TDOT DESIGN DIVISION

DRAINAGE MANUAL

CHAPTER VIII
STORMWATER STORAGE FACILITIES

CHAPTER 8 – STORMWATER STORAGE FACILITIES

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SECTION 8.01 – INTRODUCTION

This chapter will present the analysis methods and design requirements to safely incorporate stormwater storage facilities into highway projects. Stormwater storage may be used to reduce the impact of stormwater runoff from highway facilities on the environment and adjacent property owners. Stormwater storage may be required in urban and urbanizing areas due to the greater likelihood of impact to adjacent property owners. For a watershed with no defined outfall or an inadequate downstream conveyance system, the total volume of runoff is critical, and storage facilities can be used to detain the volume increases and control the discharge rate.

Highway construction will usually result in an increase in the impervious area and a more efficient stormwater conveyance system over that of the pre-developed condition. These improvements typically result in an increase in the runoff volumes and peak flow rates from the highway project. The runoff from the construction phase and completed highway project may also have increased pollutant loadings from the pavement, road salt, oils and grease from vehicles, heavy metals, and other pollutants. The purpose of a stormwater storage basin will be to reduce or minimize these impacts by providing stormwater quantity control, and to enhance water quality.

Watersheds downstream of highway projects may be adversely impacted by these increased flow rates, volumes, and pollutant loadings. Typical downstream impacts are water quality degradation, stream channel erosion, and localized flooding. To minimize these impacts on adjacent and downstream property owners, TDOT roadway projects may include mitigation measures to limit peak flow rates from the highway project to the pre-developed condition. Typical stormwater storage facilities to mitigate these adverse impacts are detention basins, extended-detention basins, retention basins, conveyance system storage, underground storage, or some combination of these facilities.

SECTION 8.02 – Documentation Procedures

The designer will be responsible to document the hydraulic analysis performed for the design of all roadway detention or retention facilities. The documentation should be stored in a project folder. The documentation should also include a discussion of any unusual features or conditions within the project. This discussion should include all of the assumptions and design decisions made by the designer to accommodate these special conditions. Any assumption made during the design of a stormwater storage facility should be clearly and concisely documented in the project file. If situations are such that the facility is designed by other than normal or generally accepted engineering procedures, or if the design of the facility is governed by factors other than hydrologic or hydraulic factors, a summary detailing the design basis should appear in the documentation file. Additionally, discuss any special considerations, parameters, or assumptions which may have influenced the design of the facility.

In general, the documentation should be sufficient to answer any reasonable question that may be raised in the future regarding the reservoir. The items listed in the following paragraphs should be in the project documentation file. The intent is not to limit the information provided, but instead provide a guide to the minimum documentation requirements consistent with the design procedures of this Manual.

The designer should provide adequate information on all hand calculation sheets to accurately identify the project design. In general, the information to be provided in the project file should include, but is not limited to, a project descriptor, project location, a description of the type of calculation, project specific location (station and offset), project designer, and the date of the computations. All hand calculations shall be prepared and assembled in a neat, legible and orderly manner.

The required documentation for roadside stormwater storage facilities should contain all necessary information to justify the design and operation of the facility. The following documentation should be included in the project folder:

For EACH stormwater facility,

- Description of the pre-project characteristics of the watershed, such as drainage boundary, outfalls, conveyance methods, pervious and impervious area, and peak run-off rates
- Description of the post-project characteristics of the watershed, similar to the pre-project description
- Hydrology of the facility’s contributing watershed. See Chapter 4 for details.
- Computations of the pre- and post-project runoff rates for the design storms
- Notes, computations, and sketches applicable to basin grading, principal spillway design, and emergency spillway design
- Soil and groundwater information, if available
- Storm storage and water quality volume computations, outlet hydraulic computations, receiving channel capacity analysis, and documentation of all storm routings including inflow and outflow hydrographs for all storm events
- A description of the operation of the facility and any special maintenance requirements deemed necessary during design.

Much of the information listed above may be generated with acceptable computer models. When computerized computations are employed, the data files should be clearly labeled with the project description, project station, purpose of the computation (i.e., existing or proposed analysis), date and initials of the designer. Any important assumptions made in developing the input data should also be documented. When possible, this information should be included with the data input into the program. Otherwise, it will be necessary to label output files. Hard copies of output reports documenting the hydraulic performance of the proposed storage facility should be included in the project folder, along with a floppy disk containing all of the relevant data files.

8.02.1 MAINTENANCE NOTE ON PLANS

Stormwater storage facilities require regular inspection and maintenance through their lives in order to preserve their original design characteristics. The following note should be added to each proposed layout plan sheet which contains a facility to store or otherwise mitigate stormwater for the project:

> **Post-Construction Maintenance Note:** The stormwater storage facility has been designed with a requirement for regular inspection and maintenance to ensure that its original design capacity and outflow characteristics will be maintained for the life of the facility. Accumulated sediment and debris should be removed as needed to preserve the original design stormwater storage volume and outflow characteristics for the life of the facility. For detention basins, the floor of the facility should be maintained so that stormwater properly exits and the basin is dry after each storm event.

SECTION 8.03 –TYPES OF STORAGE FACILITIES

8.03.1 INTRODUCTION

The types of stormwater storage facilities that may be used by the designer are described in the following paragraphs. Stormwater storage facilities can be classified by function as either, detention, extended detention, or retention. Each of these stormwater storage facilities has advantages and disadvantages that should be weighed by the designer to determine the optimal configuration for the highway project. The type of facility selected (detention, extended-detention, retention) and the relationship between its design components (inflow, storage volume, outflow) will dictate the size and extent of the basin and the hydraulic design of the facility. The facilities may be used in combination to meet design requirements and project site constraints.

Stormwater storage facilities should only be considered where they are shown to be beneficial by hydrologic, hydraulic, and cost analysis. For most situations, dry detention basins will be used to detain the increase in peak runoff due to a roadway project. Where water quality is an issue, extended detention basins should be considered as an acceptable treatment facility. Due to maintenance issues, safety hazards, and financial reasons, the use of retention basins should only be under special circumstances, such as environmental or community (aesthetic) constraints, and must be approved by the Design Manager.

Sections 8.03.2 through 8.03.6 provide an overview and discussion of the various types of storage facilities available for use on roadway design projects.

8.03.2 DETENTION BASINS

A basic dry detention basin is a surface reservoir that is used to detain stormwater runoff and release that water at a maximum rate that is equal to or less than the pre-developed runoff rate. A detention basin will reduce the peak discharge resulting from watershed development, and detain the runoff for a short period of time, prior to its controlled release. Additionally, a detention basin will be used to control or minimize potential damage or overloading of downstream conveyance systems by controlling discharge from flood events. All of the stormwater runoff will be released over a period time based on the design release rates described in Section 8.04.2.

Most detention facilities on TDOT roadway projects should be designed to have a dry bottom between rainfall events. The outflow from the detention basins can be controlled using a weir, orifice, culvert, staged riser (standpipe), or some combination of these structures. An emergency overflow structure will be required as part of the storage facility design to prevent high water elevations from adversely impacting the highway facility, adjacent property, or overtopping the embankment. Figure 8-1 represents a schematic of a basic dry detention facility layout.

Conventional dry bottom detention basins do not have water quality treatment capabilities, therefore are not recommended for use on projects where governing water quality requirements must be considered. However, they can be used in such cases if they are designed to be in combination with a pretreatment device such as a pre-manufactured stormwater quality manhole.

