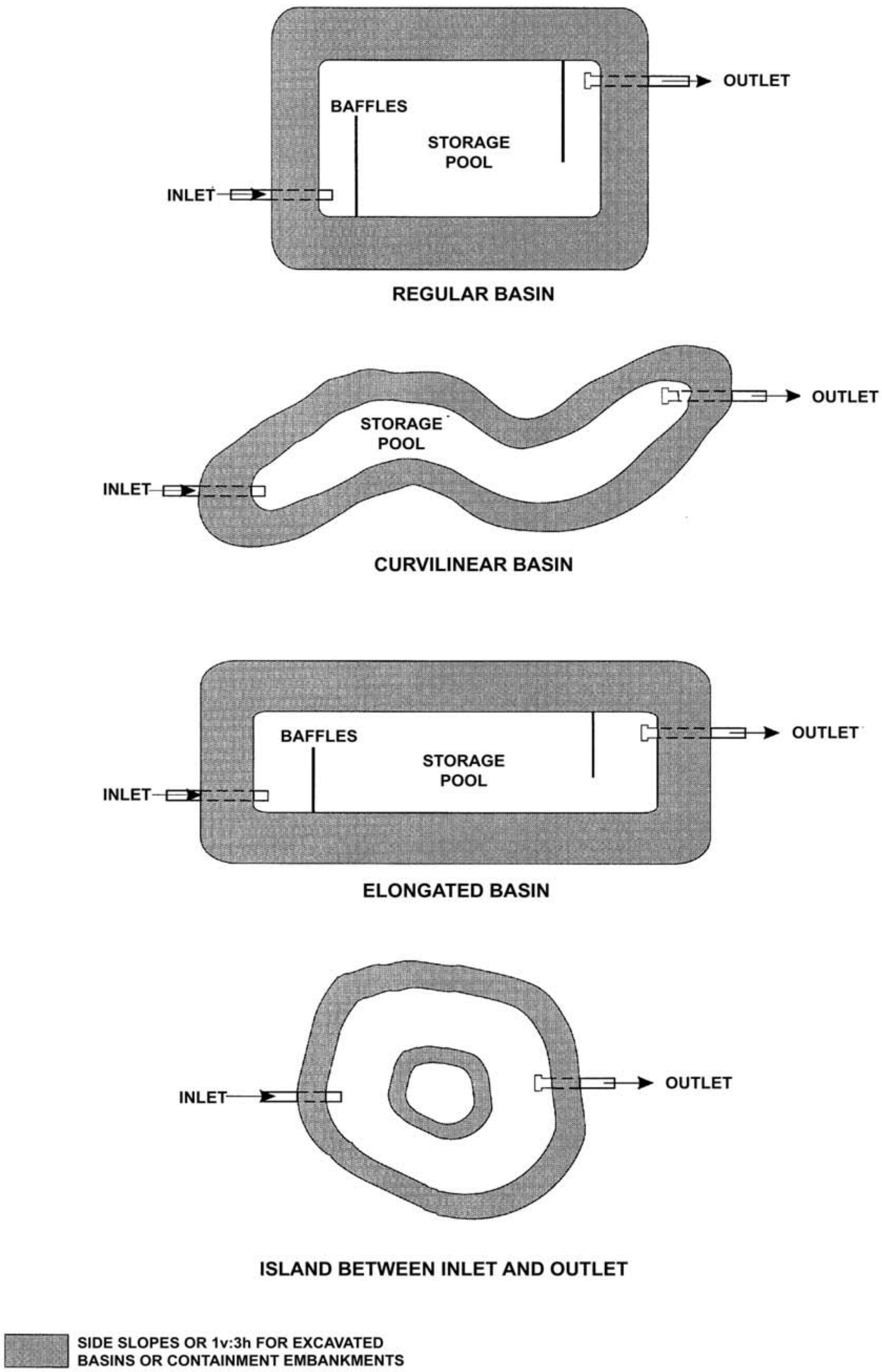


Figure 8-3 Methods of Increasing the Length-to-Width Ratio

8.4.2 Retention Basins for Regional Use/Watersheds

Retention basins are very similar to detention basins, but differ in that they retain a permanent pool during dry weather. Retention basins are usually more expensive to construct and operate than detention basins. Because of their permanent pool, they may also have recreational benefits or provide wetland mitigation. Table 8-3 summarizes considerations for a retention basin.

The use of retention storage facilities are discouraged within MDOT R.O.W. because of the extensive maintenance and the large amount of R.O.W. required. Provisions for weed control and aeration for prevention of anaerobic conditions should be considered. Also, facilities that have the potential for becoming nuisances or health hazards should not be built. MDOT will consider these structures when requested by local agencies to comply with watershed goals.

8.4.2.1 Retention Basin Design

<u>Quality</u>: The permanent pool greatly enhances the pollutant removal capacity of the basin. The length-to-width ratio of the pool should be at least 3 to 1 to maximize pollution removal. Figure 8-3, Methods of Increasing the Length-to-Width Ratio, gives some configurations to help achieve an acceptable length-to-width ratio. See Chapter 9, Stormwater Best Management Practices (BMPs), for additional discussion on quality.

<u>Quantity</u>: For quantity purposes, the retention basin is designed similarly to the detention basin. The facility should be designed to store volume from the 10 percent to 4 percent chance (10-year to 25-year) routed post-development flow. The basin should discharge at a rate equal to or less than the existing peak from a 10 percent chance (10-year) for enclosed systems, and a 4 percent chance (25-year) for open channel systems, see Table 8-1. An auxiliary spillway should be provided in combination with a principal spillway to pass the 1 percent chance (100-year) storm (see Figure 8-4, Retention Basin). Side slopes should not exceed 1V:3H. An access driveway should be provided for maintenance along with a valve to drain the pond during maintenance.

Shallow impoundments are unattractive and the need for visual screening and/or landscaping may be needed. Three feet of permanent water depth may alleviate problems with odor, weeds, and vectors. See Figure 8-4, Retention Basin, for a schematic.

Fencing should be provided for ponds with permanent depths of 2 feet or more. A 10-foot vegetative and maintenance ledge should be provided around the edge of the pond and signs should be posted.

<u>Outlets</u>: See Section 8.5 on outlet hydraulics. Weirs are the preferred method to discharge from storage facilities.

Table 8-3 Summary of Considerations for a Retention Basin

Quality	Pollutant removal is moderate to high.
Quantity	Design to store runoff from the 10 to 4 percent chance (10- to 25-year) storm. Release storage volume at a rate equal to the 10 percent chance (10-year) existing peak for enclosed systems, and the 4 percent chance (25-year) existing peak for open channel systems. Design to pass the 1 percent chance (100-year) storm through an auxiliary spillway with 1.5 feet of freeboard and within MDOT R.O.W.
Shape	3:1 length-to-width ratio; wedge shaped (wider at outlet); permanent pool depth 2 feet or more.
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 10-foot vegetative and maintenance ledge around pond; post signs.
Other	Side slopes provide easy maintenance access (1V:3H); perimeter vegetation; sediment forebay; provide valve to drain pond for maintenance.

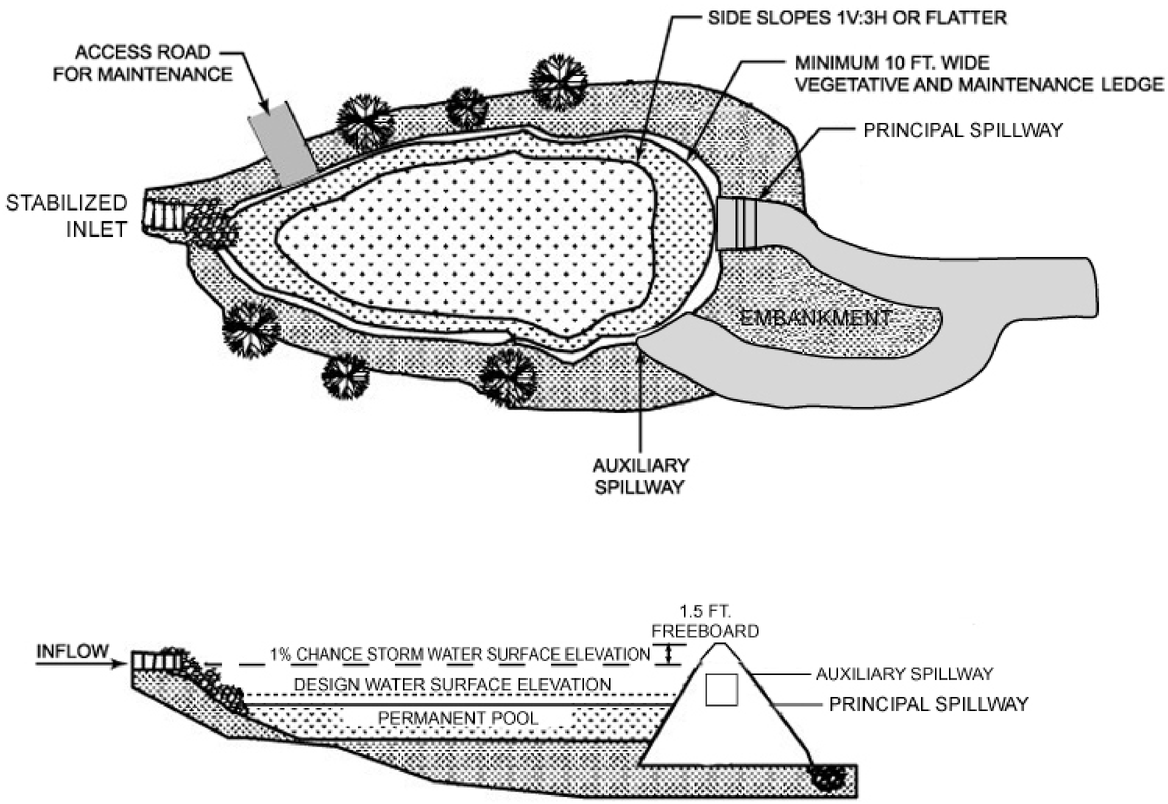


Figure 8-4 Retention Basin

8.4.3 First Flush Basins

The storage capacity in first flush basins is sized to regulate water quality by dropping out pollutants attached to sediments. First flush refers to the large percentage of storm pollutant loading that is produced by a relatively small percentage of the runoff volume during the initial stages of the runoff. The storage volume required will be based on controlling the first 0.5 inch (unpaved area) to 1 inch (paved area) of runoff volume from the drainage area and keeping the water surface elevation below the auxiliary spillway. Equation 8.1 can be used to calculate estimated runoff volume:

$$\mathbf{V_s = (1/12) \times A_p + (0.5/12) \times A_u} \qquad \mathbf{(8.1)}$$

Where: V_s = volume of runoff, acre-feet
A_p = area of impervious pavement, acres
A_u = unpaved area in acres contributing to the highway drainage, acres

