

CHAPTER 8

STORMWATER STORAGE FACILITIES FOR FLOOD CONTROL



Note: For trunkline projects, direct questions and comments to the Hydraulic Unit Supervisor, Environmental Services Section.

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Appendices

Appendix 8-A	Symbols and Acronyms
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If you require assistance accessing this information or require it in an alternative format, contact the Michigan Department of Transportation's (MDOT) Americans with disabilities Act (ADA) coordinator at [Michigan.gov/MDOT-ADA](https://www.michigan.gov/MDOT-ADA).

8.1 INTRODUCTION/PURPOSE

8.1.1 Storage Overview

The traditional purpose of stormwater drainage systems is to collect and convey stormwater runoff as rapidly as possible to a suitable location and discharged. However, as areas urbanize, this type of design may increase flooding problems downstream and limit opportunities to treat stormwater. In most situations, the temporary storage of stormwater reduces the rate of stormwater discharge from a project area using storage facilities and provides the following potential benefits:

- Reduces increased peak runoff rate caused by increased impervious areas
- Mitigates downstream drainage capacity problems
- Recharges groundwater resources
- Reduces or eliminate the need for downstream outfall improvements
- Maintains historic flow rates by controlled discharge from storage
- Provides sediment removal

This chapter provides general design criteria for detention, wet detention, and infiltration storage facilities, as well as procedures for performing basin sizing and reservoir routing calculations for flood control. Design criteria for storage facilities required for National Pollutant Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) treatment can be found in the [MDOT Stormwater Treatment Manual](#).

8.1.2 Acceptable Outfalls and Points of Discharge

Refer to Chapter 7, Inlet and Enclosed Storm Drainage Systems, for examples of acceptable outfalls and points of discharge from storage facilities.

8.1.3 Computer Programs

Hydrograph routing calculations, known as the Modified Puls Method (**Section 8.7.3**), is used to design storage facilities. Although Modified Puls calculations are not complex, they are time-consuming and repetitive. It is recommended that the designer utilize the FHWA Hydraulic Toolbox to perform reservoir routing calculations. Refer to Chapter 3, Hydrology for additional programs.

8.2 DEFINITIONS

Auxiliary Spillway - A flow path that bypasses the principal spillway for flows that exceed the design flow conditions (flow and capacity).

Detention Basin - A storage facility incorporated into the watershed to temporarily store runoff by reducing the peak of the runoff hydrograph. Water is released over time through a structural control and no permanent pool remains.

Infiltration Basin or Trench - A storage facility or trench that discharges stored water into the ground. A basin or trench used for flood control has no auxiliary flow path up to the design event.

Outfall - The location at which a point source discharge first enters a surface water of the state.

Outlet - The location at which a point source discharge leaves the MDOT right-of-way and discharges to something other than a water of the state.

Point of Discharge - The location of a point source discharge where stormwater is discharged directly into a separate storm sewer system.

Principal Spillway - A structural control designed with sufficient capacity to convey the design flood before flow occurs in the auxiliary spillway.

Routing - The process of transposing an inflow hydrograph through a detention basin and determining the outflow hydrograph from the structure.

Sediment Forebay - A portion of a storage facility that provides coarse sediment capture to extend the life of the storage facility and ease maintenance efforts.

Stormwater Control Measure (SCM) - A temporary or permanent physical feature, structural device, or nonstructural practice that is designed to reduce or prevent pollutants from entering stormwater runoff or to treat polluted stormwater. A SCM can also be used to mitigate and manage flood risks. When used for long-term impacts of stormwater runoff from a project, a SCM is called a post-construction stormwater control measure (PC-SCM). An example of a SCM for flood control is a storage basin.

Storage Facility - A drainage control facility that is designed to hold water for a considerable length of time and then release it at a controlled rate or infiltrate the stored water.

Structural Control - Use of engineered facilities and restricted outlets to control the rate of stormwater runoff released from a storage facility.

Waters of the State - Groundwaters, lakes, rivers, and streams and all other watercourses and waters, including the Great Lakes, within the jurisdiction of this state.
MCL 324.3101

Wet Detention Basin - A basin or reservoir that stores water and regulates peak discharge. It has a controlled outlet. The stored water is released to the downstream drainage system during and after a storm event. The release is through a gravity system or a pumped system. The basin maintains a permanent pool elevation, which is not included in the stage-storage curve.

Wetland - As defined in Part 303 of the Natural Resources and Environmental Protection Act (NREPA) (Public Act 451 of 1994), a wetland means a land or water feature, commonly referred to as a bog, swamp, or marsh, inundated or saturated by water at a frequency and duration sufficient to support, and that under normal circumstances does support, hydric soils and a predominance of wetland vegetation or aquatic life. A land or water feature is not a wetland unless it meets any of the following criteria:

- Is a water of the United States as that term is used in section 502(7) of the Federal Water Pollution Control Act, 33 USC 1362.
- Is contiguous to the Great Lakes, Lake St. Clair, an inland lake or pond, or a stream. As used in this subparagraph, "pond" does not include a farm or stock pond constructed consistent with the exemption under section 30305(2)(g).
- Is more than 5 acres in size.
- Has the documented presence of an endangered or threatened species under part 365 or the Endangered Species Act of 1973, Public Law 93-205.
- Is a rare and imperiled wetland.

A list of symbols and acronyms used in this chapter is provided in **Appendix 8-A**.

8.3 POLICY

The use of storage facilities for stormwater management and flood control has increased dramatically in recent years due to increased urbanization, resiliency planning, and regulatory requirements. When the downstream system at the point of discharge lacks capacity, a storage facility is designed to restrict and reduce the peak discharge to a specified flow rate. Peak flow is greater than the restricted discharge rate stored in the basin. Standard practice is to limit post-project discharge rates to pre-project rates for a range of flows up to and including the 1% AEP (100-year), 24-hour storm event. Outflow rates may be regulated by:

- NPDES MS4 permit requirements for channel protection
- Local agency ordinances/regulations
- County drain offices
- Part 31 of NREPA

Refer to Chapter 2, Policy and Procedures for Federal and State policies. For more design and engineering guidance, refer to the *AASHTO Drainage Manual* (2014), and FHWA HEC-22 (2024).

8.4 TYPES OF STORAGE FACILITIES

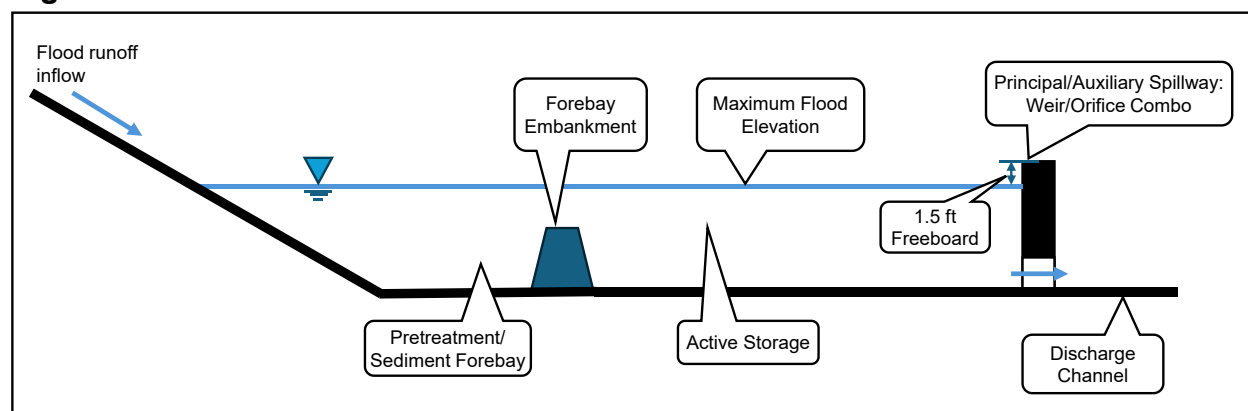
8.4.1 Detention Basin

Dry pond detention basins are depressed areas designed to reduce peak flow and store runoff during wet weather. They are dry the remainder of the time. Detention basins have comparatively low cost when built in the MDOT right-of way (ROW), few design limitations, and can serve both large and small watersheds. A schematic of a detention basin is shown in **Figure 8-1**.

Use detention basins in areas where:

- Suitable depressions occur or can be constructed. From a cost standpoint, place the basin where the least amount of earth moving is required to obtain the necessary basin volume.
- Acceptable inflow and outflow conditions can be achieved.
- Flow can enter and discharge from the basin by gravity.
- Soils provide a stable embankment.
- Storage volume for flood control is adequate to detain the runoff from the specified storm event.

Figure 8-1. Detention Basin Schematic



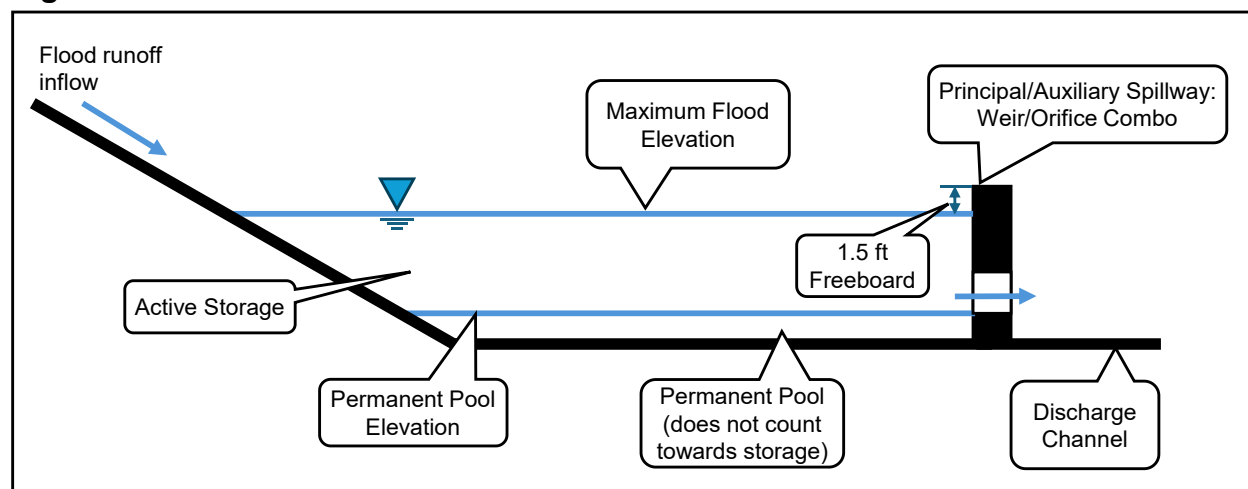
8.4.2 Wet Detention Basin

Wet detention basins are similar to detention basins except they retain a permanent pool during dry weather periods. Wet detention basins may be more expensive to construct and maintain than detention basins. They may be restricted by some agencies, such as the Federal Aviation Administration (FAA).

The use of wet detention storage facilities requires more maintenance, inspection of sediment buildup underwater, and may cause safety concerns compared to detention basins. Consider provisions for weed control and aeration for preventing anaerobic conditions. Do not build facilities that have the potential to become nuisances or health hazards. Consider the effect of wet detention basins in highly populated areas with

respect to vectors (pathogen carrying organism). A schematic of a wet detention basin is shown in **Figure 8-2**.