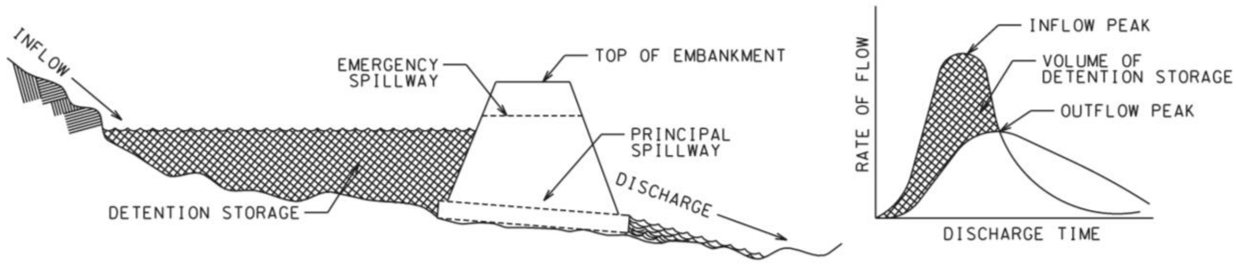


Figure 8-1
Typical Dry Detention Facility
Reference: Adapted from Nashville SWMM (1988)

Advantages to a detention basin are as follows:

- Most cost effective facility for controlling stormwater quantity
- Requires less storage volume than other stormwater reservoirs
- Completely drains shortly after storm event
- Remains dry between rain events

Disadvantages to a detention basin are as follows:

- Additional right-of-way may be required for the project
- Short term open water adjacent to the highway may be a safety hazard to errant vehicles and pedestrians
- Need for periodic maintenance to remove accumulated sediment and manage vegetation so that facility will function as designed

The "typical" plotted hydrographs of Figure 8-1 show that the peak outflow from a detention facility should always intersect the recession limb of the inflow hydrograph.

8.03.3 EXTENDED DETENTION BASINS

An extended detention basin is a retention facility which provides both water quantity and quality features in its design by slowly releasing the storm discharge over a period of days through the use of a drawdown device. Extended detention facilities are typically dry between storm events, provided the events occur at an interval greater than the drawdown time of the basin.

Extended detention facilities improve water quality by providing for sediment and other pollutants to settle out over time. The facility should be constructed with the primary outlet device of the outlet structure located at an elevation above the elevation necessary to contain and treat the "first flush" volume of runoff. The first flush volume is described in Section 8.04.3.2 of this Manual.

Water quality storage is provided below the primary outlet device of an extended detention basin. A perforated dewatering structure or one or more "bleed-down" orifices located at the lowest point of the facility will provide the necessary means for the basin to completely drain after a storm event. Draw down time of the water quality “first-flush” storage volume (below the primary outlet) for an extended detention facility should be between 24 and 48 hours (see Section 8.04.3.2). A concept diagram of an extended detention basin is provided in Figure 8-2. For an example of a properly constructed and operational extended detention facility, see Figure 8A-1 of the Appendix.

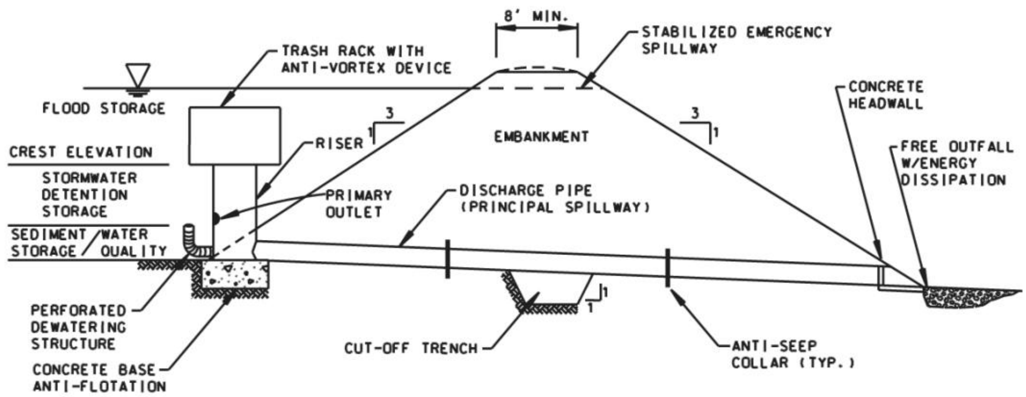


Figure 8-2
Concept Diagram of an Extended-Detention Facility

Some of the advantages of an extended detention basin include:

- Provides water quality improvements through sedimentation
- Requires less storage volume than retention reservoirs
- Nutrients removed by algae and rooted aquatic plants

The disadvantages to using an extended detention basin may include:

- Total storage volume required is greater than with a dry detention basin
- Maintenance costs may be greater than with a detention basin
- Short term pool adjacent to the highway may be a safety hazard
- Additional right-of-way may be required

8.03.4 RETENTION BASINS

A retention basin is a surface reservoir that will retain all or a portion of the stormwater runoff from the proposed highway facility. A retention basin will also treat incoming stormwater runoff, and will provide significant water quality improvements by reducing the eventual discharge of sediment, metals, floatable debris, and vehicle fluids. An example of a permanent pool retention basin is provided in Figure 8A-2 of the Appendix.

A retention basin consists of a permanent storage pool and an overlying zone of temporary storage to accommodate runoff from design storm events. The stormwater retained in the permanent pool of a retention basin will be released from the facility through groundwater infiltration or evaporation. The portion of the stormwater not retained in the reservoir will be detained and released at the design release rate similar to a detention reservoir. The live-pool of a retention basin will provide the necessary flood control volume required to attenuate the peak runoff rates equal to the pre-developed rate. An emergency overflow structure will be required to prevent high water elevations from adversely impacting the highway facility and adjacent property.

A retention basin will have a wet bottom or permanent pool. The bottom of a retention reservoir may become dry during an extended period of reduced precipitation due to groundwater infiltration and evaporation.

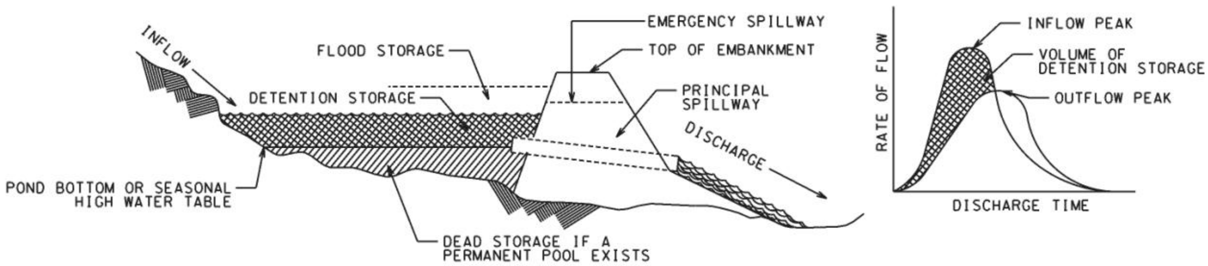


FIGURE 8-3

Typical Stormwater Retention Facility
Reference: Adapted from Nashville SWMM (1988)

The permanent pool of a retention basin allows for increased settling of sediment and pollutants. Runoff from each storm event is detained and treated in the retained pool through gravitational settling and biological uptake of nutrients. A permanent pool may also provide some level of protection from re-suspension of previously deposited sediment. The volume of the permanent pool shall be designed and configured accordingly to allow for proper settlement of pollutants. Specific design requirements for permanent pool facilities are provided in Section 8.04.9.