The outlet shall consist of a standard weir structure with a low flow orifice, see Figure 8-1, Example Weir Structure, and Section 8.4.1. The orifice shall be sized to release the volume in a 24-hour, or longer, time period. The weir crest elevation should be set to the depth of the first flush volume V_s. The weir dimensions shall be set to pass the design peak flow as noted in Table 8-1. The 1 percent chance (100-year) storm event shall pass through the first flush basin without causing a harmful interference to upstream and downstream properties The flood elevation of the receiving waters, whether an open channel or enclosed system, must be evaluated for impacts on the outlet structure and basin.

Conceptual designs for discharge from a first flush basin are similar to a detention basin and are provided from the Design Engineer - Hydraulics/Hydrology.

Table 8-4 Summary of Considerations for a First Flush Basin

Quality	Controls runoff from minor storms such as the 50 percent chance (2-year) or smaller. Pollutant removal is moderate to high.
Quantity	Storage volume required based on controlling first 0.5 inch (unpaved area) to 1 inch (paved area) of runoff below the auxiliary spillway. Release rates should drain volume over 24 hours or more.
Shape	3:1 length-to-width ratio.
Maintenance	Inspect once a year, preferably during wet weather to check operation and during dry weather to check need for sediment removal. Mow at least twice a year; remove sediment every 1-2 years.
Safety	Fence should be placed around pond if pool is deeper than 2 feet
Other	Side slopes should not be steeper than 1V:3H.

8.4.4 Infiltration Facilities

Infiltration facilities are storage facilities where the primary discharge of stormwater is to the groundwater. These include:

- Infiltration basins, and
- Infiltration trenches.

Table 8-4 provides a summary of considerations for infiltration facilities. Infiltration facilities can easily be adapted to fit the requirements of highway systems, i.e., shaped into deep-sided linear basins, constructed in borrow areas, or linked in a series of small basins. Infiltration systems are primarily applied in cases where R.O.W. is a limiting factor and where the soil is relatively permeable and the groundwater table is 5 feet or more below the bottom of the facility.

The biggest disadvantage of infiltration facilities is that over time solids may plug soil voids reducing the infiltration capacity. For this reason, a first flush basin is recommended upstream of the infiltration facility to remove as much sediment as possible.

Theoretically, since there is no outflow to surface water unless the storage capacity is exceeded, pollutant removal efficiencies approach 100 percent. True pollutant removal efficiencies, however, depend on specified requirements. Designers must consider the following:

- Soils/subsoils must be moderately to highly permeable. A permeability test, such as the falling head test, is necessary. Permeability should be greater than 1 inch/hour. County soil maps may be used for initial location.
- Groundwater table is approximately 5 feet or more below the bottom of the storage area. The location of the groundwater table must be determined by the soil borings.
- Volume must equal or exceed storage for the runoff volume.

8.4.4.1 Site Selection

Site selection is based on infiltration rate. The infiltration rate is a very important parameter when an infiltration facility is designed. After a suitable site for the facility has been found, several soil tests must be made before the facility is designed.

First, borings should be taken at the site to determine the soil types, depth to bedrock, presence and elevation of groundwater, and infiltration rates. County soil maps can be used to estimate some parameters. The infiltration rate can be determined with varying elevation heads. This can be done in the field with the falling head test.

8.4.4.2 Infiltration Basin

Stormwater from smaller, more frequent storms can be infiltrated through the bottom of the first cell of an infiltration basin, see Figure 8-5, Infiltration Basin. Flow rates in excess of the infiltration capacity are controlled by storing water in the basin until sufficient infiltration capacity becomes available. An important consideration for an infiltration basin is keeping the bottom from clogging with sediment. The clogging of basins from lack of maintenance, along with the over-estimating of their infiltration rates, may lead to the failure of many infiltration basins.

<u>Quality</u>: To protect groundwater quality, the bottom of the infiltration basin must be 5 feet or more above the seasonably high groundwater table. The primary removal mechanisms in infiltration basins are sedimentation, filtration, and biological uptake. Provide a filter strip or first flush forebay at the entrance. The filter strip should be at least 20 feet wide and be sloped from 2 to 5 percent to prevent water from ponding and to ensure a slow velocity. Vegetation can also contribute to the removal of pollutants through biological uptake. As the stormwater leaves, it is filtered again by the underlying soil.

An estimation of the maximum ponding depth for a desired drain time can be found with the equation:

$$\mathbf{d = f\ T_s} \qquad \textbf{(8.2)}$$

Where: d = depth, inch
f = steady infiltration rate, inch/hour
T_s = time of storage, hour

The recommended allowable storage time is 24-72 hours. Considering basins may fail because of clogging and an infiltration rate that is lower than expected, a shorter time in the range, such as 40 hours, might be used to compensate for inaccuracies in estimating infiltration rates. Several other considerations help enhance the pollutant removal of these facilities.

The pond bottom should be sloped as close to zero as possible in order to obtain a uniform depth of stormwater over the basin. The side slopes should be sloped at 1V:3H, or flatter, to allow for easy maintenance access and prevent erosion.

Incoming stormwater should also be considered. A combination of a level spreader/first flush basin can be constructed to spread the stormwater evenly, thereby reducing erosion, and trap sediments before they clog the basin. Riprap should be placed at the inlet to help reduce erosion. A riprap berm with or without a connecting outlet pipe separates the first flush forebay from the rest of the infiltration basin.

Quantity: For an infiltration basin, quantity can be controlled very similarly to a detention basin. The basin should be designed to contain the runoff volume from the 10 percent chance (10-year), 24-hour storm with 1.5 feet of freeboard. The designer should verify that the water surface elevation produced by a 1 percent chance (100-year), 24-hour storm is contained within the MDOT R.O.W.