Figure 8-2. Wet Detention Basin Schematic

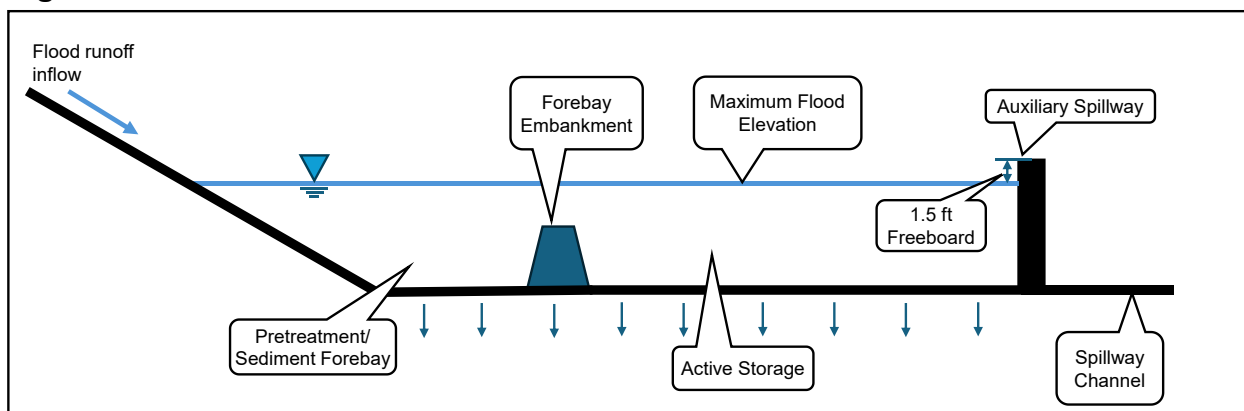


8.4.3 Infiltration Basin

Infiltration basins are storage facilities where flow above the allowable discharge rate is diverted and stored without a principal discharge. Stored flow is dewatered by infiltration through the basin soil to the groundwater table. During extreme events that exceed the basin capacity, stormwater flow is released over an auxiliary spillway. A schematic of an infiltration basin is shown in **Figure 8-3**. For trunkline projects, use of infiltration basins for flood control requires approval by the Hydraulic Unit Supervisor.

Infiltration basins can adapt to fit the requirements of highway systems, i.e., shaped into linear basins, constructed in borrow areas, or linked in a series of small basins. Infiltration systems are primarily applied in cases where ROW is a limiting factor, the soil is highly permeable, and the seasonal high groundwater table is 5 feet or more below the bottom of the basin. Ensure the underlying bedrock is a minimum of 5 feet below the bottom of the basin.

Over time silt may plug soil voids reducing the infiltration capacity requiring basin maintenance. For this reason, pretreatment is required upstream of the infiltration facility to reduce silt and sediment buildup. Infiltration basins require regular inspection for standing water. The clogging of basins from lack of maintenance may lead to failure. Infiltration basins are limited during the winter months when the ground is frozen. Limit the use of infiltration basin storage facilities for these reasons. Examine the overflow route if overtopping of the basin were to occur due to reduced infiltration rates or events greater than the design event.

Figure 8-3. Infiltration Basin Schematic

8.4.4 Underground Storage

In urban environments where land available for detention is limited, consider providing detention in buried, oversized storm sewers. Gravity discharge of the pipes is preferred; however, because of their depth and surrounding surface topography, a pumped outlet may be required (Refer to Chapter 10, Pump Stations and Section 7.8 of Chapter 7, Inlet and Enclosed Storm Drainage Systems). Due to the variety of underground storage configurations, verify with Maintenance that the access point spacing is acceptable.

8.5 STORAGE DESIGN CRITERIA

8.5.1 General Design Criteria Common to All Storage Facilities

Storage may be concentrated in large regional facilities or distributed throughout a drainage system using multiple smaller basins. The effectiveness of a storage facility depends on the amount of storage required, location within the system, and operational characteristics. Analysis of storage facilities consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage.

General design criteria for storage facilities includes:

- Discharge rate
- Storage volume
- Safety
- Grading and depth requirements
- Pretreatment
- Structural controls
- In-line and off-line storage configurations
- Location
- Soil borings to determine groundwater depths and soil strata

Design considerations that are common to all storage facilities are discussed in detail as follows. Design considerations unique to each storage facility type are discussed in their respective sections.

8.5.1.1 Discharge Rate

Manage stormwater runoff in storage facilities by limiting peak runoff rates using the following:

- Existing rates for a range of design flows: 50%, 10%, 2%, 1% AEP (2-year, 10-year, 50-year, 100-year), 24-hour design storms (i.e., post-project peak restricted to pre-project peak for a particular frequency of occurrence).
- Discharge capacity if limited by the downstream drainage system.
- A specified value for allowable discharge set by local regulations.
- Infiltration rate does not contribute to the allowable outflow rate.

8.5.1.2 Storage Volume

Store the routed 1% AEP (100-year), 24-hour design storm with 1.5 feet of freeboard below the lowest part of the basin perimeter. Use routing calculations to demonstrate that the storage volume is adequate. What is included in storage volume for each facility is described in the storage facility sections that follow.

8.5.1.3 Safety

- Maintenance Access – Provide a 10-foot wide vegetative top of berm or access above the freeboard elevation and around the entire perimeter of the basin, unless maintenance equipment can access from other locations. Provide a vehicular gate for maintenance. For special situations on trunkline projects, contact the Hydraulic Unit Supervisor.
- Fencing – Fence the storage facilities to prevent unauthorized entry to facilities. If there is a permanent water pool of 2 feet or more, a 6-foot chain link fence with a locking gate is required, unless ownership is transferred to private interests or another public agency that does not require a fence. In limited access areas, ROW fencing may be substituted. Locate the fence beyond the 10-foot maintenance ledge.
- Protective Treatments – If water is impounded to a depth greater than 2 feet during the design event, locate the basin outside the clear zone (defined in Section 7.01(DG) of the MDOT Road Design Manual) or shield from errant vehicles using protective treatments, such as guardrails.
- Signing – Post signs deterring public access.

8.5.1.4 Grading and Depth Requirements

Construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Coordinate work with the geotechnical engineer to develop the appropriate field investigation and suitable embankment material. Refer to the MDOT Geotechnical Manual for more specific guidance. Stability of the basin embankments and natural slopes require:

- Soil boring(s) for embankment basin area and outlet structure.
- Determination of groundwater elevation, including the seasonal high groundwater elevation.
- Embankment side slopes of 1V:4H or flatter preferred, 1V:3H maximum unless otherwise approved by the Geotechnical Service Section and Maintenance.
- Grassed slopes per MDOT standards regarding turf establishment. Tree and shrub plantings are prohibited. If a different vegetative cover is desired, contact the MDOT Roadside Development Unit Manager.
- An embankment top of berm or access with a minimum top width of 10 feet around the entire perimeter of the basin, unless maintenance equipment can access from other locations. For special situations on trunkline projects, contact the Hydraulic Unit Supervisor.
- Avoid placing any auxiliary spillway over a constructed embankment unless erosion protection, such as heavy riprap, is placed on the embankment. Refer to *Design of Small Dams* (U.S. Bureau of Reclamation, 1987, Section 6.22) for riprap sizing guidance.

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 the storage facility is graded based on the basin type. Bottom grading is discussed in the following section unique to facility type.

8.5.1.5 Pretreatment

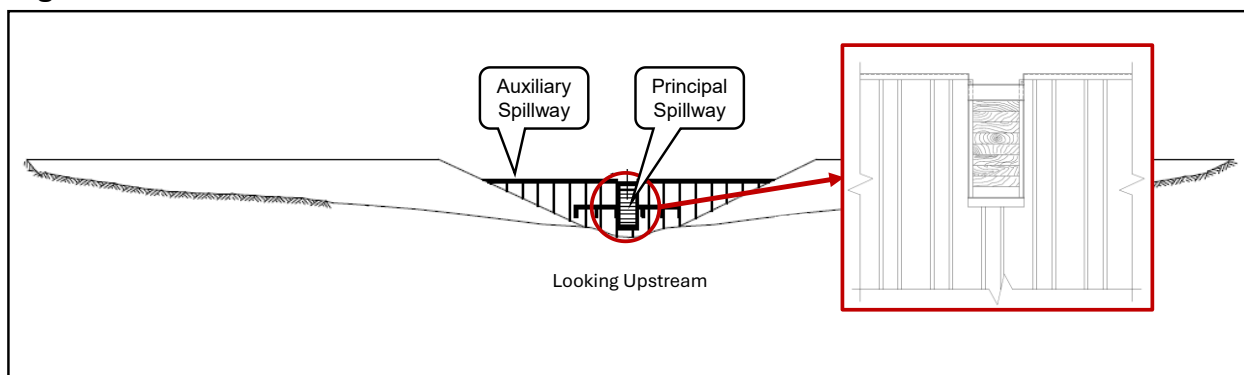
Provide pretreatment to extend the service life of the flood control facility. Pretreatment is required for all basins except wet detention basins.

Provide a vegetated swale or sediment forebay at the entrance to capture sediment and extend the life of the basin. Design the outlet to the sediment forebay to spread the flow evenly to reduce erosion and more effectively capture sediment before entering the main storage of the basin. Configure the outlet of the forebay to fully drain for a duration similar to the full body of the basin. Place riprap at the inlet to help reduce erosion. A riprap berm with or without a connecting outlet pipe separates the sediment forebay from the main basin.

8.5.1.6 Structural Controls

Structural controls selected for storage facilities typically include a principal spillway and an auxiliary spillway. Sheet pile weirs with a low-flow orifice are preferred to be used as the principal and auxiliary spillway based on constructability, maintenance, and safety considerations. Details of the control are included in **Appendix 8-B**. A cross section of the weir/orifice combo is shown in **Figure 8-4**. The principal spillway conveys flow through the orifice and the stop log weir. The top of the stop log weir is required to be less than or equal to 5 feet above the invert of the basin. Size the principal spillway based on results of routed 1% AEP (100-year) flow. Varying outlets are described in detail in **Section 8.6**.