Advantages of using a retention basin include:

- Water quality improvements are greater than those with other stormwater facilities
- Moderate to high pollutant removal capacity
- Nutrients removed by algae and rooted aquatic plants
- Heavy metals removed during sedimentation
- Generally accepted by community as an amenity if properly designed

Some of the disadvantages of a retention basin may include:

- Total storage volume required is greater than with a detention basin
- Potential downstream thermal impacts due to warming of permanent pool
- Permanent pool adjacent to the highway may be a safety hazard to errant vehicles or pedestrians, especially in urban areas
- Additional right-of-way or maintenance may be required
- Require a reliable water/groundwater source to maintain desired permanent pool
- May promote increased insect populations and nuisance odors

8.03.5 CONVEYANCE SYSTEM STORAGE

Conveyance system storage is stormwater storage incorporated into the design of the storm sewer system or roadside ditches. The linear nature of highway facilities may increase the practicality of utilizing a linear detention facility when the use of a surface basin would not be practical or feasible. Storm sewers or roadside ditches can be oversized to accommodate the required storage volume and design release rates. The control structure typically will consist of an orifice, weir, culvert, or a combination of these structures.

Linear stormwater storage facilities may be classified as either a detention or retention facility depending if a portion of the water is retained for an extended period and discharged by infiltration. For most cases, a conveyance system storage facility should be designed as a detention system to control increases in peak runoff rates. Designing the system to function as a retention facility should be avoided unless no other practical solution exists.

Conveyance system storage is also described as being either in-series or parallel to the existing or proposed storm sewer system. A parallel system, as depicted in Figure 8-4, typically consists of an existing or proposed conveyance system, designed or retrofitted with an overflow device to carry excess storm runoff to a “parallel” pipe or vault used as the storage device. This storm discharge is then released from this device, back into the storm sewer system at a controlled rate. The parallel storage device must be designed with an emergency overflow device to pass the anticipated flows should the control device fail completely.

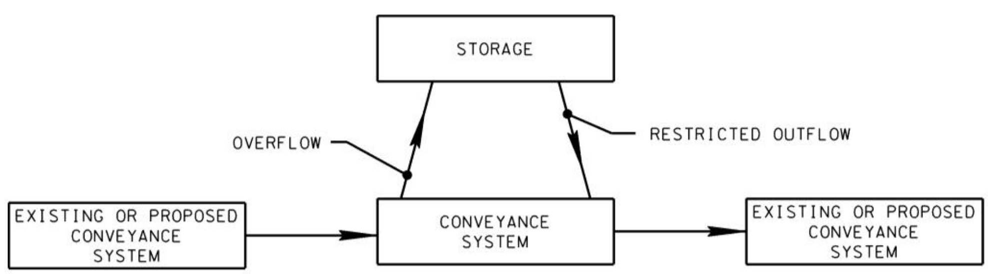


Figure 8-4
Storage Parallel to Conveyance System

The conveyance system in series consists of designing a portion of the existing or proposed stormwater system with an in-line, oversized pipe or vault. The outflow from the oversized storage area is controlled with an orifice and weir device and should be equipped with an emergency overflow device. The stormwater discharge is not diverted from the primary conveyance system; rather, it is stored and released within the system. Proper hydraulic analysis of the entire storm sewer system and maintenance considerations should be paramount during the design process for a facility of this type. A diagram depicting conveyance system storage in series is shown in Figure 8-5.

Figure 8-5
Storage In-Series with Conveyance System

Advantages of conveyance system storage are as follows:

- Additional right-of-way is typically not required
- Suitable for installation in existing and newly developing urban areas
- Used primarily for water quantity control
- Minimal hazard associated with closed, oversized storm sewer system

Disadvantages of conveyance system storage are as follows:

- Typically more expensive than open reservoirs
- Limited water quality improvement
- Control structure blockages could adversely impact the highway facility
- Maintenance access and performance typically more difficult
- Short term roadside ditch storage may require protective treatment

Where it is not feasible to use either an open reservoir or oversized storm sewer system for highway detention, the designer may consider utilizing a roadside ditch for storage and peak runoff control. The designer shall review and evaluate the potential for the ponded water to become a roadside hazard and should evaluate the safety aspects and related cost of protective devices for both vehicular and pedestrian traffic. The designer should also consider the impacts of the short term standing water, including freeboard, to pavement, possible saturation of subgrade, and flooding of adjacent property.

Where a restrictive channel section will be used as the outlet control structure for the storage facility, the channel design procedures presented in Chapter 5 should be followed. If an open channel section is used at the control section, the section should be constructed of concrete or other nonerodible material. An earth channel section will change over time due to erosion, settlement, and vegetation characteristics. These changes could result in the storage facility functioning incorrectly.

8.03.6 UNDERGROUND STORAGE

Underground storage may be classified as either detention or retention storage depending on whether a portion of the stormwater runoff is retained for an extended period or ground water infiltration is used to discharge a portion of the volume. Underground storage consists of pipes, vaults, or other storage structures. The primary difference between underground storage facility and conveyance system storage is that underground systems are connected to the storm sewer system with a control structure, but do not serve as part of the conveyance system.

Underground detention tanks should only be considered and used in extremely high density urban environments where either space is not available, or land costs become prohibitive for a larger, more traditional above ground facility. Due to high construction and maintenance costs typically associated with underground detention systems, these systems are used when space is limited and there are no other practical alternative to satisfy peak flow reduction objectives. In the suburban environment, right-of-way costs for land may also be high enough that these systems become a feasible alternative.

While underground detention systems allow for stormwater facilities to be located in urbanized areas while minimizing the need for valuable land, there are drawbacks to the facilities which inhibit its effectiveness as well as its practicability. Such inhibiting factors, depending on the design, may include:

- Underground stormwater detention tanks with the required storage volume are typically expensive to construct. The cost of building underground structures is usually prohibitive when compared to open detention basins.
- The typical outflow control for this facility, while simple, often ceases to function due to improper maintenance or may never function as theoretically designed.
- The rated efficiencies for most conventional underground facilities are so low that the Best Management Practice (BMP) cannot compensate for the increased pollution loads resulting from intense development.
- Maintenance of underground stormwater detention tanks is essential if they are to serve as a BMP. It is very difficult to inspect underground structures, particularly if entering the structure qualifies as confined space entry, which is controlled by OSHA safety regulations.
- Stormwater runoff quality is not substantially improved or enhanced by underground detention. Underground structures do not allow grass or other vegetation to absorb nutrients, minerals or pollutants from stormwater runoff. Underground structures do not take advantage of natural stormwater infiltration into the ground. Little, if any, significant water quality improvement should be expected when utilizing underground detention facilities.

These drawbacks often make development and/or redevelopment extremely difficult in intense urban areas and it is important to recognize the economic importance of these areas to the locality. At the same time, some of the heaviest transporters of non-point source pollutant loads on a land area basis are the intense urban areas. Since these areas are served by existing stormwater conveyance systems, a well designed underground detention vault may provide an alternative to conventional above ground facilities.

SECTION 8.04 – GUIDELINES and CRITERIA

8.04.1 INTRODUCTION

This section will present the design criteria for stormwater storage facilities. A detailed description of the design procedures for storage facilities will be presented in Section 8.05. In addition to the design criteria and parameters presented within this section, any stormwater facility embankment classified as a dam in Tennessee shall meet the established requirements of the Tennessee Safe Dams Act of 1973, including all subsequent amendments. The definition of a dam in Tennessee, according to the Safe Dams Act, is available in Section 8.04.5.1.1 of this document. The requirements of the Tennessee Safe Dams Act are available for review from the Tennessee Department of Environment and Conservation, and can be found at http://www.state.tn.us/environment/dws/safedams.shtml.

The need for permanent stormwater storage should be considered for any project which affects an area of one or more acres. Each stormwater outfall which discharges directly into a stream or other natural water resource with a drainage area of one acre or greater may have a written evaluation assessing whether the roadway project would result in increased runoff at that point, the potential impacts of the post-project release rates, and whether stormwater storage should be provided. The design storm events and factors to be considered in these evaluations are indicated in Section 8.04.2.1.