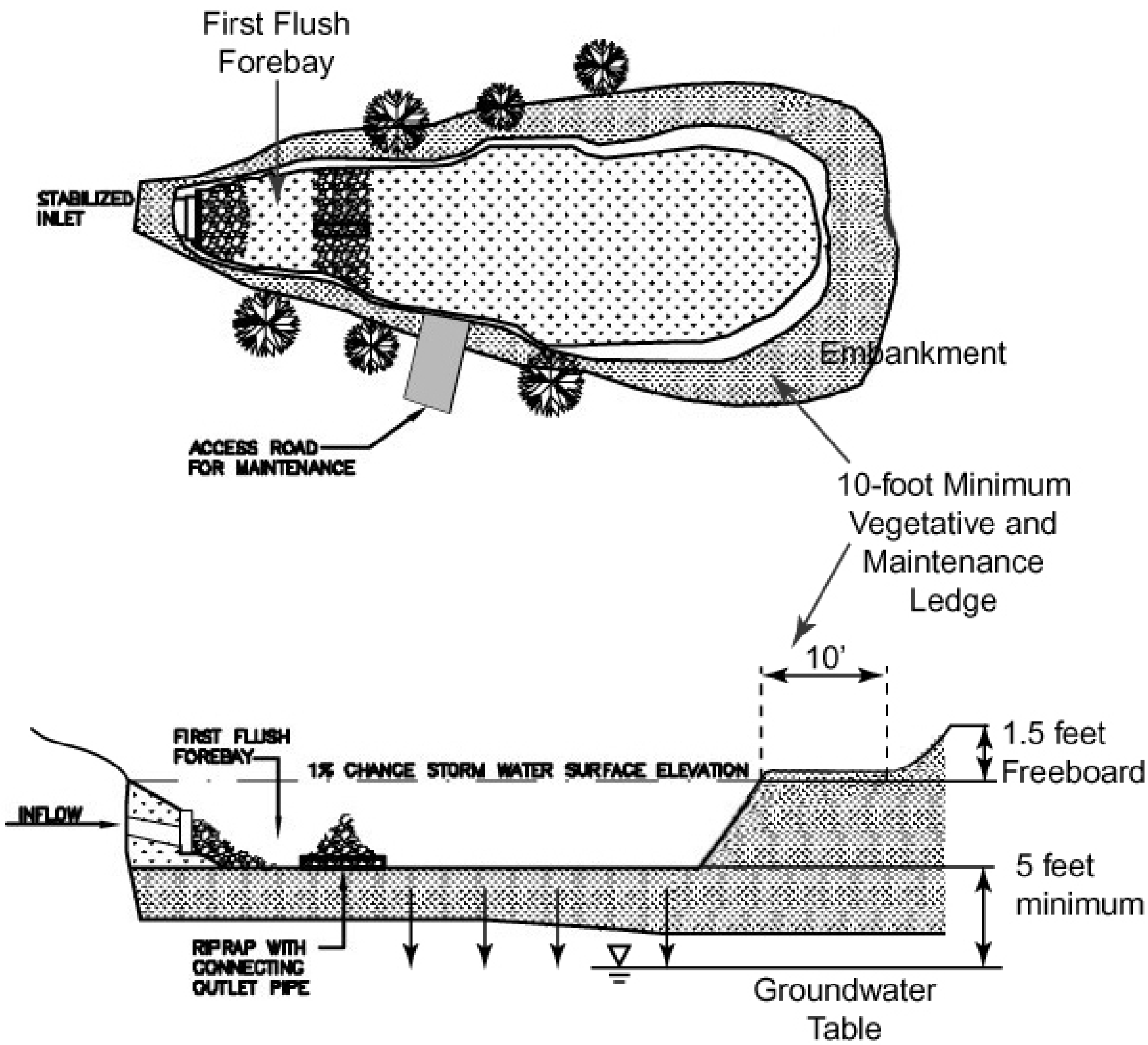


Figure 8-5 Infiltration Basin

8.4.4.3 Infiltration Trench with Perforated Pipe

An infiltration trench is a facility where a trench is excavated and then filled with a porous medium and a perforated pipe may be added. Stormwater is stored in the voids of the fill material until it can be infiltrated. 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. A typical infiltration trench is shown in Figure 8-6, Infiltration Trench with Observation Well, and a section of an observation well is shown in Figure 8-7, Infiltration Trench Observation Well.

Quality: The bottom of the facility must be 5 feet above the seasonably high groundwater table, and the bottom of the stone reservoir must be below the frost line. The runoff volume must be infiltrated within 24 to 72 hours. The primary removal mechanisms in trenches are sedimentation and filtration, along with some biological uptake. Filtering is achieved in the top layers of the facility as stormwater enters. In the stone reservoir, the main removal mechanisms are sedimentation and adsorption.

As the stormwater leaves, it is filtered again by the underlying soil where more pollutants will be removed. Unfortunately, all infiltration facilities are vulnerable to clogging, thereby reducing their effectiveness. Therefore, a vegetated buffer strip filtering the runoff is recommended upstream of an infiltration facility. The strip would decrease the amount of suspended solids in the stormwater, thus increasing the useful life of the infiltration facility. The filter strip should be at least 20 feet wide and be sloped from 2 to 5 percent to prevent water from ponding and to ensure a slow velocity.

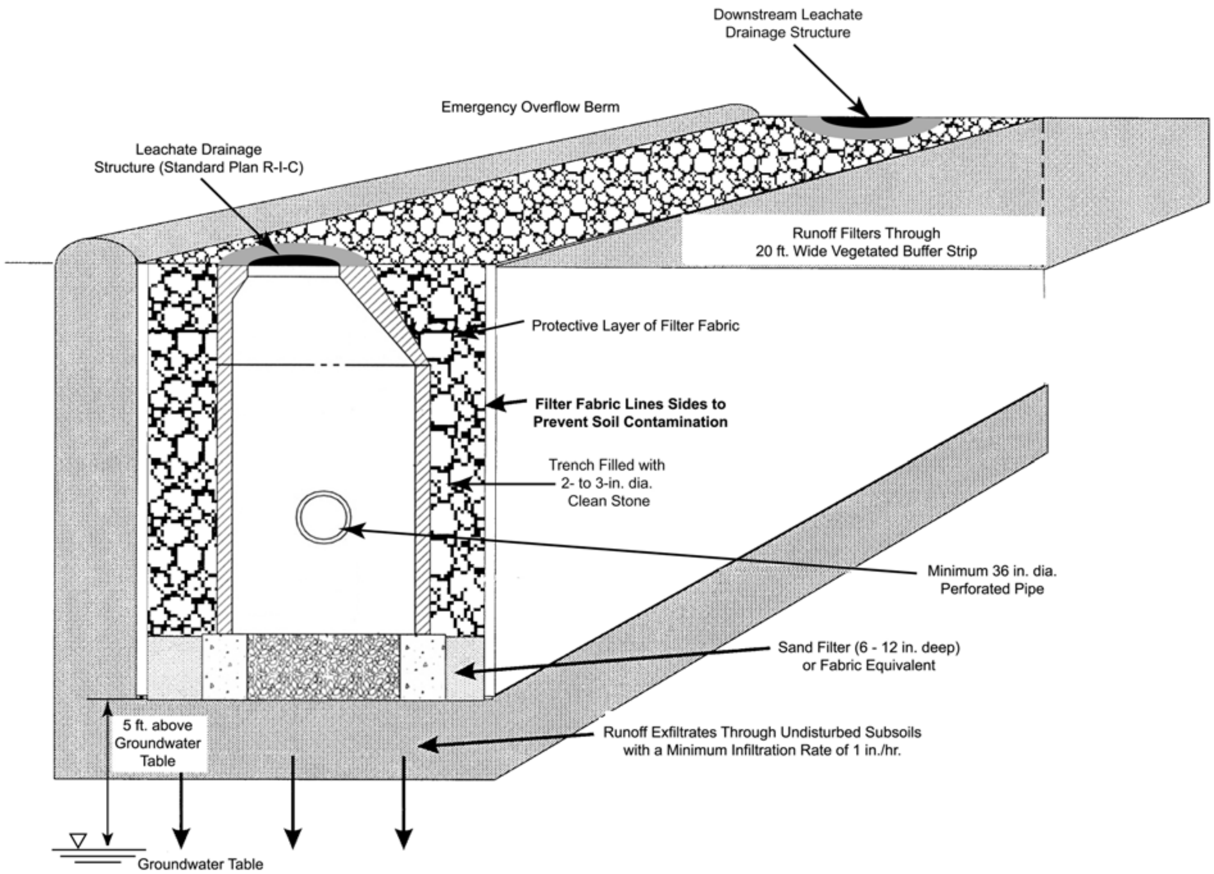


Figure 8-6 Infiltration Trench with Leachate Drainage Structure and Perforated Pipe

Quantity: Because of the large size of the perforated pipe required to control a 10 percent chance (10-year) storm, it is suggested that trenches only be used for drainage areas from pavement R.O.W. (drainage areas greater than the R.O.W. may require an infiltration basin).

After a storage volume has been determined, the dimensions of the facility can be estimated. The depth should be designed such that the bottom is 5 feet above the high water table and below the frost line. The surface area can be manipulated to suit the site conditions so long as it yields the required storage volume. The amount of surface area required is:

$$S_a = Vol_s/V_r d \tag{8.3}$$

Where: S_a = surface area, sf
Vol_s = storage volume, cf
V_r = void ratio (0.4 for 1.5 to 3 inches)
d = depth, feet

Other Considerations: Detention time is an important factor in determining the effectiveness of a trench facility. A facility which drains quickly is capable of treating more stormwater volume. A maximum detention time of 24 to 72 hours is recommended. The actual detention time can be estimated by:

$$\mathbf{T_s = dV_r/f} \qquad \textbf{(8.4)}$$

Where: T_s = storage time or detention time, hour
d = depth of storage in the trench, inch
V_r = void ratio of stone reservoir
f = steady infiltration rate, inch/hour

From this equation, it is apparent that detention time is directly related to trench depth. Since the runoff volume will most likely be much smaller than the storage required for a 10 percent chance (10-year) storm, the depth of the runoff volume would be very small in the trench. Therefore, infiltration trenches are much better suited for small drainage areas where the change in peak flow between pre- and post-construction is small. Modifications can be made to the trench design to increase the depth of the runoff volume storage, but these will increase the cost and could make this BMP option infeasible.

Table 8-5 Summary of Considerations for an Infiltration Facility (Basin/Trench)

Quality	Infiltrate within 24 to 72 hours. Pollutant removal is moderate to high.
Quantity	Control 10 percent chance (10-year), 24-hour storm volume with 1.5 feet of freeboard. The 1 percent chance (100-year), 24-hour storm should be contained within the MDOT R.O.W. Release rates are determined based on soil conditions.
Shape of Basin	Dependent on site constraints.
Maintenance	Inspect during wet weather; mow area twice a year; remove sediment every 5-10 years. If basin/trench takes longer than 72 hours to infiltrate, clean bottom.
Other Considerations	First flush basin or grass filter stone upstream to remove sediments; infiltration rate minimum 1 inch/hour; depth to groundwater 5 feet or more.

8.4.5 Erosion/Sediment Traps

During construction, sediment basins are temporary man-made depressions in the ground where runoff water is collected and stored to allow suspended solids to settle out during construction. They are temporarily used in conjunction with erosion control measures to prevent off-site sedimentation. They may consist of a dam, barrier, or excavation, a principal and auxiliary outlet structure, and water storage space. Their primary purpose is to trap sediment and other coarse material.

Sediment basins may be part of a permanent structural BMP such as detention, retention, or infiltration basin stormwater storage facility after completion of the construction project. It is therefore important to determine from the onset what the ultimate function of the basin will be and design accordingly. See Chapter 9, Stormwater Best Management Practices (BMPs), for design of temporary erosion/sediment trap basins.

8.4.6 Outlets

Principal or auxiliary spillways are often constructed as weirs as described in this section. Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, sloped weirs, and riser pipes with outlet tubes.