Figure 8-4. Cross Section View of MDOT Weir



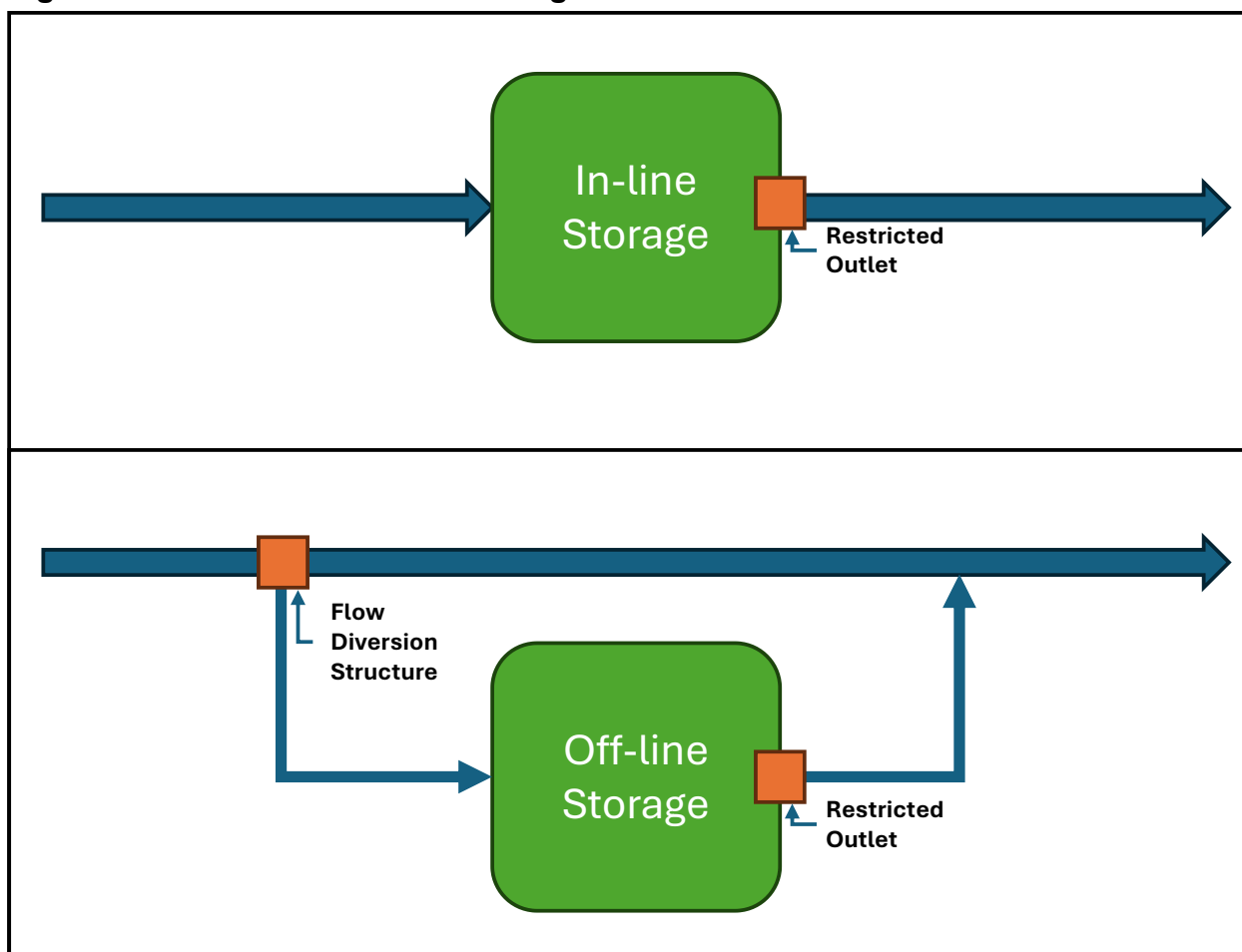
For events greater than the design storm up to the 0.2% AEP (500-year), 24-hour event, verify flow can safely pass through the storage and outlet to the downstream system without upstream flooding risk or damage to the storage and outlet structure. The

auxiliary spillway conveys flow greater than the 1% AEP (100-year) flow over the steel sheet pile portion of the weir structure.

8.5.1.7 In-line and Off-line Storage Configurations

Storage facilities are constructed to reduce the peak flow from an upstream tributary area resulting from a transportation project. Most facilities are constructed in-line, with the flow entering the facility directly with a restricted flow outlet. An in-line storage facility configuration partially fills the available storage prior to peak of the design flow rate. If space is limited, the designer may consider off-line storage to minimize the total storage volume required. Off-line storage includes a flow diversion structure in-line with the flow that directs flow toward the constructed storage only when flow approaches the limiting design flow rate. Off-line storage requires a secondary flow restriction to limit flow back to the conveyance system. Off-line storage usually requires more hydraulic head to operate as a gravity-only system while preventing upstream flooding impacts. Off-line facilities are not practical in flat terrain or in systems where there is not sufficient hydraulic head. **Figure 8-5** shows the in-line and off-line storage flow schematics.

Figure 8-5. In-line and Off-line Storage Schematics



8.5.1.8 Location

The location of storage facilities impacts the effectiveness of the facilities to control downstream flooding and potential impacts of flooding upstream of the facility. Small facilities located upstream in the system have minimal flood control benefits that quickly diminish as the flood wave travels downstream. Moreover, the presence of multiple storage facilities within 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. Design the storage facility to control runoff from a defined area and evaluate the downstream impact to other drainage structures within the watershed or drainage area.

Identify utilities in the project area. Coordinate with utility owners to identify conflicts and resolutions, if necessary.

8.5.2 Design Criteria Specific to Storage Facilities

While storage facilities share many of the same design criteria, each type of facility has unique criteria as discussed in this section.

8.5.2.1 Detention Basin Design Summary

Basic design considerations for a detention basin are summarized in **Table 8-1**.

Table 8-1. Detention Basin Criteria

Parameter	Criteria
Storage Time	• Drain within 72 hours
Size	• Release at a peak rate equal to or less than existing for a range of storms up to and including the 1% AEP (100-year) event • Provide a water surface elevation during the 1% AEP (100-year) post-development storm 1.5 feet below the lowest part of the basin embankment • Construct basin within the MDOT ROW
Pretreatment	• Sized to contain runoff from 1 inch of rainfall • Drain within 72 hours
Grading	• Grade side slopes 1V:4H preferred (1V:3H maximum) • Grade the basin bottom at a 0.3% slope towards outlet to prevent ponding • Set the bottom elevation of storage facility 1 foot, at a minimum, above the seasonal high groundwater elevation

In situations where the detention basin bottom is below the low flow orifice, such as an infiltration Stormwater Control Measure (SCM), the volume below the orifice does not count towards storage.

8.5.2.2 Wet Detention Basin Design

Wet detention basins generally follow the same design standards as detention basins. Since wet detention basins retain a permanent pool of water, there are design standards specific to this feature.

Shallow impoundments are unattractive and visual screening and/or landscaping may be needed. Three feet of permanent water depth is recommended to alleviate problems with odor and weeds. Permanent pool elevation may be set by exposed groundwater. Provide a valve to drain the pond for maintenance.

Basic design considerations for a wet detention basin are summarized in **Table 8-2**.

Table 8-2. Wet Detention Basin Criteria

Parameter	Criteria
Storage Time	• Maintain a permanent pool • Drain the volume above the permanent pool within 72 hours
Size	• Release at a rate equal to or less than existing for all storms up to and including the 1% AEP (100-year) peak • Provide a water surface elevation during the 1% AEP (100-year) post-development storm 1.5 feet below the lowest part of the basin embankment • Construct basin within the MDOT ROW • Provide a permanent pool depth of 3 feet
Storage Volume	• Do not count volume below the permanent pool elevation towards the storage requirement
Pretreatment	• Not required
Grading	• Grade side slopes 1V:4H preferred (1V:3H maximum) • Grade the basin bottom at a 0.3% slope towards drainage valve for maintenance

If a wet detention basin is proposed to be used for storm control measures and flood control for trunkline projects, contact the Stormwater Program Manager for additional information.

8.5.2.3 Infiltration Basin Design

Infiltration basins dewater by infiltration primarily through the bottom of the basin. Basic design considerations for infiltration basins are summarized in **Table 8-3**. Consider the following when evaluating use of an infiltration basin over other basin types:

- Site selection is based on infiltration rate. The infiltration rate is an important parameter when an infiltration facility is designed. [NRCS Web Soil Survey](#) maps or the [MDOT PC-SCM Screening Tool](#) may be used for initial location.

The NRCS maps generally show soil characteristics for surface soils. Hydrologic soil group (HSG) A and sometimes B, have adequate infiltration rates for construction of an infiltration basin. After a suitable site for the facility is found, several soil tests are required before designing the facility. Take borings to a sufficient depth to support infiltration calculations at the site to determine the soil types, presence and elevation of groundwater, and infiltration rates, per Section 6.3.4.2 in the MDOT Geotechnical Manual.

- Suitable soils/subsoils are required to be moderately to highly permeable. A permeability test, such as the Falling Head test, is necessary. Permeability of native soils are required to be greater than 1 inch/hour for flood control purposes. Refer to Section 6.3.4.2 of the MDOT Geotechnical Manual for permeability testing requirements.
- Use a factor of safety of 2 when calculating basin footprint by reducing the actual infiltration rate by half. Use a maximum permeability rate of 10 inches/hour (after applying the infiltration factor of safety) for any tested permeability rate of 20 inches/hour or more.
- Design the infiltration basin with the seasonal high groundwater table 5 feet or more below the bottom of the storage area. Base initial groundwater table on the soil boring information. Check nearby waterbody information or install observation wells to determine the seasonal high groundwater table elevation.
- Size storage assuming no infiltration to contain the runoff volume from the 1% AEP (100-year) with 1.5 feet of freeboard contained within the MDOT ROW.
- The recommended allowable storage time is 24 to 72 hours. The maximum ponding depth for a required drain time is estimated using **Equation 8.1**:

$$d = fT_s \quad (8.1)$$

Where:

- d = Maximum ponding depth, inch
- f = Steady infiltration rate, inch/hour
- T_s = Time of storage, hour

- For underground infiltration storage, including bottomless catch basins (leaching basins) and manholes, construct the bottom of the facility 5 feet above the seasonal high groundwater table. The design water level is required to be below

the frost line. Underground infiltration storage is discouraged due to the inspection, sedimentation, and maintenance requirements.

- Provide emergency overflow or upstream bypass when volume exceeds infiltration storage volume.

Table 8-3. Infiltration Basin Criteria

Parameter	Criteria
Storage Time	• Infiltrate within 24 to 72 hours
Size	• Store the 1% AEP (100-year) storm volume assuming no infiltration with 1.5 feet of freeboard within the MDOT ROW
Storage Volume	• Do not count the infiltration rate towards the storage requirement
Pretreatment	• Sized to contain runoff from a 1-inch rainfall
Grading	• Grade side slopes 1V:4H preferred (1V:3H maximum) • Provide a flat bottom of the basin with zero slope • Set the basin bottom elevation 5 feet, at a minimum, above the seasonal high groundwater elevation

8.5.2.4 Underground Storage Design

When space is limited, consider underground storage. Calculate the required storage volume the same as for basins on the surface. Gravity discharge from the storage is preferred; however, because of their depth, a pumped outlet may be required (Refer to Chapter 10, Pump Stations).

A minimum slope for storage pipes will be required to maintain scour velocity and reduce sediment accumulation. For trunkline projects, contact the Hydraulic Unit Supervisor before starting design of an underground storage facility. Stormwater tunnels are discussed in Chapter 7, Road Storm Drainage Systems.

8.6 STRUCTURAL CONTROLS

Principal and 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 as follows. Equations for broad-crested weirs, v-notch weirs, and sloped weirs are also provided. Sheet pile weirs with a low-flow orifice are preferred. Drawings are included in **Appendix 8-B**.

8.6.1 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated in **Figure 8-6** and **Figure 8-8**. The discharge equation for this configuration in English Units is (Chow, 1959):

$$Q = \left[3.27 + 0.40 \left(\frac{H}{H_c} \right) \right] LH^{1.5} \quad (8.2)$$

Where:

- Q = Discharge, cfs
- H = Head above weir crest excluding velocity head, ft
- H_c = Height of weir crest above channel bottom, ft
- L = Horizontal weir length, ft

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

$$Q = 3.3LH^{1.5}$$

A sharp-crested weir with two end contractions is illustrated in **Figure 8-7**. The discharge equation for this configuration is (Chow, 1959):

$$Q = \left[3.27 + 0.40 \left(\frac{H}{H_c} \right) \right] (L - 0.2H)H^{1.5} \quad (8.3)$$

Where: Variables are the same as **Equation 8.2**.