Although stormwater storage should be considered at any project outfall point where the post-project peak discharge exceeds the pre-project peak discharge, not all such sites will require stormwater storage. Storage will not be required at sites where the post-project peak discharge will not have adverse impacts on any of the factors described in Section 8.04.2.1. The Design Manager will determine whether stormwater storage is required at these sites.

In response to the NPDES Phase II program, many communities in Tennessee have instituted detention and stormwater quality requirements. Most TDOT projects are linear in nature and could potentially be located within a number of different jurisdictions. Variations in local stormwater management requirements could result in confusion and inconsistent design through the project. Thus, TDOT is not obligated by the local stormwater ordinances of any county, city or other jurisdiction in which a project may be located. The Department has developed its own stormwater standards which should be applied to all TDOT projects regardless of location. These stormwater standards should be communicated to each of the jurisdictions in which the project is located.

Where a project impacts an existing detention or retention facility, the existing facility should be modified to preserve its original stormwater storage capacity and operating characteristics. Stormwater from a TDOT roadway project should not be discharged into basins owned by others, including master development detention facilities, unless site conditions do not allow for another option. Where site conditions necessitate that stormwater from the project be discharged into an existing detention basin, it should be modified to provide any additional storage which might be needed to accommodate the project without changing its operating characteristics.

Proposed stormwater basins should be designed to attenuate discharge only from the roadway project. To the greatest extent possible, the Designer should attempt to bypass off-site “run-on” flows around or away from TDOT proposed detention facilities. Where this is not possible, the basin should be designed to allow off-site flows to pass through.

8.04.2 HYDROLOGIC AND HYDRAULIC CONSIDERATIONS

8.04.2.1 EVALUATION CRITERIA

The decision of whether stormwater detention would be required for a specific outfall location should be based on a full hydrologic and hydraulic evaluation of the site. An evaluation report should be developed for each project stormwater outfall which meets all of the following criteria:

- the outfall is located within a project which affects an area of one or more acres;
- the drainage area at the outfall is one acre or more
- 10% or more of the area draining to the outfall in the post-project condition is impervious surface; and
- the outfall discharges directly into a stream or other natural water resource

The hydrologic and hydraulic evaluation of each outfall location should be documented in a report. This report should include computations of the percent of impervious area within the drainage area of each outfall for both the pre-project and post-project conditions. This report should make a recommendation on the need for stormwater mitigation based on the criteria presented in the following discussion.

Meeting pre-developed peak runoff rates is not always the sole intent or design goal of stormwater mitigation. At some sites, it may be necessary to select a lower design release rate for various other reasons. Specific factors which should be considered in determining the extent of stormwater mitigation for a specific outfall are listed as follows:

- Sites where the project impacts an existing detention facility: See Section 8.04.1 for more information.

- Increased post-project runoff: Stormwater detention should be considered at any outfall where the post-project peak discharge for the 10-year design storm has been increased by 10% or more as compared to the pre-project condition. However, any increase in peak discharge should not exceed the downstream conveyance capacity, as described in the following paragraph.

- The capacity of the downstream conveyance system: Where the capacity of the downstream conveyance system is less than the existing pre-developed discharge for the 10-year storm event, then the maximum allowable release rate for the facility should be the downstream channel capacity, not the pre-developed peak runoff rate. The downstream capacity should be determined by computing the discharge at which the resulting water surface elevation is likely to cause downstream property damage.

- The potential for erosion in the receiving stream: A lower allowable release rate should be considered at sites where the Ecology Report has determined that significant portions of the downstream channel are degraded, eroding or unstable.

- Water quality concerns in the receiving stream: At some sites, it may be necessary to provide stormwater storage in order to prevent pollutants in on-site runoff from being released into the receiving waters.

- Drainage basins which drain to sinkholes: The evaluation of such sites should include consideration of the capacity of the sinkhole to accommodate increases in flow rates or total flow volume. Where a sinkhole is found to have limited capacity for increased flow volumes, it may be necessary to divert runoff from a portion of the existing drainage basin into a stormwater storage facility for another area.

- Outfall locations where the receiving stream is in or leads to an environmentally sensitive area: Environmental concerns which may necessitate the use of stormwater mitigation measures include public or private water supplies, high-quality natural areas, streams listed as being impaired, wetlands, the presence of endangered species, or other environmentally sensitive factors. These factors are typically identified in the Ecology Report provided by the Environmental Division.

8.04.2.2 RELEASE RATE AND TIMING

The maximum release rate from a TDOT stormwater storage facility will be controlled to approximate the pre-developed peak discharge rates for the 2-year/24-hour and 10-year/24-hour design storms. The facility release rate must be equal to or less than the pre-developed rate for the 2-year and 10-year design storms. Typically, a multi-stage outlet control structure (see Section 8.05.8) will be required to satisfy this requirement for both the 2-year and 10-year design storms. It can be assumed that if the release rate for both the 2-year and 10-year storms are controlled, then intermediate storms such as the 5-year storm are also adequately controlled.

Additionally, the facility should be designed with an emergency spillway capable of passing the 100-year/24-hour post-developed peak discharge without overtopping the facility. Design of the emergency spillway is presented in Section 8.04.7 of this chapter.

The timing of peak discharges from a stormwater facility and its effect on the natural drainage characteristics of the overall watershed should be considered by the designer. A detention facility constructed on a lower tributary to a larger watershed will reduce the peak discharge and extend the duration of flow from the watershed sub-basin in which it is located. However, extending the recession limb of the storm hydrograph for the sub-basin and delaying the peak flow of the tributary channel could adversely affect the main channel serving the composite watershed. If the peak discharge from the tributary's stormwater storage facility arrives at the main channel when the main channel peak does, an increase in peak flow downstream will occur, even with construction of the facility. The resulting adverse change to a composite watershed hydrograph as a result of downstream detention is shown graphically in Figure 8-6. The design of a facility should include a review of the entire watershed to verify that the release rate and timing from the tributary will not cause an increase in peak flow to the main channel.

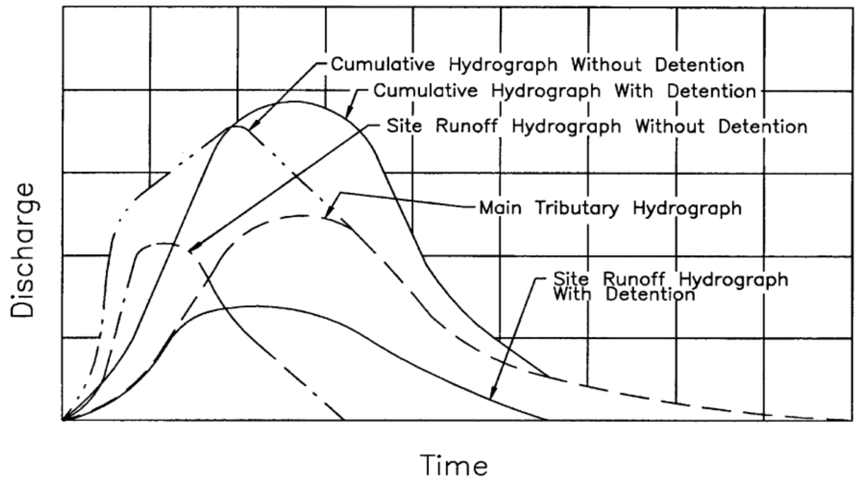


Figure 8-6
Cumulative Watershed Hydrograph With and Without Detention
Reference: USDOT, FHWA, HEC-22 (2001)

8.04.3 STORAGE REQUIREMENTS

A properly designed stormwater storage facility shall provide sufficient storage volume to attenuate the post-developed peak discharge rate to a rate equal to or less than the pre-developed peak discharge rate. Additional storage volume may be necessary to satisfy water quality requirements or to mitigate downstream capacity issues. Sedimentation during and after construction may necessitate additional volume to accommodate potential sediment loads. Routing computations must be performed to demonstrate that the storage volume provided is adequate.