8.4.6.1 Sharp-Crested Weirs

A sharp-crested weir with **no end** contractions is illustrated in Figure 8-7, Sharp-Crested Weir (No End Contractions) and Figure 8-9. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [3.27 + 0.40\,(H/H_c)]\,LH^{1.5}} \qquad \mathbf{(8.5)}$$

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

A sharp-crested weir with **two end** contractions is illustrated in Figure 8-8, Sharp-Crested Weir (Two End Contractions). The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [3.27 + 0.40\,(H/H_c)]\,(L - 0.2H)\,H^{1.5}} \qquad \mathbf{(8.6)}$$

Where: Variables are the same as equation 8.5.

As H/H_c becomes less than 0.3,, the weir equation reduces to:

$$Q = 3.3\,L\,H^{1.5}$$

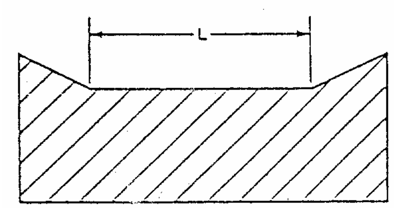


Figure 8-7 Sharp-Crested Weir (No End Contractions)

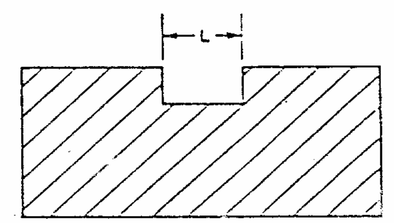


Figure 8-8 Sharp-Crested Weir (Two End Contractions)

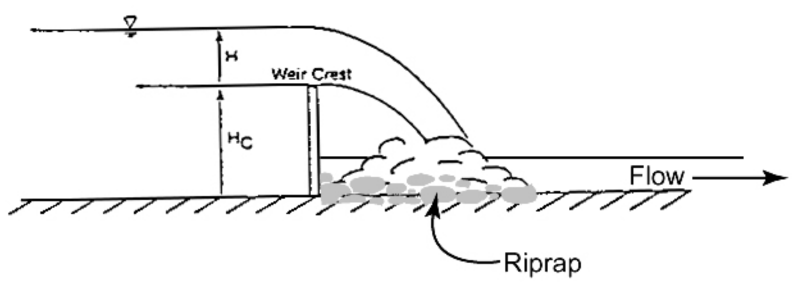


Figure 8-9 Sharp-Crested Weir and Head (No End Contractions)

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

$$\mathbf{Q_S = Q_f(1 - (H_2/H_1)^{1.5})^{0.385}} \quad \textbf{(8.7)}$$

Where: Q_s = submergence flow, cfs
Q_f = unsubmerged weir flow from equation 8.5 or 8.6, cfs
H_1 = upstream head above crest, feet
H_2 = downstream head above crest, feet

8.4.6.2 Broad-Crested Weirs

A broad-crested weir has a small crest width relative to the head that occurs during embankment overtopping. The equation generally used for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{3/2} \tag{8.8}$$

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

If the upstream edge of a broad-crested weir is rounded 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 3.08. For sharp corners on the broad-crested weir, a minimum C value of 2.58 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 8-6.

Table 8-6 Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head (feet)

Measured Head	Breadth of Crest of Weir (feet)										
H* (feet)	0.5	0.75	1.0	1.5	2.0	2.5	3.0	4.0	5.0	10.0	15.0
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	2.6	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.6	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.6	2.6	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	2.7	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	3.2	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	2.7	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

* Note: Measured at least 2.5H upstream of the weir.

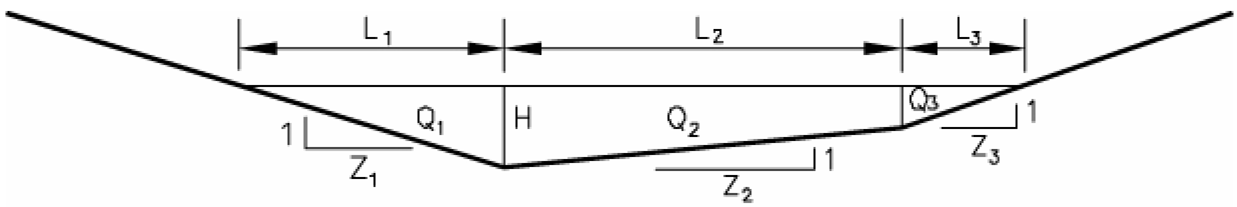
Source: Brater and King, 1976

8.4.6.3 Sloped Weirs

Sloped weirs a special case of broad-crested weirs. When sloping roadways act as the weir outlet, they are considered sloped weirs.

For weirs which are not flat, it is difficult to identify the height over the weir, h.

There are two methods which provide reasonable results. Each involves computing a discharge through subsections of the weir (see Figure 8-10, Weir Flow Over Sloping Weir).



STANDARD WEIR EQUATION:

$$Q_1 = C_1 L_1 \; [(0+H_1)/2]^{3/2}$$
$$Q_2 = C_2 L_2 \; [(H_1 + H_2)/2]^{3/2}$$
$$Q_3 = C_3 L_3 \; [(H_2 + 0)/2]^{3/2}$$

SLOPING WEIR EQUATION:

$$Q_1 = \frac{2}{5} Z_1 (C_1 H_1^{5/2} - 0)$$
$$Q_2 = \frac{2}{5} Z_2 (C_2 H_2^{5/2} \; C_1 H_1^{5/2})$$
$$Q_3 = \frac{2}{5} Z_3 (C_2 H_2^{5/2} \; 0)$$

Figure 8-10 Weir Flow Over Sloping Weir

<u>Standard Weir Equation - Average Height</u>

This method involves computing an average height for each subsection and using the standard weir equation:

$$Q = CLH^{3/2} \qquad (8.8)$$

$Q_1 = C_1 L_1 [(O+H_1)/2]^{3/2}$
$Q_2 = C_2 L_2 [(H_1 + H_2)/2]^{3/2}$
$Q_3 = C_3 L_3 [(H_2 + O)/2]^{3/2}$

Q total - $Q_1 + Q_2 + Q_3$ (for three sub-sections)

Sloping Weir Equation

The sloping weir equation is defined as:

$$\mathbf{Q = (2/5\ Z\ (CH_2^{5/2} - CH_1^{5/2})} \quad \textbf{(8.9)}$$

Where Z is the slope of the weir subsection (1 foot vertical: Z feet horizontal)

$Q_1 = 2/5\ Z_1\ (C_1\ H_1^{5/2})$

$Q_2 = 2/5\ Z_2\ (C_2\ H_2^{5/2} - C_1\ H_1^{5/2})$

$Q_3 = 2/5\ Z_3\ (C_2 H_2^{5/2} - 0)$

Q total = Q1 + Q2 + Q3 (for three sub-sections)

Discharge Coefficients

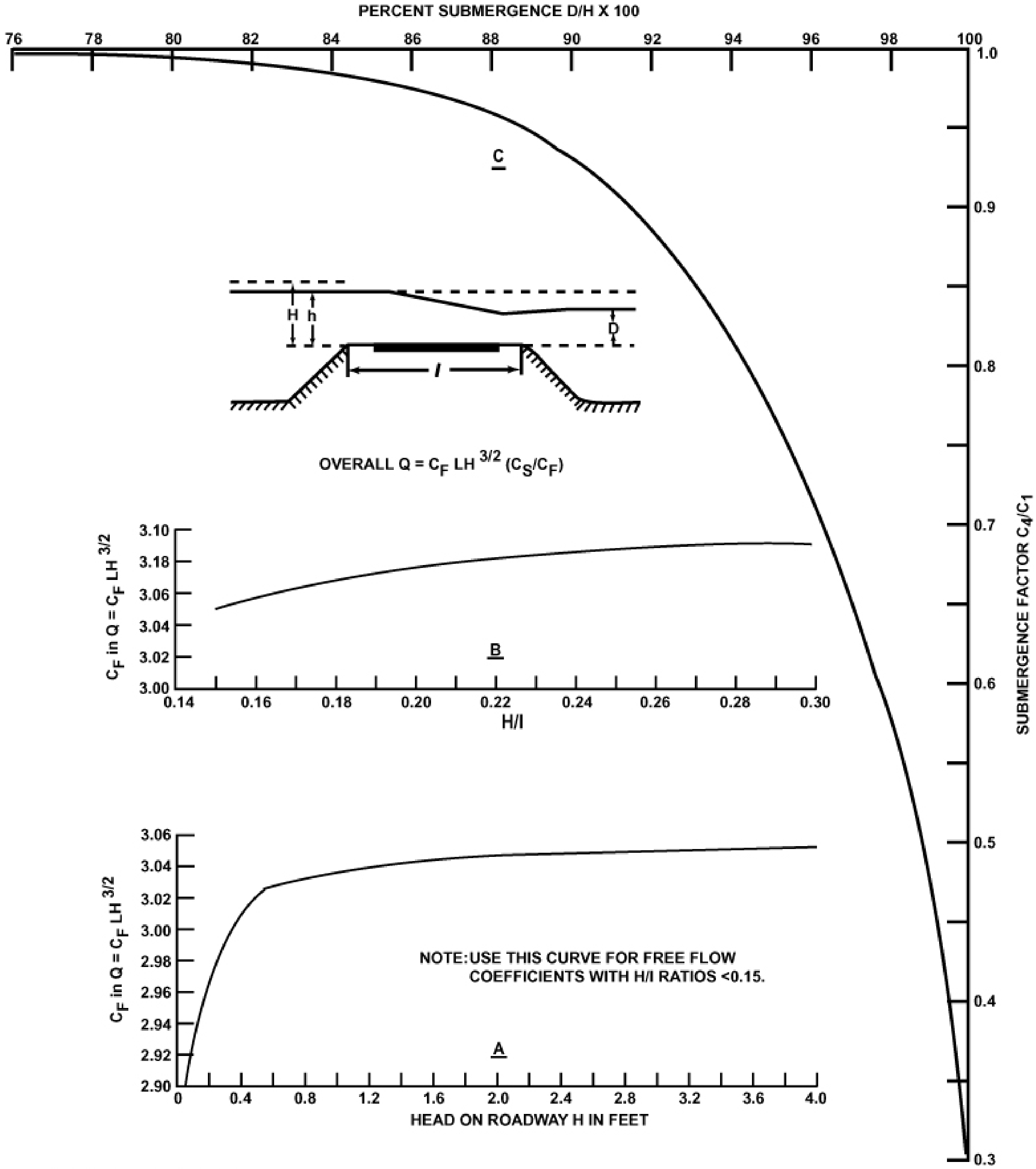
There are charts available which indicate what C value should be used, considering the head (H) on the embankment. These charts are for unobstructed (no railings, etc.) and unsubmerged flow. Generally, using a C value of about 2.9 to 3.0 for paved roads and 2.8 to 2.9 for gravel roads will provide reasonable results. It would be possible to compute a new C for each H; however, the effect will not be significant. If there are numerous obstructions on the weir (curb and gutter, guardrail, etc.), judgment must be used to reduce the C value.