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

$$Q_s = Q_f \left(1 - \left(\frac{H_2}{H_1} \right)^{1.5} \right)^{0.385} \quad (8.4)$$

Where:

Q_s = Submergence flow, cfs

Q_f = Unsubmerged weir flow from **Equation 8.2** or **Equation 8.3**, cfs

H_1 = Upstream head above crest, ft

H_2 = Downstream head above crest, ft

Figure 8-6. Sharp-Crested Weir – No End Contractions (FHWA HEC-22, 2024)

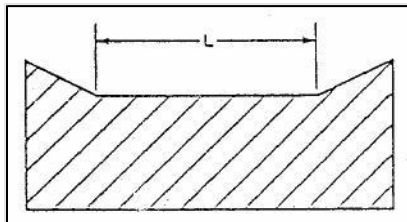


Figure 8-7. Sharp-Crested Weir – Two End Contractions (FHWA HEC-22, 2024)

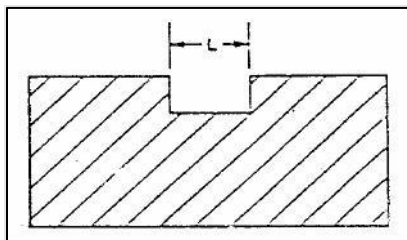
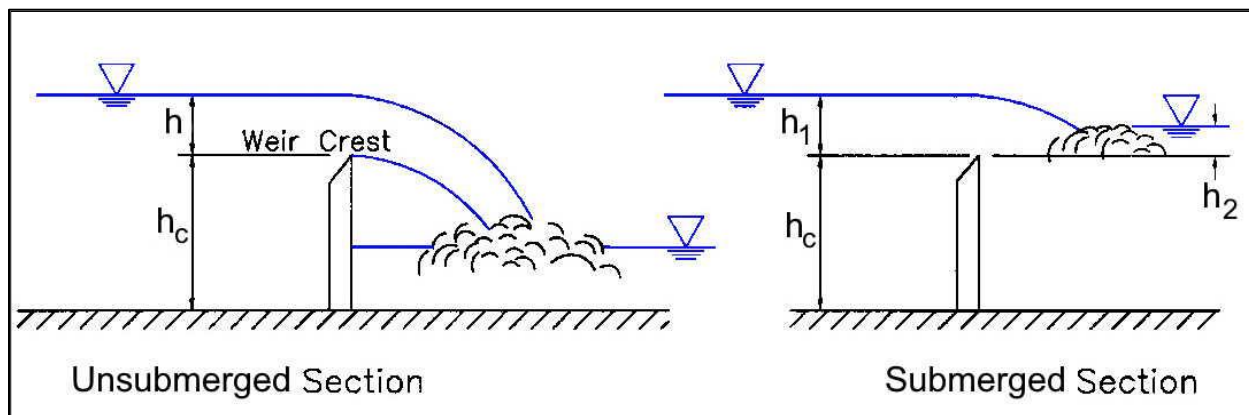


Figure 8-8. Sharp-Crested Weir and Head (FHWA HEC-22, 2024)



8.6.2 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 a broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{3/2} \quad (8.5)$$

Where:

- Q = Discharge, cfs
- C = Broad-crested weir coefficient
- L = Broad-crested weir length, ft
- H = Head above weir crest, ft

If the upstream edge of a broad-crested weir is rounded to prevent contraction and 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, use a minimum C value of 2.58. Additional information on C values as a function of weir crest breadth and head is provided in **Table 8-4**. Note that the head, H, is measured at least 2.5H upstream of the weir.

Table 8-4. Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head (Brater and King, 1976)

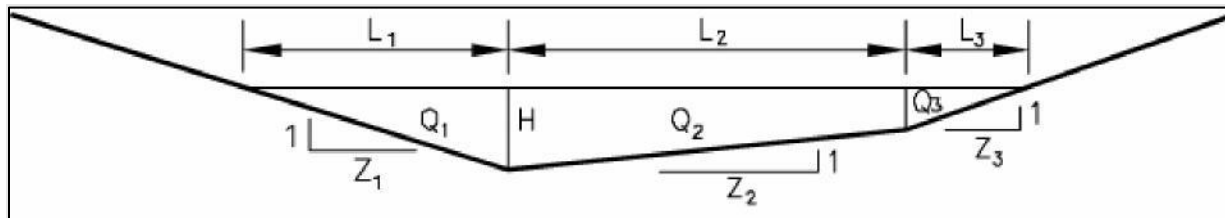
H* (ft)	0.5	0.75	1.0	1.5	2.0	2.5	3.0	4.0	5.0	10.0	15.0
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.6	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.6	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.6	2.6	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	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

8.6.3 Sloped Weirs

Sloped weirs are a special case of broad-crested weirs. In situations where sloping roadways act as the weir outlet, the roadway is considered a sloped weir.

For weirs that are not flat, it is difficult to identify the height over the weir, h . There are two methods available to compute flow over a sloped weir. Each involves computing a discharge through subsections of the weir (refer to **Figure 8-9**).

Figure 8-9. Weir Flow Over Sloping Weir



8.6.3.1 Standard Weir Equation - Average Height

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

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

$$Q_1 = C_1 L_1 \left(\frac{0 + H_1}{2} \right)^{3/2}$$

$$Q_2 = C_2 L_2 \left(\frac{H_1 + H_2}{2} \right)^{3/2}$$

$$Q_3 = C_3 L_3 \left(\frac{H_2 + H_3}{2} \right)^{3/2}$$

$$Q_{\text{total}} = Q_1 + Q_2 + Q_3$$

8.6.3.2 Sloping Weir Equation

The sloping weir equation is defined as:

$$Q = \frac{2}{5}Z \left(CH_2^{5/2} - CH_1^{5/2} \right) \quad (8.6)$$

Where Z is the slope of the weir subsection (1V:ZH)

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

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

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

$$Q_{\text{total}} = Q_1 + Q_2 + Q_3$$

8.6.3.3 Discharge Coefficients

Charts are available that show 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 provide reasonable results. Computing a new C for each H is possible; however, the effect is not significant. If the weir has numerous obstructions (curb and gutter, guardrail, etc.), use judgment to reduce the C value. The reduction of the C value is proportional to the percent of obstructed weir crest length.

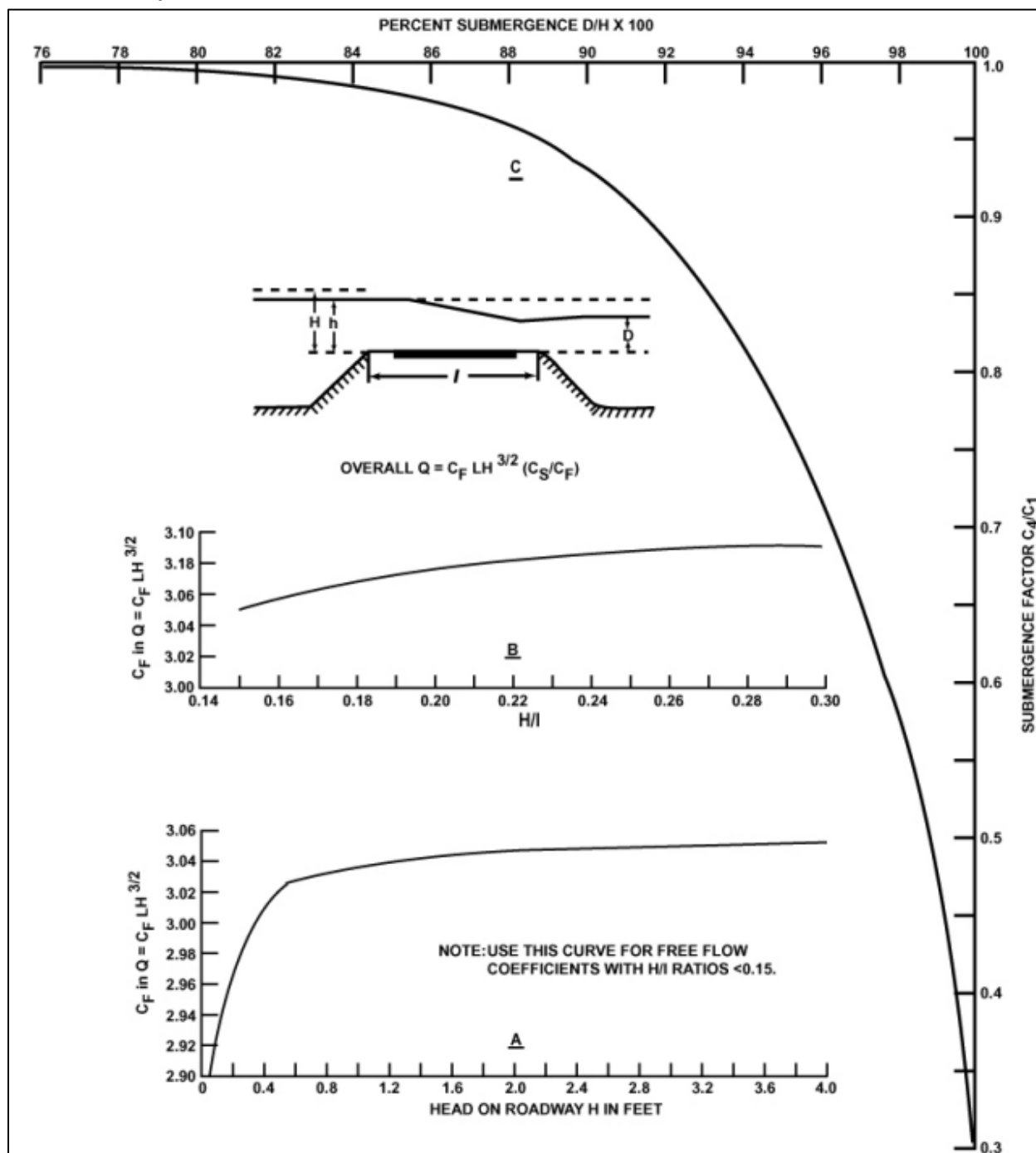
8.6.3.4 Weir Length

Exercise caution in determining the effective weir length of a roadway. It is important to eliminate portions of the roadway in which the areas upstream and downstream of a roadway are higher than the road, making that portion of the weir ineffective.

8.6.3.5 Submergence

Tailwater submergence of a weir will significantly reduce the weir efficiency. **Figure 8-10** shows a graph used to adjust the discharge due to tailwater submergence.