The following subsections provide general guidance on providing stormwater storage for both quantity and quality control.

8.04.3.1 QUANTITY

The intent of stormwater quantity control is to limit the peak runoff from new development to a level equal to or less than the pre developed rate, or to provide improvements to flooding and erosion control issues in a downstream conveyance system by limiting the discharge from a proposed highway facility to a level consistent with the capacity of the downstream system. Quantity control structures reduce the peak storm discharges and increase the duration of the flow events.

By providing a storage facility to control the quantity of stormwater discharge, the following potential benefits may be achieved:

- Reduced peak runoff rates produced from roadway project development
- Mitigate any downstream drainage capacity problems
- Decrease the potential for downstream channel erosion
- Groundwater recharge
- Need for downstream outfall improvements reduced or even eliminated

8.04.3.2 QUALITY

The intent of stormwater quality control is to minimize the adverse impacts to the aquatic environment by reducing the level of soluble and particulate constituents generated by land development and highway construction projects. Although, it is not the intent of this chapter to provide a detailed discussion on the theory of stormwater quality, or the design measures to implement, the following paragraphs provide the designer with an initial guide to the basic design practices of a quality control facility. If a stormwater quality facility is required on a project due to environmental constraints, the designer is expected to research other reference material to gain further knowledge on the proper design methods, procedures, and requirements for such a facility. Additionally, project development should be coordinated with the Environmental Division.

Stormwater quality control offers several benefits including:

- Control of sediment deposition
- Settling out of roadway runoff pollutants
- Stormwater filtration provides improved water quality

If required for water quality purposes, a stormwater storage facility shall be designed to capture and treat the “first-flush” volume of runoff from a storm event. The purpose of providing adequate volume to capture the first-flush is to prevent sediment, debris, automotive fluids, deicing salts, and other pollutants from entering natural water courses. For design purposes, the “first flush” or water quality volume shall be defined as the first ½ -inch depth of stormwater runoff multiplied by the total impervious area within the contributing watershed. In equation form, this would be:

$$V_{wq} = \frac{0.5 inch}{12 \dfrac{inch}{foot}} \times A_{imp} \qquad (8\text{-}1)$$

Where: V_{wq} = required water quality volume, (ft^3)
A_{imp} = total impervious area, (ft^2)

The first flush volume within a detention facility must be released at a controlled rate over a specific time period. Drawdown time for the first-flush volume of an extended-detention basin should range between 24 and 48 hours, and between 24 and 60 hours for a retention basin. The desired pollutant removal rate will govern the final design of the drawdown device and the residence time (permanent pool volume divided by average outflow rate) of the first flush volume. Additional time may be required at specific sites to satisfy special pollutant removal requirements.

The type and required management practice for stormwater quality control is determined by the percent of new impervious area and the area within the right-of-way and easements at each project outfall. In urban areas where underground or in-line conveyance system detention is required, and water quality is an issue, the design should consider alternate means of stormwater quality control measures such as prefabricated pretreatment structures, oil/water separators, grassed filter strips in ditches and medians, or offsite constructed wetlands.

In rural areas, an extended detention basin may be adequate to effectively meet quality control requirements.

8.04.4 GRADING AND DEPTH

The following sections provide grading and geometric requirements which should be considered for all proposed stormwater storage facilities to be included on TDOT roadway projects. Additional requirements may be required for structures meeting the requirements of the Safe Dams Act (Section 8.04.5.1.1).

8.04.4.1 GENERAL RECOMMENDATIONS

The following general grading recommendations should be considered and applied to all detention and retention structures designed as part of a TDOT project. Additional requirements specific to detention, extended detention, or retention facilities can be found in Sections 8.04.4.3 and 8.04.4.4, respectively. As a general rule, the recommendations for grading and design of stormwater basins should include:

- The designer should attempt to minimize the proposed final depth of ponded water in the basin to the maximum practical extent. To minimize the potential hazards associated with open water, the ponded depth should be limited to 2 feet maximum for water quality, and 4 feet maximum for quantity control of the design storm. Total maximum depths to 3 feet are preferred.
- A shallow basin with a large surface area will perform better than a deep facility with the same volume.
- A maximum side slope of 3H:1V, where possible. The maximum side slope of 3H:1V is based on the ability of maintenance vehicles or conventional mowing equipment to safely traverse the side slopes.
- Side slopes meeting clear zone requirements for the roadway if any portion of the facility is located within the clear zone.
- Embankment slopes of 3H:1V, especially when embankment heights exceed 6 feet, and in no case shall embankment slopes be steeper than 2H:1V.
- The top of all earthen embankments shall be graded to provide positive drainage.
- Facilities should not be located on fill slopes or on or near slopes exceeding 15%.
- Construction, maintenance, and safety should be considered during the planning and design as outlined in Section 8.06.

8.04.4.2 BASIN GEOMETRY

Designers should attempt to provide stormwater facilities with a minimum length to width ratio of 3:1. The length is considered the distance between the basin inlet and the principal outlet control device. The width may be considered the average width at the expected design storm water surface elevation. The purpose of providing this design ratio is to reduce velocity and minimize the potential for short circuiting of flow through the basin. The basin’s longest dimension should be parallel to the contours to minimize required earthwork. Where the basin width is not constant, the wider dimension of the basin should be located at the outlet end of the facility. A flow expansion can be produced by designing the narrow end of the structure at the inlet to the facility. The corners of all basins should be rounded to provide adequate turning radius for maintenance and mowing equipment.

8.04.4.3 DETENTION REQUIREMENTS

Where a detention or extended-detention basin is to be used on a roadway project for quantity or quality purposes, the designer shall consider the following specific requirements in addition to the general design recommendations of Section 8.04.4.1. Each requirement should be considered during the planning and design of the facility to insure the facility functions as intended for adequate storm control, pollutant removal, and ease of maintenance.

- The bottom of a detention facility should be sloped no more than 2% and no less than 0.5% from the inlet toward the outlet device to allow for positive drainage of the facility.
- The maximum depth of ponded water should be less than 10 feet. Geotechnical slope stability analysis should be performed for any of the embankment conditions described in Section 8.05.4.1.
- Slope adjacent areas above the design high water elevation toward the basin to prevent standing water.
- A concrete low-flow channel or “trickle ditch” may be required across the bottom of the basin from the inlet to the outlet to convey low flows and dry weather inflows (base flow) to the principle outlet without detention, and to prevent standing water. Ideally, low flow within the basin should be allowed to spread out across the basin floor. If a low flow channel is used as part of the design, it shall be designed to overflow during a storm event, and spread the flow across the basin floor. Where water quality is the governing design criteria of the facility, low-flow channels are not recommended for use because they increase the potential for short circuiting, thereby negating the intended purpose of the facility. See Figure 8A-3 in the Appendix for an example of a storage basin with a low flow “trickle ditch”.
- The bottom of facility should be graded toward the outlet or a low-flow channel to prevent standing water.
- The facility should be designed with dedicated access to the basin floor for sediment removal or mowing equipment. Typically, a sloped ramp or a basin side slope of not less than 4H:1V, at a convenient location, will be adequate for access.
- For permanent facilities, provide bottom, side slopes, and embankment slopes with a permanent vegetative cover unless bedrock is exposed.