Weir Length

Caution must be used in determining the effective weir length of a roadway. It is important to eliminate any portion of the roadway in which the areas upstream and downstream of a roadway are higher than the road, which makes that portion of the weir ineffective.

Submergence

It is very important to note that tailwater submergence of a weir will significantly reduce the weir efficiency. Figure 8-11, Discharge Coefficients for Flow over Roadway Embankments, shows a graph used to adjust the discharge due to tailwater submergence.



SOURCE: FHWA, HDS1, 1978

Figure 8-11 Discharge Coefficients for Flow over Roadway Embankments

8.4.6.4 V-Notch Weirs

The discharge through a 90-degree v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5H^{2.5} \quad \textbf{(8.10)}$$

Where: Q = discharge, cfs
H = head on apex of notch, feet

8.4.6.5 Orifices (or Culverts acting as Orifices)

Pipes smaller than 1 foot may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions:

$$Q = 0.62A\,(2gH)^{0.5} \quad \textbf{(8.11)}$$

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

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

8.4.6.6 Riser Pipes

Many local agencies use perforated riser pipes as a method of controlling release rates from storage facilities. These are depicted in Figure 8-12, Perforated Pipe with Riser Tube Outlet.

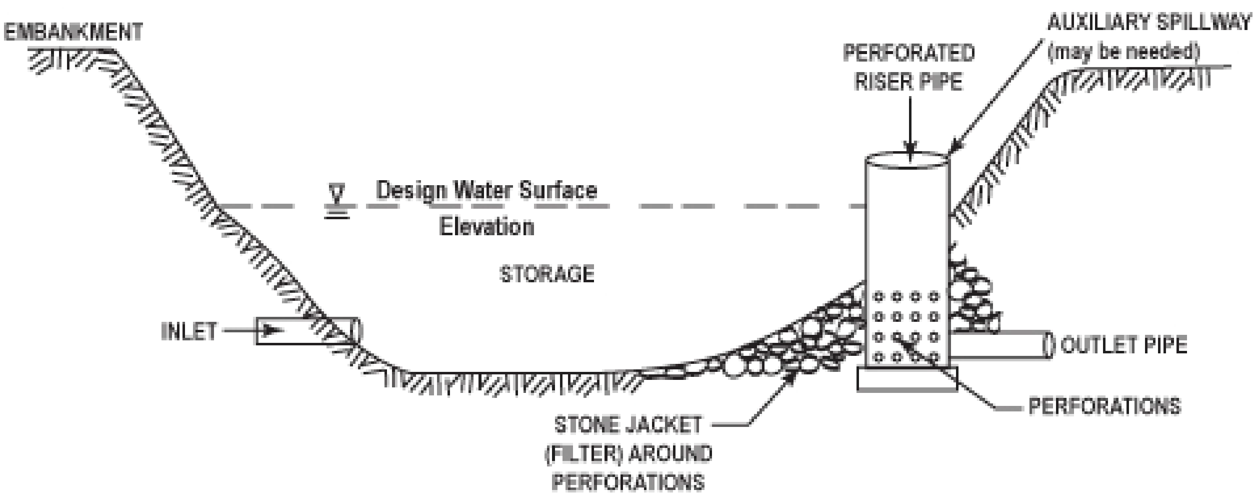


Figure 8-12 Perforated Pipe with Riser Tube Outlet

The riser structure is a pipe placed on end. The riser diameter is a minimum of 24 inches. Perforations are placed in the riser to allow restricted flow rates to enter. The outlet pipe carries the water from the riser through the embankment to the receiving water. When the water rises high enough, the top end of the riser will allow water to enter. The hydraulics of water entering the top of the riser pipe is generally calculated with the weir equation.

The perforations in the riser are generally small and prone to plugging. Typically, a stone jacket is placed around the riser to filter objects that could plug the perforations.

The orifice equation is used to calculate the number, elevation, and size of the perforations. A tailwater elevation for the perforations also needs to be calculated. This can be accomplished by using the culvert procedures (Chapter 5, Culverts) to estimate the energy losses in the outlet pipe. Example calculations are shown in Section 8.4.7.4.

8.4.7 Hydrograph Routing Procedure Overview

8.4.7.1 Data Needs

The following storage and discharge data will be needed to complete storage design and routing calculations. Using this data, a design procedure is followed to route the inflow hydrograph through the storage facility. Basin and outlet geometry can be changed until the desired outflow hydrograph is achieved.

- Inflow hydrograph for all selected design storms.
- Stage-storage curve for proposed storage facility (see Figure 8-13, Example Stage-Storage Curve).
- Stage-discharge curve for all outlet control structures (see Figure 8-14, Example Stage-Discharge Curve).

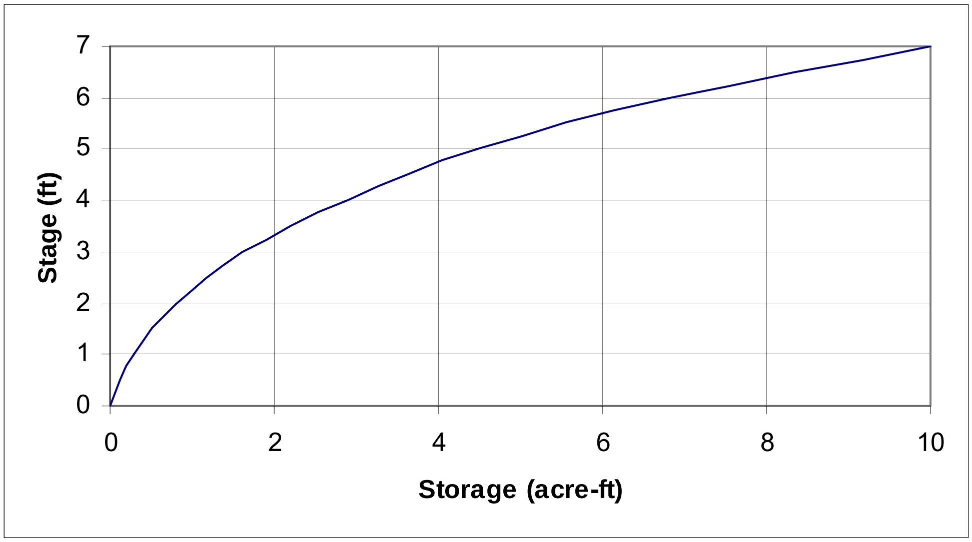


Figure 8-13 Example Stage-Storage Curve

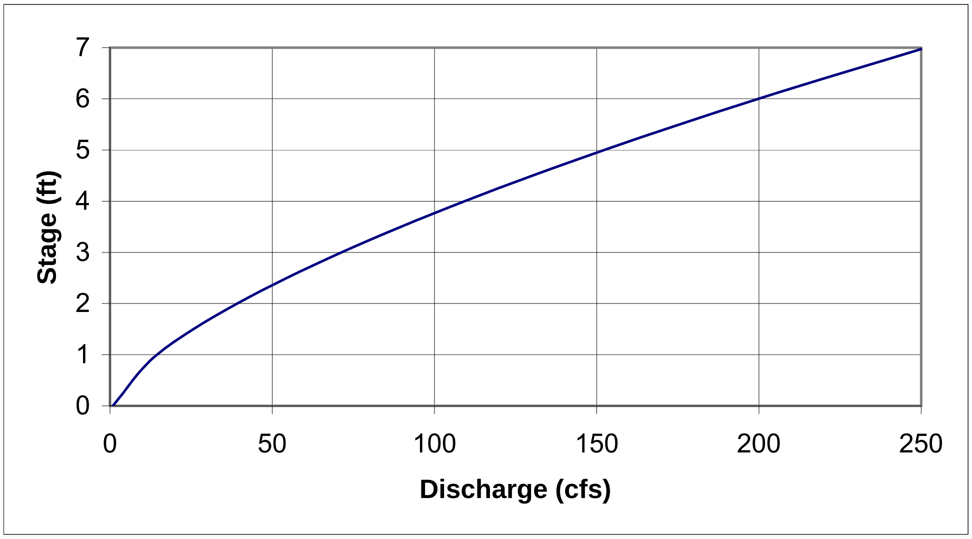


Figure 8-14 Example Stage-Discharge Curve

8.4.7.2 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 are usually developed using a topographic map and one of the following formulas: the average-end area formula, frustum of a pyramid formula, or prismoidal formula. 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:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (8.12)$$

Where:
$V_{1,2}$ = storage volume between elevations 1 and 2, cf
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, sf
d = change in elevation between elevations 1 and 2, feet

The frustum of a pyramid is expressed as:

$$V = d/3\,[A_1 + (A_1 A_2)^{0.5} + A_2] \qquad (8.13)$$

Where:
V = volume of frustum of a pyramid, cf
d = change in elevation between points 1 and 2, feet
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, sf

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)\,ZD^2 + 4/3\,Z^2\,D^3 \quad \textbf{(8.14)}$$

Where:
- V = volume of trapezoidal basin, cf
- L = length of basin at base, feet
- W = width of basin at base, feet
- D = depth of basin, feet
- Z = side slope factor, ratio of vertical to horizontal

8.4.7.3 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 auxiliary.