Figure 8-10. Discharge Coefficients for Flow over Roadway Embankments (FHWA HDS 1, 1978)



8.6.4 V-Notch Weirs

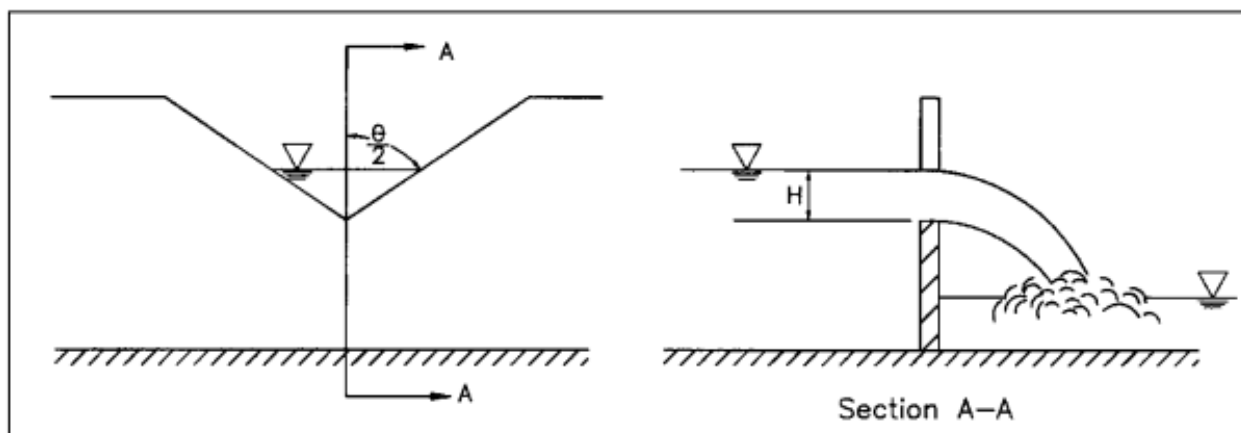
The discharge through a v-notch weir shown in **Figure 8-11** can be calculated using **Equation 8.7** (Brater and King, 1976).

$$Q = K_u \tan\left(\frac{\theta}{2}\right) H^{2.5} \quad (8.7)$$

Where:

- Q = Discharge, cfs
- K_u = 2.5 (English units)
- θ = Angle of v notch, degrees
- H = Head on apex of notch, ft

Figure 8-11. V-Notch Weir (Source HEC-22)



8.6.5 Orifices

Outlet losses are calculated using the orifice equation. For square-edged entrance conditions use **Equation 8.8**:

$$Q = 0.6A\sqrt{(2gH)} \quad (8.8)$$

Where:

- Q = Discharge, cfs
- A = Cross-section area of orifice, sft
- g = Acceleration due to gravity, 32.2 fps
- H = Head on orifice measured from the centroid of opening, ft

In cases where the tailwater is higher than the center of the opening, calculate the head as the difference in upstream and downstream water surface elevations.

8.7 SIZING METHODS AND ROUTING CALCULATIONS

Storage design is an iterative process that involves calculations that route the inflow hydrograph through the storage facility. Change basin and outlet geometry until the desired outflow hydrograph is achieved. The following storage and discharge data is needed for the routing procedure:

- Inflow hydrograph for all selected design storms. Develop inflow hydrographs to the storage facilities using NRCS WinTR-55. For more details, refer to the procedures presented in Chapter 3, Hydrology.
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures

8.7.1 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir as shown in **Figure 8-12**. The data for this type of curve are usually developed using a topographic map and the average-end area formula. Increments of stage used should provide a smooth curve without significant inflection points. The average-end area formula is expressed as:

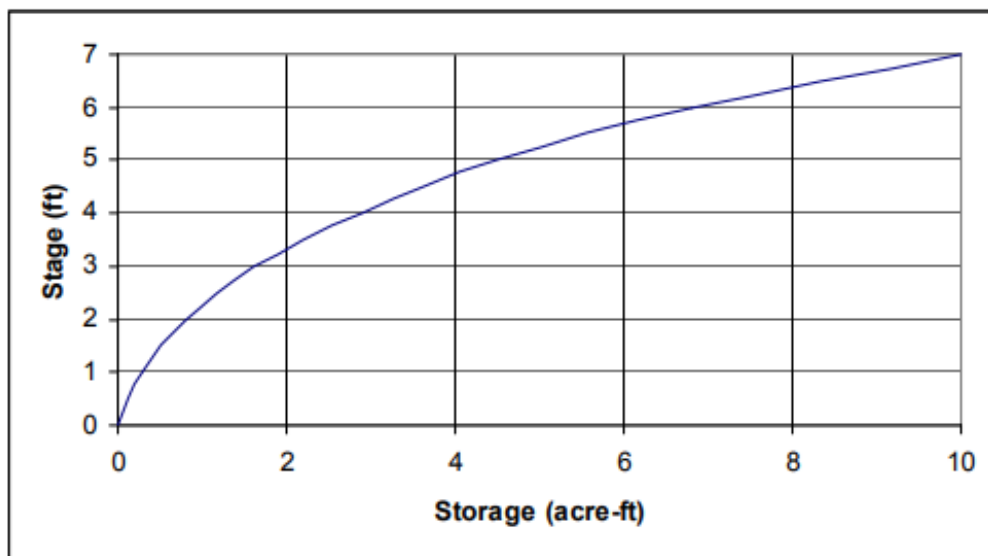
$$V_{1,2} = \left[\frac{(A_1 + A_2)}{2} \right] d$$

Where:

$V_{1,2}$ = Storage volume between elevations 1 and 2, cft

$A_{1,2}$ = Surface area at elevations 1 and 2 respectively, sft

d = Change in elevation between elevations 1 and 2, ft

Figure 8-12. Example Stage-Storage Curve

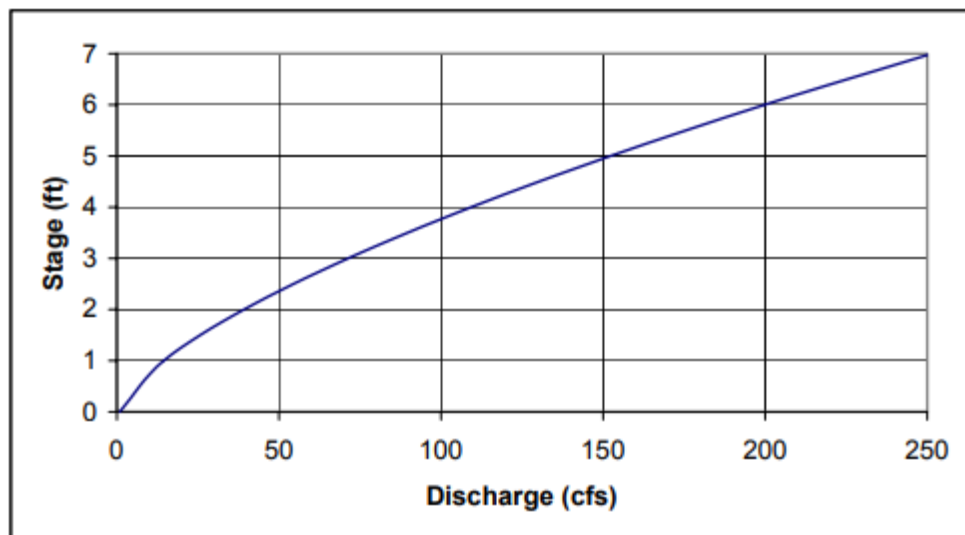
8.7.2 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility as shown in **Figure 8-13**. A typical storage facility has two spillways: principal and auxiliary.

Design the principal spillway with a capacity sufficient to convey the design flood before flow occurs in the auxiliary spillway. Sheet pile weirs with a low-flow orifice are preferred as the principal spillway.

Design the auxiliary spillway 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. Ensure that the tailwater does not influence the stage-discharge curve. The weir and orifice equations assume a free discharge. Do not include infiltration in stage-discharge curves for flood routing for detention, wet detention, or infiltration basins. Extend the stage-discharge curve to include the 0.2% AEP (500-year) design flow.

Figure 8-13. Example Stage-Discharge Curve**8.7.3 Routing Calculations (Modified Puls Method)**

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

Use **Equation 8.9**:

$$(I_1 + I_2) + \left(\frac{2S_1}{dt} - O_1 \right) = \left(\frac{2S_2}{dt} + O_2 \right) \quad (8.9)$$

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 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 8-5**. 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 **Equation 8.9**. 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 and 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 step.
4. Repeat Steps 1 to 3 until the entire outflow hydrograph, $O(t)$, is calculated.

Step 4 Graph the results of the 1% AEP (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% AEP (10-year) storm peak flow).

Because of the large number of computations necessary to perform routing, a computer is typically used to complete the procedure. The procedure lends itself well to spreadsheets or commercially available computer software such as NRSC WinTR-55 or FHWA Hydraulic Toolbox.

8.7.4 Design Procedure

A general procedure for the design of storage facilities is presented (assuming a 10% AEP (10-year), 24-hour storm existing conditions release rate).

- Step 1** Determine the 10%, 4%, and 1% AEP (10-, 25- and 100-year, respectively), 24-hour 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 volume of the pretreatment, if applicable.
- Step 4** Determine/calculate approximate volume of detention storage required to attenuate the post-development inflow hydrograph peak flow to the allowable outflow for each design storm. This is an iterative process as the allowable outflow and outlet structure configuration both influence the stage-storage-discharge curves.
- Step 5** 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 6** 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 7** Adjust the basin grading and outlet as necessary to adjust the design to function within the established parameters.
- Step 8** Design an auxiliary spillway for flows that exceed the 1% AEP (100-year), 24-hour flood flows. Check freeboard is 1.5 feet or more for the 1% AEP (100-year), 24-hour flood flow. Verify flows greater than the design storm up to the 0.2% AEP (500-year), 24-hour event can safely pass through the storage and outlet to the downstream system.
- Step 9** Finalize plan and design documentation.

A significant number of reservoir routing calculations may be needed to obtain the desired results.

8.8 EXAMPLES

8.8.1 Example: Developing a Stage-Discharge Curve

Design a detention basin to release storage through a sheet pile weir with a low flow orifice. Size the orifice and weir together to satisfy pre-developed flow rates for a range of storms. The sizing procedure is an iterative process. This example shows how to develop a stage-discharge curve for these type of outlet structures. The pre-developed peak flow rates for the 50%, 10%, 2%, 1% AEP (2-year, 10-year, 50-year, 100-year), 24-hour design storms are 17, 35, 55, and 64 cfs, respectively. The groundwater table is below the bottom of the basin. Assume a free discharge from the basin.

The top of the stop log portion of the outlet is the primary weir and is set at a depth of 4 feet from the basin bottom. The low flow orifice is located at the bottom of the basin. Initially, size the low flow orifice to release at the existing 50% AEP (2-year), 24-hour flow rate when the basin is approximately 3.5 feet full (0.5 feet below weir crest). Assume the initial orifice height is 0.5 feet.

Using the orifice equation, solve for area. H is measured from the centroid of the orifice.

$$Q = 0.6A\sqrt{(2gH)}$$

$$A = \frac{Q}{0.6\sqrt{(2gH)}}$$

$$A = \frac{17 \text{ cfs}}{0.6\sqrt{(2 * 32.2 \text{ ft/s}^2 * 3.25 \text{ feet})}}$$

$$A = 2.0 \text{ ft}^2$$

Using the orifice area calculated, the estimated dimension of the orifice is 0.5 feet high x 4 feet wide.

Next, size the primary weir to pass the 1% AEP (100-year), 24-hour existing flow (minus the flow through the orifice) without exceeding 1.5 feet of freeboard. Assume a head over the primary weir of 1.25 feet will satisfy freeboard requirements.