8.04.4.4 RETENTION REQUIREMENTS

Where a retention basin is to be used on a roadway project for water quality and quantity purposes, the designer shall consider the following specific design requirements in addition to those of Section 8.04.4.1. Each requirement should be considered during the planning and design of the retention facility so that the facility will function as intended for adequate storm control, pollutant removal, improved safety, and ease of maintenance.

- An average permanent pool depth of 3 to 6 feet is suggested.
- To avoid thermal stratification and anaerobic conditions, permanent pool depths should be limited to a maximum of approximately 12 feet.
- Approximately 15% of the retention basin area should be less than 18 inches deep. Figure 8-7 shows how this can be accomplished by providing a 10 to 15 foot wide aquatic bench extending from the waters edge to a point where the depth is between 12 to 18 inches. The aquatic bench shall have a maximum depth of 18 inches at the normal pool elevation and should be located around the perimeter of the facility.

- Where retention basin pool depth exceeds 3 feet at the normal pool elevation, a safety bench shall be required as shown in Figure 8-7. The safety bench is intended to provide a buffer and level of safety for maintenance equipment and other persons operating along the edge of the permanent pool. The safety bench shall be 10 to 15 feet wide and should slope toward the water surface with a suggested maximum 15H:1V slope. When retention facility side slopes are 4H:1V or flatter above the permanent pool, the safety bench may be omitted.
- Permanent pool facilities with fish habitat must have a deep water pocket with a minimum depth of 10 feet or greater, with an area that is equal to at least 25% of the total proposed bottom area. Shallow retention basins will not support year round fish populations.
- Retention facilities should be wedge shaped, with the wide end near the outlet where feasible, to increase the potential for flow expansion resulting in a greater degree of sedimentation.
- Retention facilities should be multi-celled utilizing a sediment forebay as the first cell to intercept incoming sediment prior to entering the permanent pool. For additional information on the design of a sediment forebay, see Section 8.04.9.
- To minimize the potential for breeding of mosquitoes, a 1 foot drop should be considered above the normal pool elevation between the safety and aquatic benches.
- Geotechnical slope stability analysis should be performed for any of the embankment conditions described in Section 8.05.4.1.

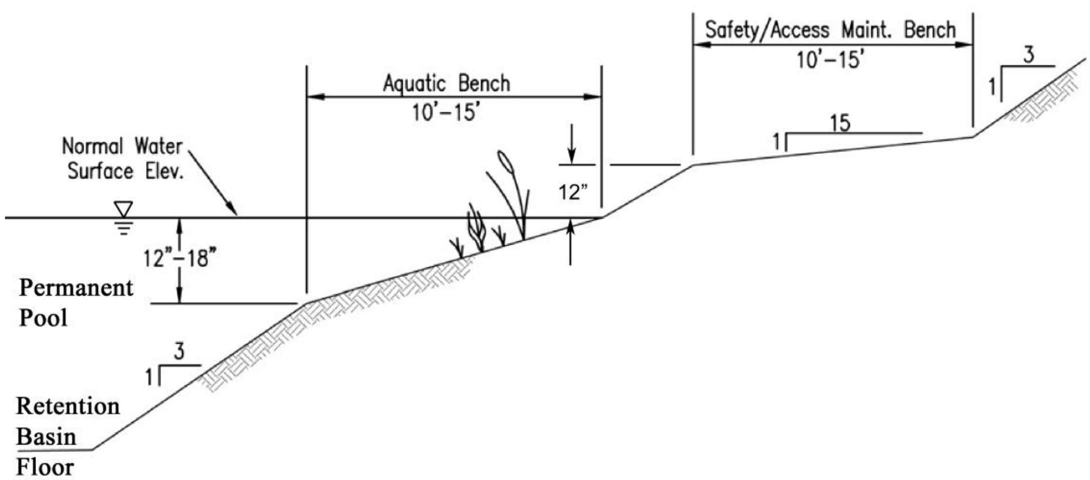


Figure 8-7
Typical Aquatic & Safety Bench Plan
For Retention Basins
Reference: Virginia DCR, (1999)

8.04.5 EMBANKMENTS

Embankment design is an important aspect of the stormwater storage facility and should be given special attention by a trained design professional. The following design criteria and details shall be incorporated in the planning and design of embankments to be used on TDOT stormwater storage basins. Further information, design procedures, and detailed discussion may be obtained by accessing the referenced documents in the chapter Appendix.

8.04.5.1 GENERAL CONSIDERATIONS

Embankments are complex structures that must be designed and constructed with consideration given to the following:

- Purpose of impoundment
- Hazard potential present
- Site specific conditions
- Foundation conditions
- Available construction materials
- Ability to support vegetation

Environmental and economic impacts should also be considered during the design process. The principal influence of environmental concerns on selection and design of an embankment is the need to consider protection of the environment, which may affect the type of embankment to be constructed, the dimensions, and the location of the emergency spillway. Economic analysis and comparison of possible alternative designs should be considered during the design and approval process.

In general, detailed geotechnical analysis should be performed for embankments greater than 10 feet in height or where the embankment side slope exceeds 2H:1V. However, geotechnical analysis should also be considered for lower embankments in flat topography where the potential exists for impounding large quantities of water, or in steep areas with shallow bedrock where the potential for sliding exists.

8.04.5.1.1 HAZARD POTENTIAL

Where specifically required, the design and construction of embankments are required to meet the latest requirements of the Tennessee Safe Dams Act of 1973, including all subsequent amendments. As defined in the Act, a dam is an artificial barrier which impounds or diverts water and is, or will be, 20 feet or more in height ***or*** has a storage volume of 30 acre-feet or more; provided however, any such barrier that is less than 6 feet in height (regardless of storage capacity) ***or*** will have a maximum storage capacity of 15 acre-feet or less (regardless of height), *shall not be considered a dam*. The height of a dam in Tennessee is further defined in Section 8.04.5.3.

The hazard potential associated with a stormwater impoundment is defined in the Tennessee Safe Dams Act. It is based on the potential for loss of life, damage to property, and/or economic loss due to structure failure. Stormwater storage impoundments for Department projects will typically be smaller than those regulated under the Safe Dams Act, but the potential for structural failure leading to property damage or loss of life may still be present. For those facilities ***not*** covered under the Tennessee Safe Dams Act, minimum guidelines are provided in subsequent sections of this chapter.

The embankment design criteria presented in the following sections *does not* apply to structures meeting the definition, and falling under the requirements of the Safe Dams Act. Additionally, the design procedures presented may not apply to embankments or outfall structures with maximum heights of 3 feet or less. The designer is encouraged to apply these minimum standards; however, for small facilities sound engineering practices, judgment, and supporting technical calculations may govern the final design of the facility. On projects containing structures meeting the criteria for a dam in Tennessee, the designer should coordinate with the Hydraulics Section of the Structures Division.

8.04.5.2 EMBANKMENT TYPES

The two types of earthen embankments described in this Manual are homogeneous and zoned embankments. The purpose of the stormwater storage facility, site specific conditions, and the available material for construction should govern the selection of the type of embankment. The two types of embankments are briefly described in the following two sections. The designer is responsible for researching additional material to gain a comprehensive understanding of the design and construction of both embankment types.

8.04.5.2.1 HOMOGENEOUS EMBANKMENTS

A homogeneous earthen embankment is constructed of similar earthen material throughout, excluding the outer slope protection. The material used for a homogeneous embankment must be impervious to provide an adequate water barrier and the side slopes of the facility must be relatively flat for stability and ease of maintenance. With a homogeneous embankment, an internal drain may be used to prevent the downstream slope from becoming saturated and susceptible to piping and/or slope failure (See Figure 8A-4 of the Appendix).

For most design projects consisting of relatively small dry detention or extended detention facilities, a homogeneous embankment will be adequate. Any homogeneous embankment on a project which exceeds 15 feet in height shall be designed with seepage controls, or otherwise, a zoned embankment should be provided. A cut-off core trench should be used at all locations where seepage may be expected.