The principal spillway is usually designed with a capacity sufficient to convey the design flood before flow occurs in the auxiliary spillway. A pipe, outlet culvert, weir, or combination outlet can be used for the principal spillway. Tailwater influences and structure losses must be considered when developing discharge rating curves.

The auxiliary spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway.

The stage-discharge curve is a summation of the discharge characteristics of both the principal and auxiliary spillways.

8.4.7.4 Example Riser Pipe/Outlet Tube Stage-Discharge Curve

A detention pond is to be designed to release storage through a riser pipe and outlet tube. The following example shows how to develop a stage-discharge curve for these type of outlet structures. See Figure 8-15, Riser Pipe/Outlet Tube Structure.

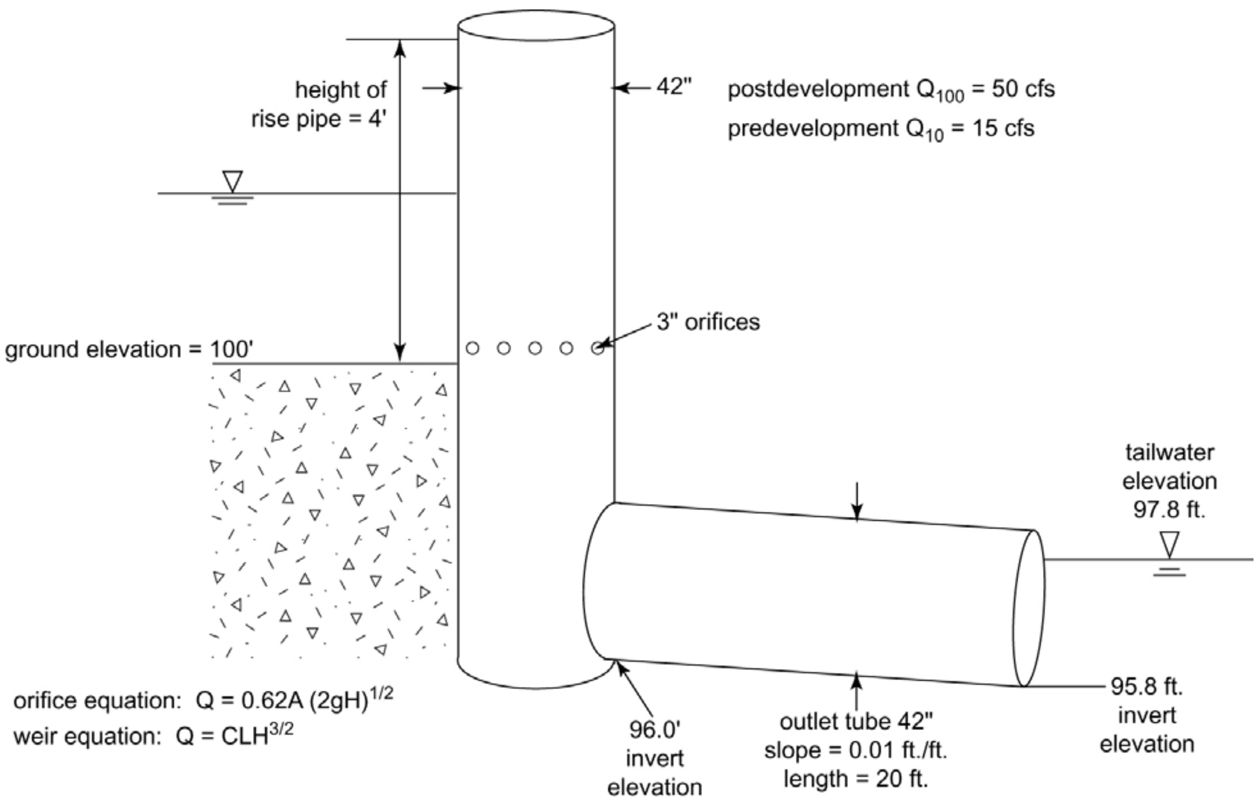


Figure 8-15 Riser Pipe/Outlet Tube Structure

The orifices should be sized to release at the pre-development 10 percent chance (10-year) rate when the basin is full. Use 3-inch diameter holes.

$$Q = 0.62\,[\pi\,((3/12)/2)^2]\,(2 \times 32.2 \times 4 \text{ feet})^{1/2}$$
$$= 0.5 \text{ cfs}$$

Q_{10} = 15 cfs

15/0.5 = 30 holes

Check that this many holes will fit around the riser pipe.

30 holes (3-inch/1 hole) = 90 inches

circumference of pipe = π D = π (42) = 131 inches

131 > 90 OK

Calculate the stage-discharge curve by finding the outflow from the orifices as the stage increases in 0.5-foot increments. Once the stage reaches the riser pipe rim elevation, add in the weir flow coming in through the top of the pipe; the bottom of the orifices are set at Stage 0.

When stage is at 0.5-foot flow through the orifices:

$$Q = [0.62\ A\ (2\ gH)^{0.5}]\ 30\ \text{orifices}$$

$$A = \pi\ [(3/12)/2]^2 = 0.0491\ \text{sf}$$

$$Q = [0.62\ (0.0491)\ (2 \times 32.2 \times [0.5 - 0.125])^{0.5}]30$$

$$Q = 4.5\ \text{cfs}$$

Continue this until reaching the riser pipe rim elevation (see Table 8-7).

Verify the riser pipe will pass the 100-year post-construction flow (minus the flow through the orifices) without exceeding the 1.5 feet of freeboard.

$$Q_{100} - Q_{10} = 50\ \text{cfs} - 15\ \text{cfs} = 35\ \text{cfs}$$

$$Q = CLH^{3/2}$$

$$Q = 35\ \text{cfs}$$

$$L = \pi\ D = \pi\ (42/12) = 11\ \text{feet}$$

$$C = 3.2$$

$$H = (Q/CL)^{2/3}$$

$$= [35/(11 \times 3.2)]^{2/3}$$

$= 1.0$ foot OK, water surface elevation meets 1.5-foot freeboard requirements.

Continue to calculate the stage-discharge curve through the stage 1 foot above the riser pipe rim.

Table 8-7 Riser Pipe/Outlet Tube Stage-Discharge Example

Stage	Orifice Discharge	Weir Discharge (Over Riser Pipe Rim)	Discharge Total
0	0	0	0
0.5	4.5	0	4.5
1.0	6.9	0	6.9
1.5	8.6	0	8.6
2.0	10.0	0	10.0
2.5	11.3	0	11.3
3.0	12.4	0	12.4
3.5	13.5	0	13.5
4.0	14.4	0	14.4
4.5	15.3	12.4	27.8
5.0	16.2	35.2	51.4

Graph the results of the total discharge versus stage.

Use the culvert worksheets and charts in Chapter 5, Culverts, to verify that the maximum design discharge does not submerge the orifices. By doing this, we find that the flow, 51.4 cfs, does not submerge the orifices. Therefore, the orifices are the control for basin water surface elevations up to the riser pipe rim elevation.

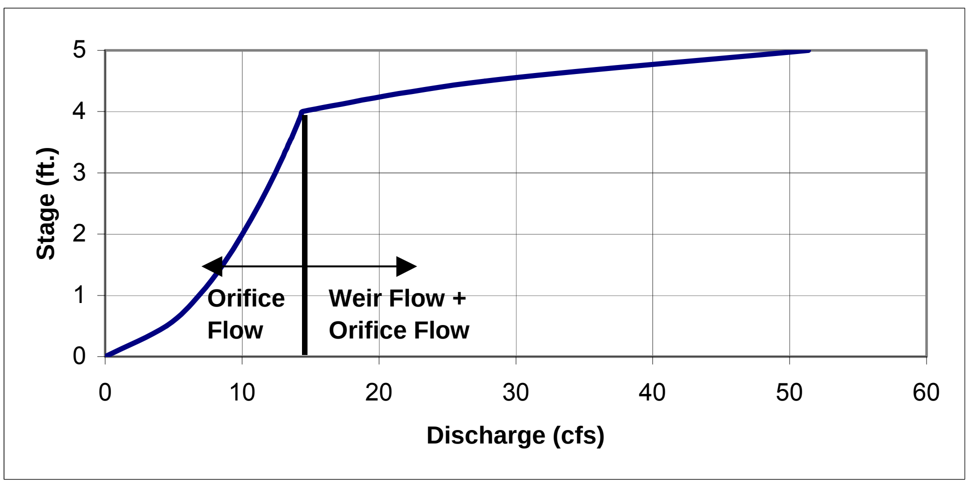


Figure 8-16 Riser Pipe/Outlet Tube Stage-Discharge Curve

8.4.7.5 Routing Procedure

A general procedure for using the above data in the design of storage facilities is presented below (assuming a 10 percent chance (10-year) storm existing conditions release rate).

Step 1 Determine the 10 or 4 percent and 1 percent chance (10-, 25- and 100-year, respectively) existing and proposed design flood hydrographs for the storage facility.