Calculate flow through the orifice when the depth is 5.25 feet:

$$Q_{orifice} = 0.6A\sqrt{(2gH)}$$

$$Q_{orifice} = 0.6 * 3.75 \text{ ft}^2 * \sqrt{(2 * 32.2 \text{ ft/s}^2 * 5.0 \text{ feet})}$$

$$Q_{orifice} = 21.1 \text{ cfs}$$

$$Q_{100} - Q_{orifice} = 64 \text{ cfs} - 21.1 \text{ cfs} = 42.9 \text{ cfs}$$

Using the weir equation, solve for length:

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

$$L = \frac{Q}{CH^{3/2}}$$

$$L = \frac{42.9 \text{ cfs}}{3.3 * (1.25 \text{ ft})^{3/2}}$$

$$L = 9.3 \text{ feet}$$

Assume a primary weir length of 10 feet.

The calculated orifice and weir sizes are shown in **Figure 8-14**. Use these dimensions and calculate the stage-discharge curve by finding the discharge from the outlet as the stage increases in 0.5-foot increments. From depths of 0 to 4 feet, flow will only be through the orifice. For depths greater than 4 feet, flow will be comprised of the orifice flow and primary weir flow.

The stage discharge data for this outlet is reflected in **Table 8-5**. The stage-discharge curve is shown in **Figure 8-15**. The inflection point in the curve, denoted by the dotted line, indicates the point when flow over the weir begins.

Figure 8-14. Sheet Pile Outlet Structure

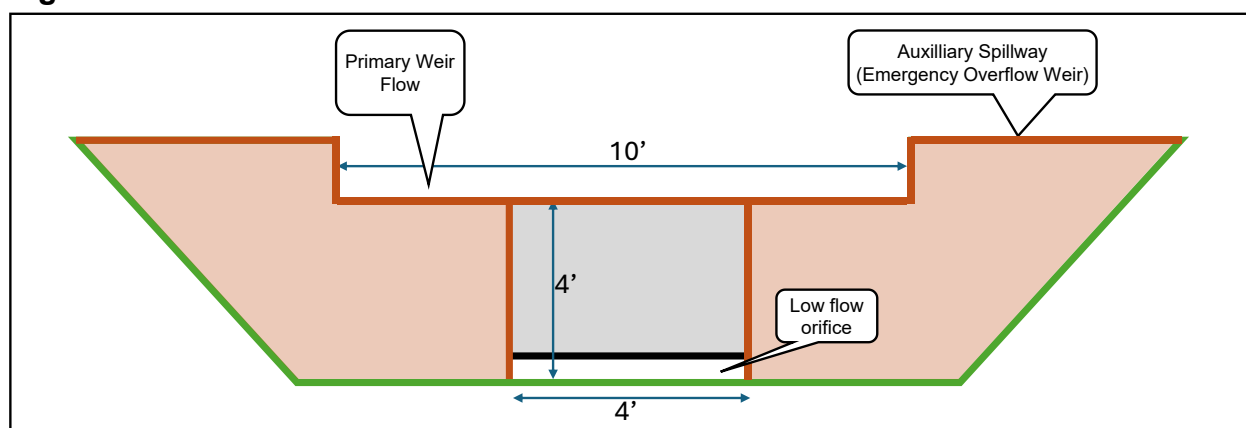
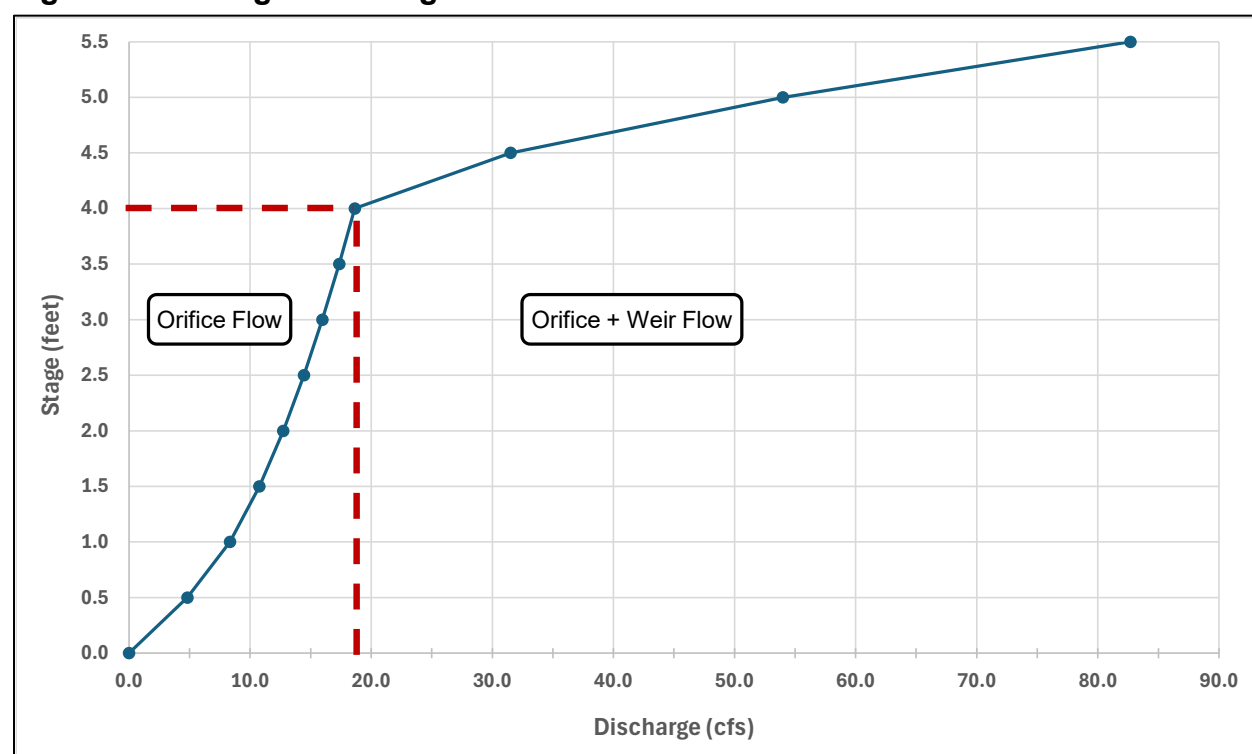


Table 8-5. Stage-Discharge Summary Table

Stage	Orifice Discharge (cfs)	Weir Discharge (cfs)	Total Discharge (cfs)
0.0	0.0	0.0	0.0
0.5	4.8	0.0	4.8
1.0	8.3	0.0	8.3
1.5	10.8	0.0	10.8
2.0	12.7	0.0	12.7
2.5	14.4	0.0	14.4
3.0	16.0	0.0	16.0
3.5	17.4	0.0	17.4
4.0	18.6	0.0	18.6
4.5	19.9	11.7	31.5
5.0	21.0	33.0	54.0
5.5	22.1	60.6	82.7

*When depths are less than 0.5 foot, inflow = outflow

Figure 8-15. Stage-Discharge Curve

8.8.2 Example: Detention Basin Design and Outlet Structure

In the previous example, an outlet was sized to accommodate a range of pre-developed design storms. In this example, using the outlet structure designed in the previous example, size a detention basin to accommodate post-developed flows. Use the Modified Puls method to route post-developed inflows. The routed flows should not exceed the pre-developed peak flow rates and the depth in the basin cannot exceed the freeboard depth of 5.25 feet. Pre-developed inflows for the 50%, 10%, 2%, 1% AEP (2-year, 10-year, 50-year, 100-year), 24-hour design storms and the maximum space available for a basin is given in the following steps.

Step 1 The following data was obtained describing the post-development inflow hydrographs to a proposed detention basin during the 50%, 10%, 2%, 1% AEP (2-year, 10-year, 50-year, 100-year), 24-hour design storms shown in **Table 8-6**. The pre- and post-developed peak flows for a range of AEP storms are shown in **Table 8-7**.

Table 8-6. Inflow Hydrograph Data

Time (min)	50% AEP (2-year)	10% AEP (10-year)	2% AEP (50-year)	1% AEP (100-year)
0	0	0	0	0
5	8	12	16	18
10	25	38	48	53
15	49	75	96	105
20	60	91	117	129
25	60	92	117	129
30	57	88	112	124
35	51	78	100	110
40	43	66	84	93
45	34	53	68	75
50	26	40	51	57
55	18	28	35	39
60	11	18	23	25
65	8	12	15	17
70	5	8	10	11
75	3	4	6	6
80	0	0	0	0
85	0	0	0	0

Table 8-7. Pre- and Post-Developed Peak Flow Rates

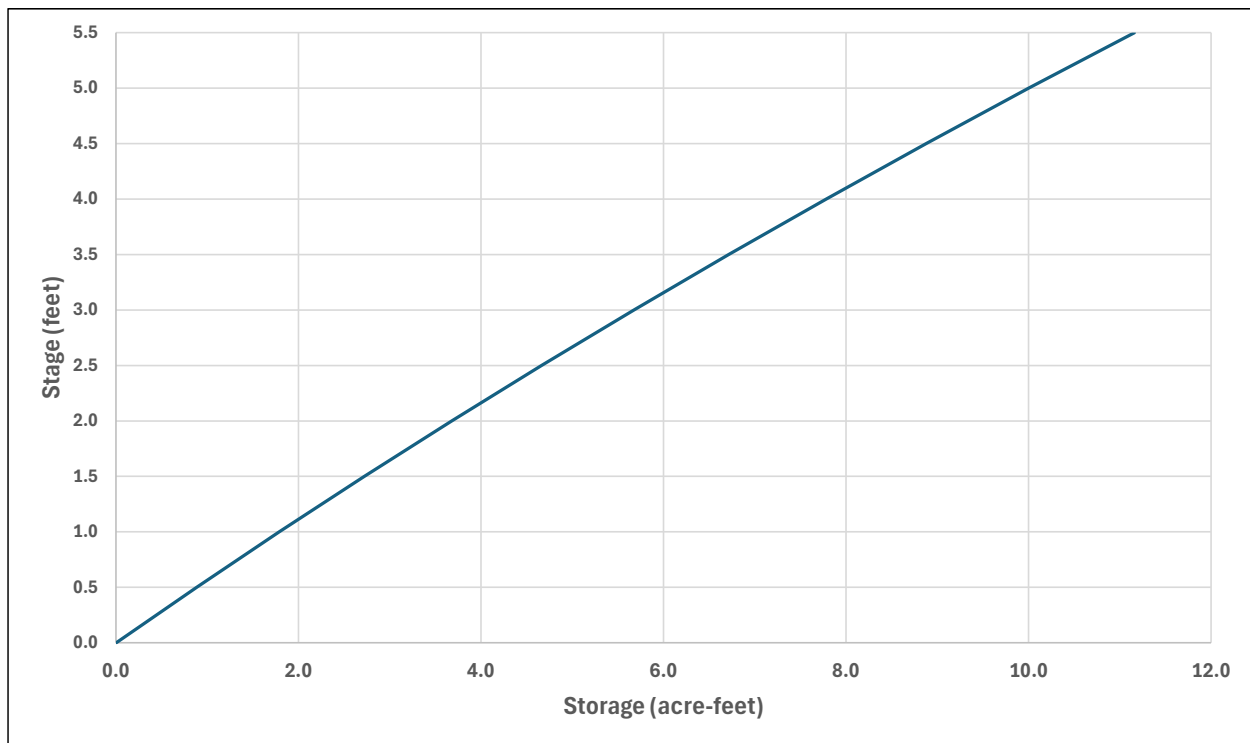
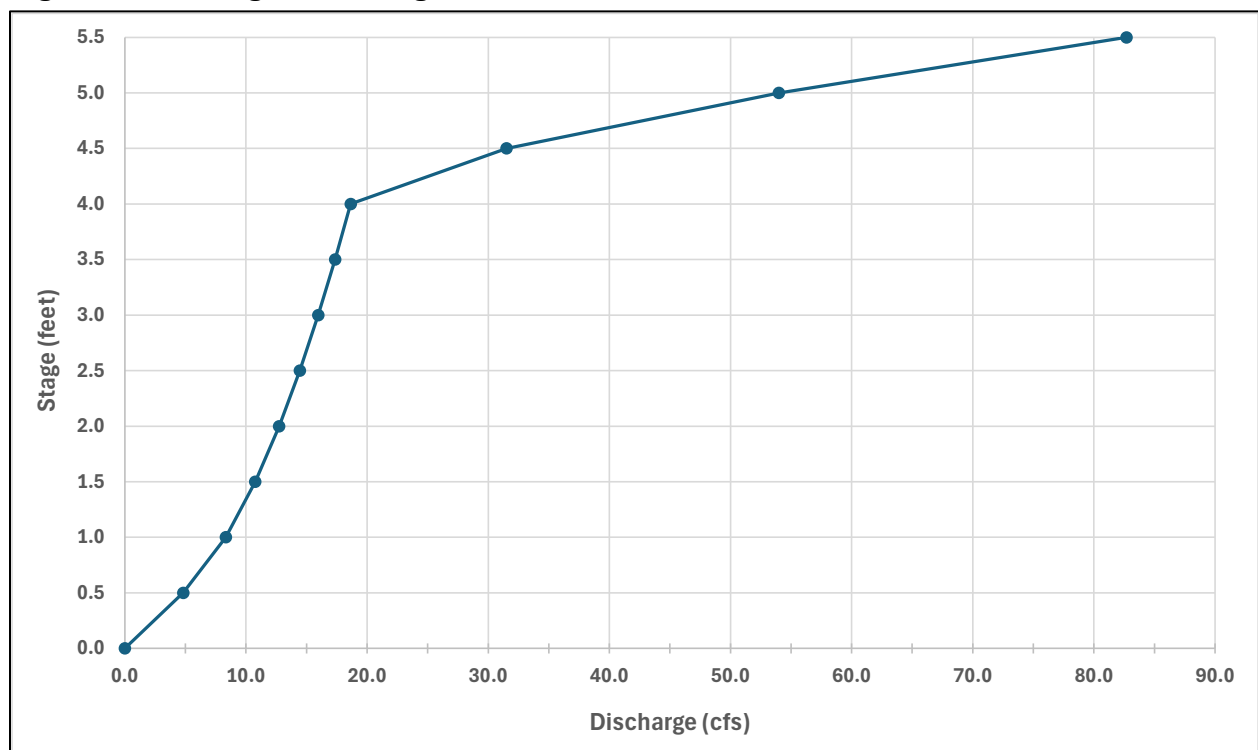
Design Storm	Pre-developed Peak Flow (cfs)	Post-Developed Peak Flow (cfs)
50% AEP (2-yr), 24-hr	17	60
10% AEP (10-yr), 24-hr	35	92
2% AEP (50-yr), 24-hr	55	117
1% AEP (100-yr), 24-hr	64	129