8.04.5.2.2 ZONED EMBANKMENTS

A zoned embankment contains an impervious core (typically clay) flanked by layers (zones) of more pervious material having differing degrees of porosity, permeability, and density. The pervious layers support and protect the impervious embankment core. Internal drains between the impervious zone and the downstream pervious material, and between the downstream pervious material and the foundation will be required. Embankment zones shall be a minimum of 10 feet wide except for filters and drains of a specified material gradation. Figure 8A-5 of the Appendix shows typical zoned embankments.

The use of a zoned embankment should be considered under the following conditions:

- The proposed impoundment requires an embankment greater than 15 feet in height
- The need to obtain a stable structure with the most economical use of available construction materials
- To control embankment seepage in a safe manner
- To minimize the uncertainties of material strengths and resultant stability

8.04.5.3 EMBANKMENT HEIGHT AND FREEBOARD

The height of an earthen embankment is the vertical distance from the natural bed of the stream or watercourse, measured at the downstream toe of the embankment, to the top of the embankment. Where the embankment is not located within a watercourse, the height is the vertical distance between the lowest elevation, measured at the outside limit of the embankment, and the top of the embankment.

For roadway projects, the designer shall provide a minimum of 1 foot of freeboard above the computed emergency overflow water surface elevation when the spillway is discharging at design depth, measured to the lowest point along the top of the embankment (excluding the emergency spillway). In the rare event that a stormwater facility measures in excess of 660 feet in length (measured perpendicular to the embankment), a minimum freeboard of 2 feet will be required to accommodate potential wave action. Additionally, the design height must also meet the requirements for minimum emergency spillway depth (Section 8.04.7.1).

An allowance for settlement should be included with embankment designs. Settlement includes the consolidation of the fill materials due to the weight of the embankment and the increase in moisture resulting from the storage of water. Settlement depends on the material characteristics and the methods used during construction of the embankment. The design height of an embankment should be increased a minimum of 5 percent to compensate for the settlement, except where detailed soil analyses indicate a lesser amount is adequate.

8.04.5.4 TOP WIDTH

The minimum top width of an earthen embankment ***not*** classified as a dam in Tennessee (See Section 8.04.5.1.1) shall be as shown in Table 8-1.

Total Height of Embankment (ft)	Minimum Top Width (ft)
Under 10	8
10-15	10
15-20	12
20 or greater	15

Table 8-1
Minimum Top Width of Embankment
Reference: USDA, SCS, Eng. Field Manual, Chap. 11

The total height of an earthen embankment provided in Table 8-1 is defined in Section 8.04.5.3. The designer may be required to increase the minimum top widths to accommodate embankment zoning, provide safe access for maintenance vehicles, or for structural stability.

8.04.5.5 STABILITY AND CUT-OFF TRENCH

Embankment and side slope stability is dependent upon the intended use of the facility, expected water surface elevation, embankment height, cross section of embankment, foundation material and construction material and methods. All stormwater facility embankments must be stable against force conditions which could occur after construction of the embankment. The following three conditions are the most critical in regard to embankment failure:

- Seepage through the foundation or embankment or both
- Shear stress from the weight of embankment material exceeds the strength of the materials
- Differential settlement due to material variations, height variations, or compression of the foundation strata

Geotechnical slope stability analysis will be required for embankments with heights exceeding 10 feet and for embankments designed with slopes steeper than 2H:1V. For all other structures, the Structures Division should determine the need and extent of any required stability analysis.

When required, a geotechnical engineering study to evaluate the stability of the proposed embankment should be performed following the completion of a preliminary design. The geotechnical investigation should consist of:

- Site investigation to include test borings, pits, review of available soils information, and an evaluation of the foundation materials, reservoir area, and preliminary embankment design.
- Soil testing of the sampled material to include analysis of shear strength and permeability of the proposed foundation and construction materials.
- Report of findings by a geotechnical engineer to include a discussion on the suitability of the soil, required construction methods, and additional design features such as interior filters or toe drains which may be required as part of the design.

The previous list may not be applicable to all projects depending on the size and scope of the planned facility. The Structures Division is responsible for determining the final need (if any) for testing and the extent of any required tests applicable to a specific project.

The foundation under an embankment must insure stable support for the structure and provide the necessary resistance to the passage of water. The area under the footprint of the embankment must be cleared of all organic material prior to placing material lifts for the embankment. A cut-off trench (keyway) should be provided at all earth fill embankments unless geotechnical analysis indicates otherwise or a detailed design of drainage diaphragms is provided. When required, a foundation cut-off trench shall be provided along the axis of the embankment (at or upstream of the embankment centerline) and should extend up the abutments of the structure to a point above the expected 10-year water surface elevation.

Cut-off trenches should be constructed of successive thin layers of clay or other relatively impervious material, compacted to optimum moisture content, and should extend a minimum of 4 feet into a suitable and stable impervious soil layer beneath the embankment. The trench shall have a bottom width adequate to allow for excavation and compaction, but shall not be less than 4 feet wide. The side slopes of the trench shall be no steeper than 1H:1V, and the trench shall have a minimum depth of 4 feet. Figure 8-2 and Figure 8A-7 show examples of cut-off trenches under an earthen embankment.

8.04.5.6 EMBANKMENT SEEPAGE

To the extent necessary, especially for larger impoundments and permanent pool facilities, an analysis should be made of anticipated seepage rates and pressures through a proposed embankment. The rate of seepage through an embankment is dependent on the consistency of the reservoir level and the permeability of the material used for construction. Seepage control is necessary to prevent excessive uplift pressures, instability of the downstream slope, piping through the embankment, and erosion of material by migration through open joints in the foundation. This section is intended to provide the designer with *limited* guidance on control of seepage within a stormwater storage basin embankment. For detailed information on seepage analysis and control for embankments, the designer should refer to USACE, EM 1110-2-1901. Guidelines for minimizing seepage and piping along a principal spillway conduit are provided in Section 8.04.6.4.

The phreatic surface is the upper surface of seepage and is the zero pressure line or phreatic line. For retention facilities, the phreatic line should be considered to begin at the permanent pool elevation and extend through the embankment at a slope of 4H:1V. For detention and extended detention facilities, the line should be considered to begin at the maximum 10-year water surface elevation. When the phreatic line intersects the downstream face of the embankment at the toe elevation or above, seepage will occur along the downstream face. Figure 8A-7 of the chapter Appendix provides a diagram showing the location of the phreatic line within a cross-section of an earthen embankment. The position of the saturation line in a homogeneous embankment is independent of the type of soil used in the embankment.

The embankment design for a storage facility (especially a retention basin) should include seepage control measures when:

- The phreatic line intersects the downstream slope
- Pervious layers within the foundation are not intercepted by a cutoff trench
- Possible seepage at the ends of the embankment could result in a wet embankment
- Special conditions exist which necessitate drainage to insure embankment stability

Seepage through an embankment can be controlled by one or more of the following methods:

- Constructing structures with flat embankment grades so that the phreatic line is contained within the downstream slope
- Constructing zoned embankment with increasing layers of permeability from the embankment core toward the slopes
- Utilizing vertical and horizontal drains within the downstream portion of the embankment, to include foundation drains, blanket drains, or toe drains
- Including a toe drain in larger homogeneous embankments, where required

8.04.6 PRINCIPAL SPILLWAY

The principal spillway serves as the primary outlet device for a stormwater storage facility. The primary function of the principal spillway is to pass stormwater discharge through the embankment in a safe, non-erosive manner while minimizing the frequency of use of the emergency spillway. The principal spillway is designed to attenuate, release, and pass storm events at the allowable discharge rates.