Step 2 Determine the allowable outflow from the facility based on existing flows. This may be controlled by downstream conditions, local regulations, or other factors. Use engineering judgment to determine appropriate parameters.

Step 3 Determine/calculate approximate volume of detention storage required to attenuate the post-development inflow hydrograph peak flow to the allowable outflow for each of the design storms.

Step 4 Based on the storage volume calculated, design a preliminary grading plan for the proposed facility, allowing for 1.5 feet freeboard above the maximum storage elevation.

Step 5 Design a principal spillway that will control the design release rate and provide storage volume for the storage facility. Perform routing calculations to verify the facility design.

Step 6 Adjust basin grading and outlet as necessary to fine tune the design to function within the established parameters.

Step 7 Design an auxiliary spillway to pass the 1 percent (100-year) flood flows. Check freeboard is 1.5 feet or more.

Step 8 Finalize plan and design documentation.

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

8.4.8 Routing Calculations

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

Use the following equations:

$$(I_1 + I_2) + (2S_1/dt - O_1) = (2S_2/dt + O_2) \tag{8.15}$$

Where subscripts 1 and 2 refer to the values at time t - dt and t, respectively and

I = inflow (cfs)
S = storage (acre-ft.)
dt = change in time (s)
O = outflow or discharge (cfs)

Step 1 Develop or obtain an inflow hydrograph for the post-development conditions.

Step 2 Develop the stage-storage and stage-discharge characteristics for the reservoir or storage facility as shown in the example problem Table 8-8. Combine these characteristics to develop a relationship between stage and 2S/dt + O.

Step 3 Develop a routing procedure table by routing the known inflow hydrograph through the storage facility. Use the following procedure:

1. Substitute known values of I_1 and I_2 and $2S_1/dt - O_1$ into the left hand side of Equation 8.15. This gives the value of $2S_2/dt + O_2$. Assume the storage is zero at the initial stage of zero.
2. From the reservoir characteristics table, determine the discharge, O_2, corresponding to the calculated value of $2S_2/dt + O_2$ (by interpolation). Also determine the stage of the storage facility (by interpolation).
3. Subtract $2O_2$ from $2S_2/dt + O_2$ to yield $2S_2/dt - O_2$ at the end of this time step.
4. Repeat Steps 1 to 3 until the entire outflow hydrograph, O(t), is calculated.

Step 4 Graph the results of the 1 percent chance (100-year) storm inflow and routed outflow vs. time. Verify the routed peak flow does not exceed the allowable peak release rate (the pre-development 10 percent chance (10-year) storm peak flow).

Because of the large amount of computations necessary to perform routing, a computer tool is typically used to complete the procedure. The procedure lends itself well to spreadsheets or commercially available computer software such as TR-55 or HEC-1.

8.4.8.1 Example Problem - Hydrograph Routing Calculation

Step 1 The following data was obtained describing the post-development inflow hydrograph to a proposed detention basin during the 1 percent chance (100-year) storm, Table 8-8. The peak flow for the pre-developed 10 percent chance (10-year) storm is 60 cfs.

Table 8-8 Inflow Hydrograph Data

Time (min.)	1 Percent Chance (100-year.) Storm Inflow, I (cfs)
0	0
10	50
20	178
30	250
40	165
50	90
60	50
70	29
80	16
90	9
100	5
110	3
120	1

Step 2 The following stage-storage and stage-discharge characteristics (Table 8-9, Figure 8-17, Stage-Storage Curve, and Figure 8-18, Stage-Discharge Curve) were determined for the storage facility. The value for 2S/dt + O was calculated for each stage in the table. The length of the sharp-crested weir is 5 feet, and the orifice diameter is 4 inches. The orifice is centered 4 feet below the weir crest at the bottom of the pond.

Table 8-9 Storage Facility Characteristics

Stage	Storage (acre-feet)	Weir Discharge (cfs)	Orifice Discharge (cfs)	Total Outflow, O (Discharge) (cfs)	2S/dt + O (cfs)
0	0	0.0	0.0	0.0	0.0
1	0.3	0.0	0.4	0.4	44.0
2	0.8	0.0	0.6	0.6	116.8
3	1.6	0.0	0.8	0.8	233.1
4	2.9	0.0	0.9	0.9	421.9
5	4.5	13.5	1.0	14.5	667.9
6	6.8	38.2	1.1	39.2	1026.6
7	10	70.1	1.1	71.3	1523.3

Note: dt = 10 minutes = 600 seconds

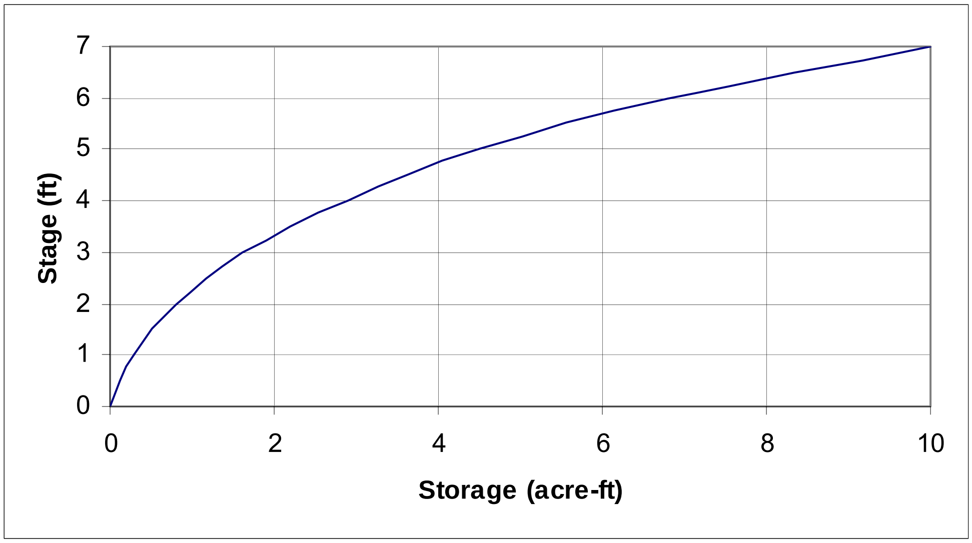


Figure 8-17 Stage-Storage Curve

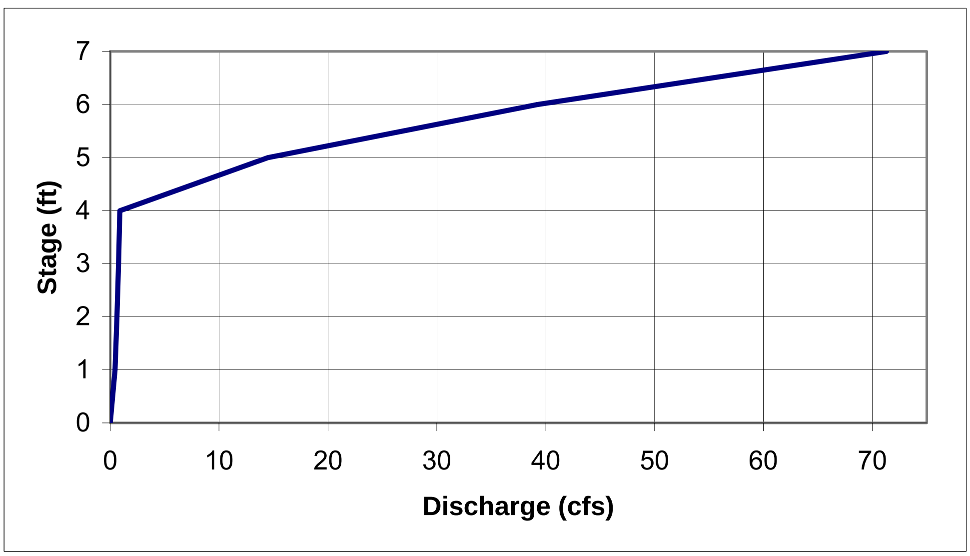


Figure 8-18 Stage-Discharge Curve

Step 3 The following routing procedure Table 8-10 was developed based on steps outlined in Section 8.4.7.

Example calculations for time = 20 minutes:

Inflow is known from Table 8-8 above.

Using Equation 8.15:

$(I_1 + I_2) + (2S_1/dt – O_1) = (2S_2/dt + O_2)$
I_1 = 50 cfs
I_2 = 178 cfs
$(2S_1/dt – O_1)$ = 49.2 cfs
$(2S_2/dt + O_2)$ = (50 + 178) + (49.2) = 277.2 cfs

Then from the value of 277.2 cfs for (2S/dt + O), interpolate the corresponding discharge, O, and stage of the storage facility by using the characteristics in Table 8-9.

Interpolation:
<u>Outflow (discharge)</u>
(421.9 - 277.2)/(421.9 - 233.1) = (0.9 - O)/(0.9 - 0.8)
O = 0.8 cfs
<u>Stage</u>
(421.9 - 277.2)/(421.9 - 233.1) = (4 - Stage)/(4 - 3)
Stage = 3.2 feet

The value for (2S/dt - O) is found by subtracting 2O from (2S/dt + O):
(2S/dt - O) = (277.2) - 2(0.8) = 275.5 cfs

Table 8-10 Routing Procedure Table

Time (minutes)	I (cfs)	2S/dt - O (cfs)	2S/dt + O (cfs)	O (cfs)	Stage (feet)
0	0	0	0	0	0
10	50	49.2	50.0	0.4	1.1
20	178	275.5	277.2	0.8	3.2
30	250	669.6	703.5	17.0	5.1
40	165	998.7	1084.6	43.0	6.1
50	90	1146.0	1253.7	53.9	6.5
60	50	1174.1	1286.0	56.0	6.5
70	29	1145.4	1253.1	53.8	6.5
80	16	1090.8	1190.4	49.8	6.3
90	9	1025.9	1115.8	45.0	6.2
100	5	959.8	1039.9	40.1	6.0
110	3	897.5	967.8	35.2	5.8
120	1	840.3	901.5	30.6	5.7

Step 4 Plot the results of the routed 1 percent chance (100-year) storm inflow and the post-developed 1 percent chance (100-year) storm hydrograph, Figure 8-19, Plot of Routed Hydrograph. Verify the routed peak outflow (56.0 cfs) does not exceed the pre-developed peak discharge (60 cfs).