Step 2 The following stage-storage and stage-discharge characteristics were determined for the storage facility and summarized in **Table 8-8**. The basin will not be used for channel protection treatment. The stage-storage curve reflects the maximum storage available on site. The basin bottom geometry is 275 feet x 275 feet with 4H:1V side slopes. The depth cannot exceed 5.25 feet to satisfy 1.5 feet freeboard requirements for the 1% AEP (100-year), 24-hour storm. The outlet is a sheet pile weir with a low flow orifice designed in the previous example problem. The length of the primary weir is 10 feet with a weir crest 4 feet above the bottom of the basin. The rectangular orifice is 0.5 feet high x 4 ft wide at the bottom of the basin. The value for equation $2S/dt + O$ used for the routing procedure was calculated for each stage. The stage-storage curve is shown in **Figure 8-16** and the stage-discharge curve is shown in **Figure 8-17**.

Table 8-8. Storage Facility Characteristics

Stage	Storage (acre-ft)	Orifice Discharge (cfs)	Weir Discharge (cfs)	Total Discharge/ Outflow, O (cfs)	$\frac{2S}{dt} + O$ (cfs)
0.0	0.0	0.0	0.0	0.0	0.0
0.5	0.9	4.8	0.0	4.8	260.6
1.0	1.8	8.3	0.0	8.3	527.3
1.5	2.7	10.8	0.0	10.8	800.5
2.0	3.7	12.7	0.0	12.7	1080.9
2.5	4.7	14.4	0.0	14.4	1368.8
3.0	5.7	16.0	0.0	16.0	1664.4
3.5	6.7	17.4	0.0	17.4	1967.8
4.0	7.8	18.6	0.0	18.6	2279.2
4.5	8.9	19.9	11.7	31.5	2610.3
5.0	10.0	21.0	33.0	54.0	2959.4
5.5	11.2	22.1	60.6	82.7	3323.0

Note: $dt = 5$ minutes = 300 seconds

Figure 8-16. Stage-Storage Curve**Figure 8-17. Stage-Discharge Curve**

Step 3 A routing procedure table (**Table 8-9**) was developed using the Modified Puls routing procedure outlined in **Section 8.7.3**. The 1% AEP (100-year), 24-hour inflow hydrograph, stage-storage curve, and stage-discharge curves are used in the calculations.

Example calculations for time at 20 minutes are shown:

Using **Equation 8.9**,

$$(I_1 + I_2) + \left(\frac{2S_1}{dt} - O_1 \right) = \left(\frac{2S_2}{dt} + O_2 \right)$$

$$I_1 = 105 \text{ cfs (from } t = 15 \text{ min)}$$

$$I_2 = 129 \text{ cfs (from } t = 20 \text{ min)}$$

$$\left(\frac{2S_1}{dt} - O_1 \right) = 233.9 \text{ cfs (from } t = 15 \text{ min)}$$

$$\left(\frac{2S_2}{dt} + O_2 \right) = (I_1 + I_2) + \left(\frac{2S_1}{dt} - O_1 \right)$$

$$\left(\frac{2S_2}{dt} + O_2 \right) = (105 + 129) + 233.9 = 468.3 \text{ cfs}$$

Then from the value of 468.3 cfs for $(2S/dt + O)$, interpolate the corresponding discharge, O , and stage of the storage facility using the characteristics in **Table 8-9**.

Outflow/Discharge Interpolation

$$(527.3 - 468.3)/(527.3 - 260.6) = (8.3 - O)/(8.3 - 4.8)$$

$$O = 7.6 \text{ cfs}$$

Stage Interpolation

$$(8.3 - 7.6)/(8.3 - 4.8) = (1 - \text{Stage})/(1 - 0.5)$$

$$\text{Stage} = 0.9 \text{ feet}$$

Subtract $2O$ from $((2S_2)/dt + O_2)$ to yield $((2S_2)/dt - O_2)$

$$\frac{2S_2}{dt} - O_2 = 468.3 - 2(7.6) = 453.1 \text{ cfs}$$

Table 8-9. Routing Procedure Table for 1% AEP (100-year), 24-hour Storm

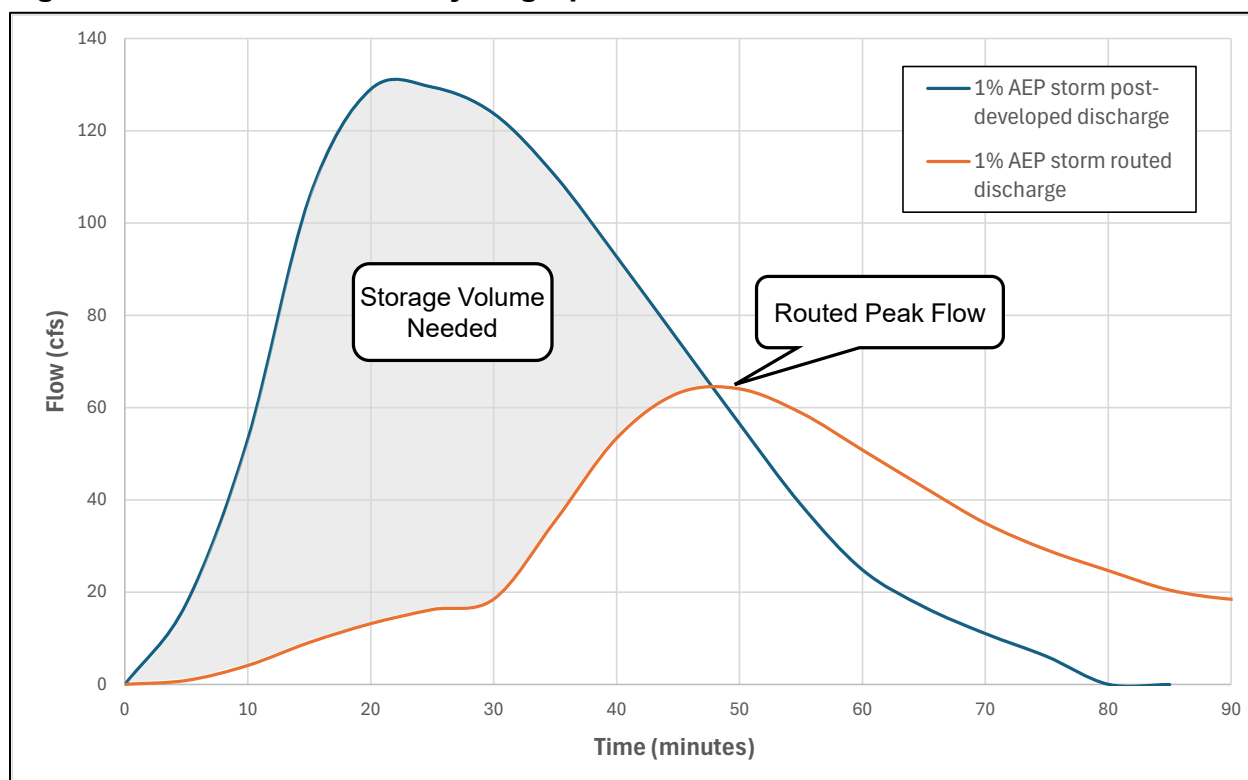
Time (min)	I (cfs)	$\frac{2S}{dt} - O$ (cfs)	$\frac{2S}{dt} + O$ (cfs)	O (cfs)	Stage (ft)
0	0	0.0	0.0	0.0	0.0
5	18	16.9	17.6	0.3	0.0
10	53	84.3	87.6	1.6	0.2
15	105	233.9	242.9	4.5	0.5
20	129	453.1	468.3	7.6	0.9
25	129	691.6	711.5	10.0	1.3
30	124	921.1	944.7	11.8	1.8
35	110	1128.7	1155.1	13.2	2.1
40	93	1303.2	1331.7	14.2	2.4
45	75	1440.5	1470.5	15.0	2.7
50	57	1540.7	1571.6	15.5	2.8
55	39	1604.5	1636.1	15.8	3.0
60	25	1636.3	1668.3	16.0	3.0
65	17	1646.0	1678.0	16.0	3.0
70	11	1641.8	1673.8	16.0	3.0
75	6	1627.1	1658.9	15.9	3.0
80	0	1601.6	1633.2	15.8	2.9
85	0	1570.3	1601.6	15.6	2.9

The routed peak flow is 16.0 cfs, which is well below the pre-developed peak discharge of 64 cfs. The peak stage is 3.0 feet, which is below the maximum allowable depth of 5.25 feet. The size of the basin or the outlet can be changed to meet the design requirements. In this example, the basin size is reduced. Through an iterative process, the basin footprint is adjusted until the requirements are met. The final footprint of the basin bottom is 170 feet x 165 feet. The updated routing procedure is summarized in **Table 8-10**. The routed peak flow of 64 cfs is equal to the pre-developed peak discharge. The maximum depth in the basin is 5.2 feet.