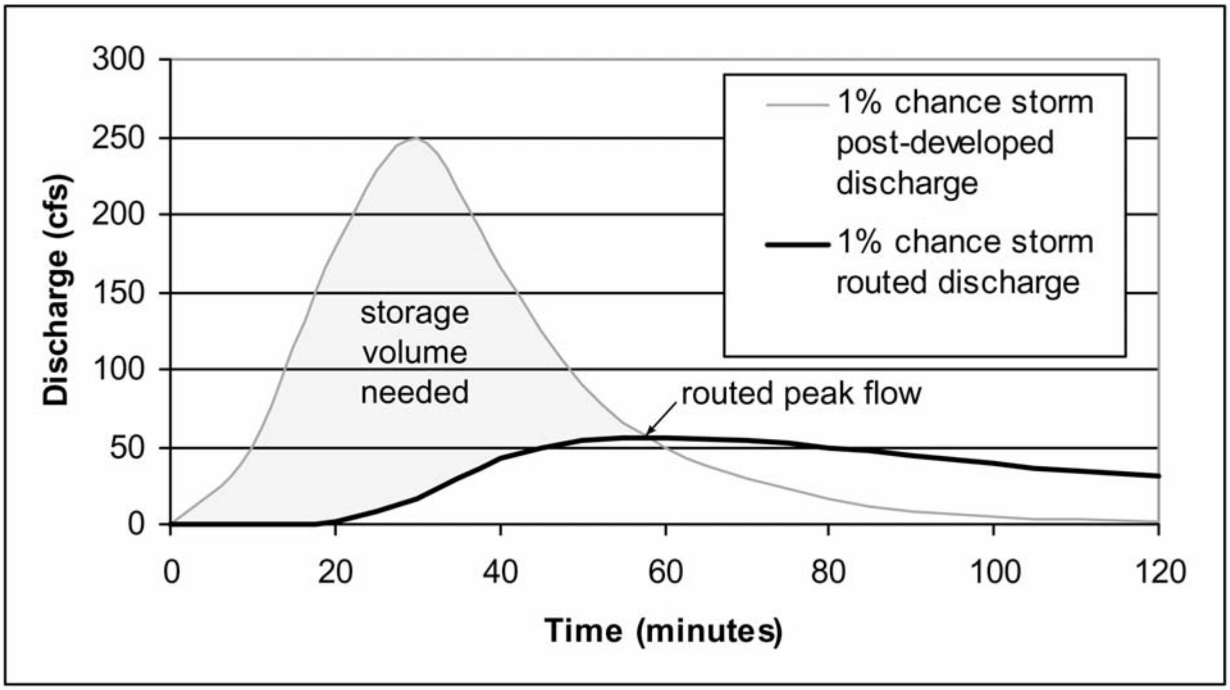


Figure 8-19 Plot of Routed Hydrograph

8.5 MAINTENANCE

8.5.1 Introduction

Storage facilities are often used in urban drainage systems to temporarily store or detain excess stormwater runoff and then release it at a regulated rate to downstream areas. Using this approach, runoff is stored in constructed or natural basins from which it is released continually until the water elevation in the facility reaches its design dry-weather stage. To function properly, the storage volume must be maintained at its design level and outlet facilities must be kept open and free from obstructions or clogging.

An important step in the design process is identifying whether special provision specifications are warranted to properly construct or maintain proposed storage facilities. To minimize long-term operational costs, storage facilities that require extensive maintenance are discouraged. The following maintenance problems are typical of urban detention facilities, and facilities shall be designed to minimize these problems:

- 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 potential for problems to develop.

- Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers.
- Constructing traps to remove sediment may control sedimentation. Sediment traps also reduce erosion and sediment transport in low-flow channels.
- Bank deterioration can be controlled with protective lining or by limiting bank slopes.
- Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or by constructing underdrain facilities to lower water tables.
- In general, when the above problems are addressed, mosquito control will not be a major problem.

- Outlet structures should be selected to minimize the possibility of blockage (i.e., very small pipes tend to block quite easily and should be avoided). Weirs are recommended with either sheet piling or earth embankment weirs.
- Finally, one way to deal with the maintenance associated with litter and damage to fences and perimeter plantings is to locate the facility for easy access where this maintenance can be conducted on a regular basis.

8.5.2 Maintenance Program

A good maintenance program should include preventive maintenance as well as corrective maintenance. The maintenance program should include:

- periodic inspection, adjustment, replacement;
- preventive maintenance - doing maintenance to prevent problems from occurring, such as removal of debris at inlets; and
- corrective maintenance - making changes to the system so that system functions as intended at the lowest annualized cost, such as resetting an inlet to reduce ponding.

The major prerequisite for a preventive and corrective type of program is an assured source of funds. The other component of a maintenance program, emergency repairs, is characterized by crisis-type responses to problems that go unrecognized, or are unattended, over a lengthy period of time. Often, such problems are treated as emergencies and, in some instances, little time is available for study, design, competitive bidding, etc.

8.5.3 Inspection Intervals

Inspection of major detention facilities should be made as frequently as experience shows necessary, perhaps monthly as a minimum and more often in wet seasons. Where debris is a problem, inspections must be spaced according to debris generation. In any event, it is important to conduct inspections and cleanup work following major individual runoff events. It is sometimes necessary to make inspections during rainstorms when intense rainfall occurs.

Besides removing debris blockages during inspections, mechanical equipment such as generators, float valves, pumps, discharge controls, and other electrical and mechanical equipment should be checked and adjusted as necessary.

Inspect detention basins once a year, preferably during wet weather. Inspect after major flood events. Basins should be mowed as required (at least twice a year) and sediment should be removed every 5 to 10 years.

Infiltration facilities should be inspected at least once a year.

Observation of time to drain is an early indicator of plugging and need for maintenance. Maintenance would involve removal of silt material or debris that lead to reduction of infiltration capacity of the basin. Returning basin to design grade is critical to keep design storage volume.

8.5.4 Maintenance Tasks

Maintenance tasks can be grouped in three general categories:

- aesthetic maintenance,
- nuisance maintenance, and
- operation maintenance.

Of these, the most important, from the standpoint of health and safety, is operation maintenance.

8.5.5 Operation Maintenance

This category can be characterized as that level of maintenance required to ensure against failure of major structural components and/or flow controls, and to ensure that the facility continues to function as designed. Neglecting this level of maintenance could cause dam failure and subsequent property damage as well as possible loss of life. In addition, neglect often causes a facility to cease functioning as it was originally designed to do. A program of scheduled, periodic inspections of the facility is essential to recognize potential structural maintenance needs. The following is a partial list of items that should be checked periodically and corrective action taken as required:

- settling of embankment,
- woody growth on the embankment (roots can create channels for dam leakage and eventual failure),
- signs of piping (leakage) through embankment,
- signs of seepage or wet spots on the downstream face of an embankment (may require toe drains or chimney drains to solve problems),
- riprap failures,
- deterioration of primary and auxiliary spillways,
- various stage/outlet controls,
- effectiveness of debris racks,
- outlet channel conditions,
- safety features (access controls to hazardous areas),
- mechanical and electrical equipment (pumps, generators, automatic controls, etc.),

- access for maintenance equipment,
- availability of manufacturer's mechanical and electrical information manuals, and availability of design information such as rating curves and tables for spillway flow, bypass flow, total flow, and storage and pump-out calculations.

In addition, the following actions should be taken on each facility, as required.

- Replace soil removed by rodent burrows.
- Inspect drainage systems and relief wells annually for proper functioning and clean out or replacement as necessary.
- Maintain riprap or other wave-protective measures and replace as needed.
- Remove and/or stabilize slide material as soon as practical. It may be necessary to construct a berm or flatten the slope.
- Replace eroded material and establish vegetation in eroded areas in emergency spillways, swales and other areas.
- Repair any unusual seepages, boils, or settlements in fill areas, or sinkholes in pool areas.

Also, observations should be made of any changes in topography, downstream drainage systems or land uses that may have a bearing on the operational effectiveness and safety of detention facilities.

Infiltration facilities should be promptly removed of any accumulated sediment. The pervious bed of the infiltration facility should be replaced if no longer functioning.

8.5.6 Volume Maintenance

One of the most important variables in the design of a detention facility is the volume available for storage of runoff. If a detention facility is allowed to accumulate sediment and debris which will decrease the storage volume, the ability of the facility to function as designed can be greatly reduced. Thus it is essential to maintain the design volume. To facilitate the inspection of the facilities for volume control, it is recommended that some marker be installed in the detention facility to indicate the maximum level for silt buildup before the facility must be dredged or cleaned. This marker could be a small pipe with a stripe or suitable indicator at the cleanout level. A suitable indicator could also be place on the outlet device or in some location which can be easily identified during the inspection process.

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Michigan Association of County Drain Commissioners:

www.macdc.net

Michigan Department of Environmental Quality (BMP Manual):

Guidebook:
www.michigan.gov/deq

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Impact on Hydrology due to development:
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