Table 8-10. Updated Routing Procedure Table

Time (min)	I (cfs)	$\frac{2S}{dt} - O$ (cfs)	$\frac{2S}{dt} + O$ (cfs)	O (cfs)	Stage (ft)
0	0	0.0	0.0	0.0	0.0
5	18	15.9	17.6	0.8	0.1
10	53	78.3	86.5	4.1	0.4
15	105	218.7	236.9	9.1	1.2
20	129	426.8	453.1	13.2	2.1
25	129	652.8	685.2	16.2	3.1
30	124	868.9	905.9	18.5	3.9
35	110	1032.0	1102.9	35.4	4.6
40	93	1128.2	1234.9	53.4	5.0
45	75	1169.2	1295.5	63.1	5.2
50	57	1172.2	1300.3	64.1	5.2
55	39	1149.9	1267.7	58.9	5.1
60	25	1112.1	1213.7	50.8	4.9
65	17	1068.4	1153.8	42.7	4.7
70	11	1026.4	1096.2	34.9	4.6
75	6	985.2	1043.5	29.1	4.4
80	0	942.0	991.3	24.7	4.2
85	0	901.2	942.1	20.4	4.1

Step 4 Plot the results of the routed 1% AEP (100-year), 24-hour storm and the post-developed 1% AEP (100-year), 24-hour storm inflow hydrographs as shown in **Figure 8-18**. Verify the routed peak outflow does not exceed the pre-developed peak discharge for a range of design flows as shown in **Table 8-11**.

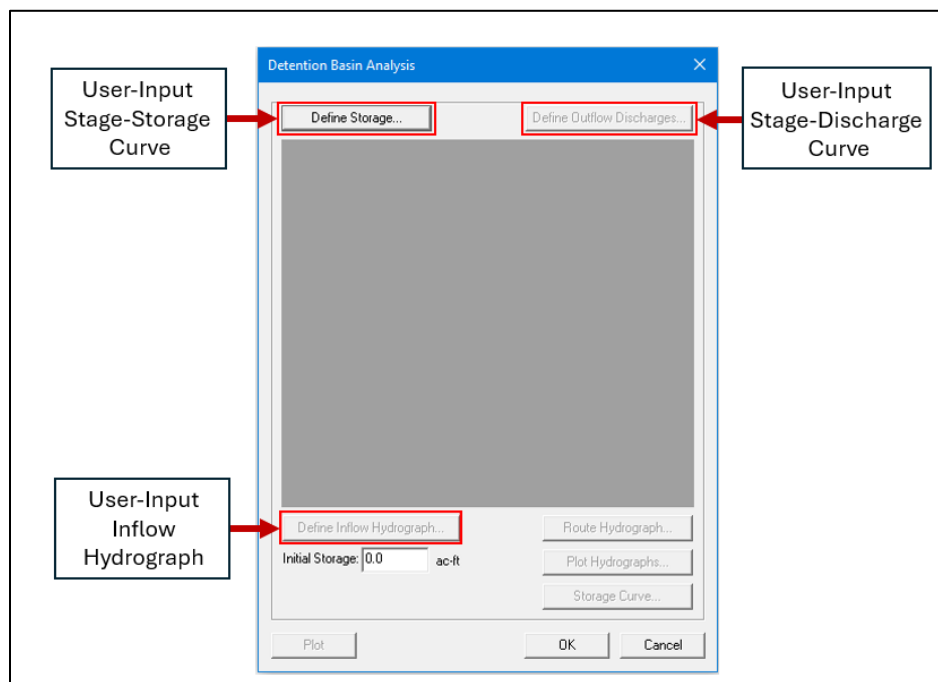
Figure 8-18. Plot of Routed Hydrograph**Table 8-11. Pre-Developed, Post-Developed, and Routed Peak Flow**

Design Storm	Pre-developed Peak Flow (cfs)	Post-Developed Peak Flow (cfs)	Routed Peak Flow (cfs)
50% AEP (2-yr), 24-hr	17	60	16
10% AEP (10-yr), 24-hr	35	92	28
2% AEP (50-yr), 24-hr	55	117	52
1% AEP (100-yr), 24-hr	64	129	64

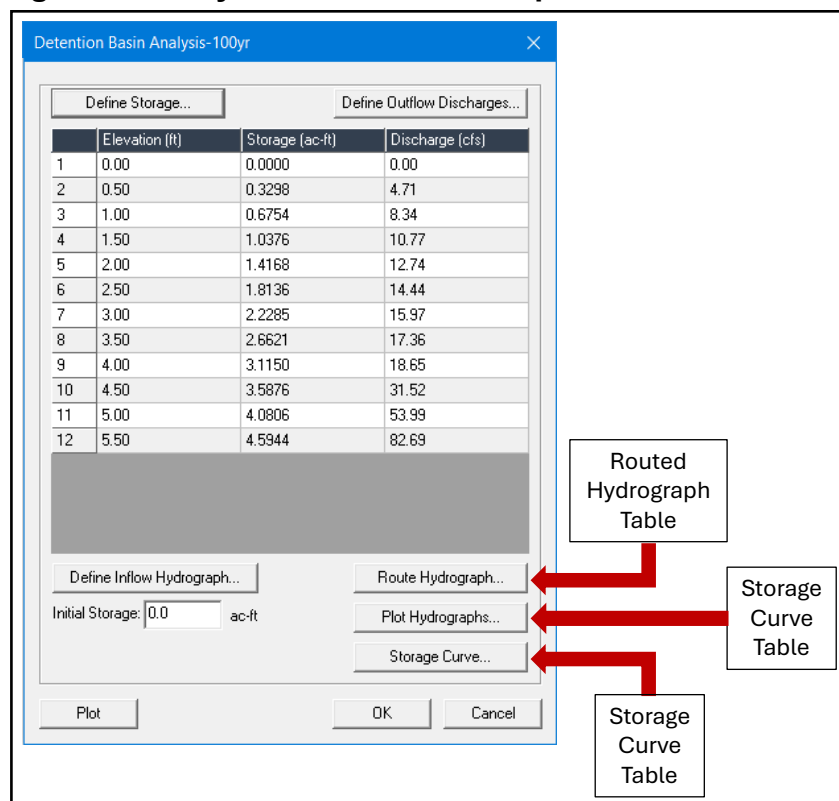
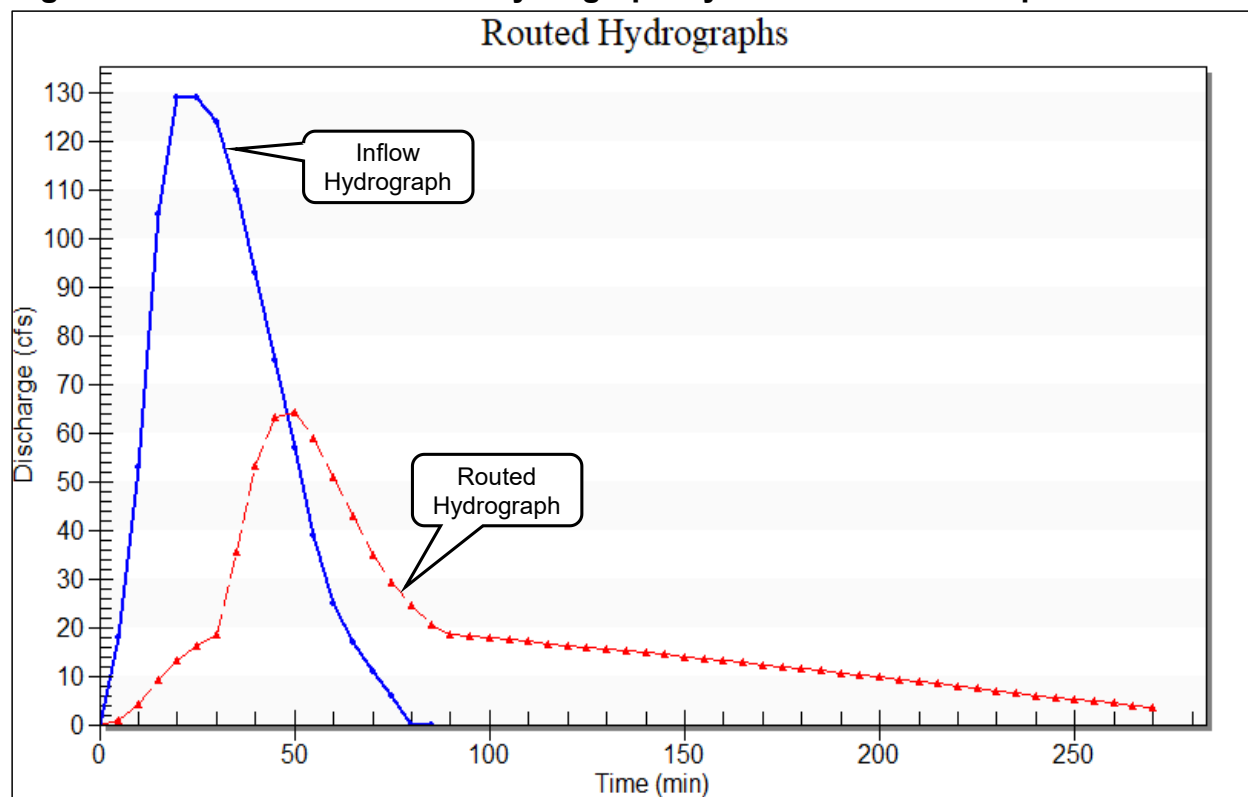
8.8.3 Example: Use Hydraulic Toolbox to Perform Hydrograph Routing

The hydrograph routing example shown in **Section 8.8.2** can also be solved using computer software FHWA Hydraulic Toolbox. The “New Detention Basin Analysis” calculator was used within the software to route the inflow hydrograph through the storage.

Step 1 Input the inflow hydrograph, stage-storage curve, and stage-discharge curve given in the example problem in **Section 8.8.2** into the calculator as shown in **Figure 8-19**.

Figure 8-19. Hydraulic Toolbox Inputs

Step 2 The results, as shown in **Figure 8-20**, can be viewed in three different formats: the routed hydrograph in table format, a graph of the inflow hydrograph and the routed hydrograph, and a storage curve. The graph of the inflow and routed hydrographs are shown in **Figure 8-21**.

Figure 8-20. Hydraulic Toolbox Outputs**Figure 8-21. Inflow and Routed Hydrograph Hydraulic Toolbox Output.**

8.9 MAINTENANCE

Maintain the storage volume at its design level and keep the outlet structure open and free from obstructions or clogging to ensure the storage facility functions properly. For detention, wet detention, and infiltration basin maintenance, follow the methods described in the MDOT Maintenance Activity Guide, Activity 1750, Retention, Infiltration or Detention Basin Maintenance.

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