The most commonly used principal spillway configuration is the drop-inlet spillway (riser and pipe barrel) as shown in Figure 8-8 and 8A-6. Drop-inlet spillways will typically consist of a single or multi-staged riser control structure with a discharge pipe extending through the embankment as shown in the figures. In some cases, the principal spillway may only consist of a pipe barrel through an embankment with appropriate end treatments (endwalls).

8.04.6.1 GENERAL REQUIREMENTS

The control structure for the stormwater storage facility should consist of a combination of orifices and/or weirs to control the 2-year through 10-year flow rate from the stormwater storage facility. For storms greater than the 10-year/24-hour storm, flows will pass over a weir into the spillway conduit, which will carry the outflow to the downstream receiving channel. The capacity of the principal spillway conduit shall be adequate to discharge long-duration, continuous, or frequent flows without flow through the emergency spillway.

The principal spillway crest elevation shall be no less than 0.5 feet below the crest elevation of the emergency spillway for basins having a contributing drainage area of 20 acres or less. Where the contributing drainage area to the storage facility exceeds 20 acres, the crest of the principal spillway should be no less than 1 foot below the emergency spillway crest.

For storage facilities without a separate emergency spillway, the principal spillway must function as the emergency spillway. In this situation, the principal spillway must comply with the emergency spillway hydrologic criteria presented in Section 8.04.7.4. Additionally, the designer shall insure adequate protection against clogging is provided at the inlet of the principal spillway.

8.04.6.2 RISERS

Risers are used in detention basins to control one or more discharge rates. Risers may be either round, square, or rectangular depending on the final design requirements. Risers may be placed at the interior toe of the embankment slope or in the embankment with only the outlet control devices and the riser top exposed. Risers are to be structurally designed to withstand all water and earth loads anticipated on the device. Most risers for principal spillways are constructed of concrete or corrugated metal anchored in concrete. Prefabricated pipe risers shall be a minimum of at least one standard pipe size larger than the pipe barrel material, and of the same material. Prefabricated concrete risers should be anchored together for stability and flotation requirements and should have watertight joints between the riser sections.

The riser of a drop-inlet spillway should have a cross-sectional area that is larger than the outlet conduit to minimize surging, noise, vibration, cavitation forces, and vortex action. Additionally, full-flow should be established in the riser (and the conduit) at the lowest head over the riser as possible. The riser size shall be such that the storm discharge through the structure goes from weir-flow control to barrel-flow control without going into orifice-flow control within the riser (see Section 8.05.8 for a description of these flow regimes). The connection between the riser and the principal spillway conduit must be watertight.

All risers should be analyzed for the possibility of floatation by performing buoyancy calculations. This analysis should assume that all outlet devices are clogged. The factor of safety against floatation shall be 1.25 or greater (i.e. downward forces = 1.25 x upward forces).

All risers shall consist of a vertical pipe (barrel) or box of concrete or corrugated metal anchored to an anti-floatation base. The base of the principal spillway riser must be firmly anchored to prevent flotation. Detailed design and computations for determining anchoring requirements must be made for all TDOT stormwater storage facilities if:

- The riser height is greater than 10 feet
- The facility is designed as a retention (permanent pool) basin

For risers 10 feet or less in height, the anchoring may be accomplished in one of two ways:

- A concrete base 18 inches thick and twice the width of the riser diameter should be used. The riser should be embedded a minimum of 6-inches into the concrete base.
- A square steel plate welded to the base of the riser having a minimum thickness of ¼ inch, and having a width equal to twice the diameter of the riser. The steel plate shall then be covered with a minimum of 2.5 feet of compacted gravel or soil to prevent flotation.

8.04.6.3 CONDUITS

The barrel of the principal spillway, which extends through the embankment, should be designed to carry the flow provided by the riser. All conduits (barrels) under or though a TDOT stormwater storage basin should meet the following requirements:

- Pipe barrels shall be circular in cross section. Elliptical pipe is not permitted
- The minimum acceptable inside diameter should be 18 inches
- Spillway conduits shall have a positive slope toward the outlet
- Pipes shall be capable of withstanding external loading without yielding, buckling, or cracking which could result in joint separation
- Inlet or outlet end sections (i.e. endwalls), when used, shall be structurally sound and made of material compatible with the principal spillway pipe
- The pipe joints and all appurtenances of the spillway shall be watertight

The barrel of drop-inlet spillways should be straight in alignment when viewed in plan.

8.04.6.3.1 MATERIALS

Acceptable pipe material for TDOT stormwater basins will consist of the following:

- Concrete with rubber gaskets
- Corrugated steel or aluminum
- Welded steel
- Cast or ductile-iron
- High-density polyethylene (HDPE)

Gasketed, water-tight concrete or corrugated metal pipe should be used as the principal spillway conduit on most TDOT stormwater storage facilities (the preferable material). Under certain conditions, welded steel, cast-iron, HDPE, or ductile-iron pipe may be used as a principal spillway conduit. Fill heights and foundation conditions may require special design consideration for these pipe materials. Their use will be approved only on a case-by-case basis by the Design Manager. If used, proper cradling or encasement in concrete should be provided.

Where HDPE or CMP spillway conduit is used, the designer should attempt to specify the longest section of pipe possible, thereby minimizing or eliminating pipe joints. The maximum deflection of HDPE or CMP conduits should be 5 percent of the pipe diameter. If the potential for flotation exists, the pipe should be anchored with flowable fill.

8.04.6.3.2 CONSTRUCTION CONSIDERATIONS

Conduit joints shall be designed and constructed to remain water tight under maximum anticipated hydrostatic head and after joint elongation caused by foundation consolidation. Joints should be made watertight by the use of flanges with gaskets, coupling bands with gaskets, bell and spigot ends with gaskets, or by welding.

To provide adequate structure life, a protective coating of asphalt on galvanized corrugated metal pipe shall be considered in areas having a history of pipe corrosion or where the soil pH is lower than 5.

Cathodic protection should be provided for galvanized corrugated metal pipe or welded steel pipe where soil and resistivity studies warrant the need for the protective coating, or where the need and importance of the structure warrants additional protection and longevity.

TDOT standard granular pipe bedding ***shall not*** be permitted under principal spillway conduits unless it is designed to serve as a drainage diaphragm (see Section 8.04.6.4.2). Where required, concrete and high-density polyethylene pipes shall have a concrete cradle extending up the sides of the pipe at least 50 percent of its outside diameter with a minimum thickness of 6 inches. For large basins or permanent pool facilities, a concrete cradle will serve to reduce the potential for piping and provides a 90 degree bedding angle for the loading. Concrete bedding (flowable fill) may be used when a concrete cradle is not needed for structural reasons.

8.04.6.3.3 USE OF MULTIPLE CONDUITS

Due to compaction issues and the increased potential for piping along conduits, the use of multiple conduits through an earthen embankment should be avoided and will only be allowed under special circumstances. For most TDOT stormwater facilities, the need for multiple pipe barrels will not be necessary. In special cases where multiple barrels are used as the principal spillway device, the following minimum criteria shall apply:

- The pipe barrels should be circular in cross-section
- Sufficient space shall be provided between barrels to allow for anti-seep collars and proper compaction around the conduits using mechanical and hand-operated compaction equipment
- Minimum spacing between conduits shall be equal to or greater than one-half the pipe diameter, but not less than 2 feet

8.04.6.4 SEEPAGE AND PIPING

Seepage of water along the exterior surface of a principal spillway conduit causes the progressive internal erosion of embankment material (piping) from around the pipe. Piping is generally caused by poor compaction of material around the pipe and inadequate seepage control measures. Embankment material must be carefully placed and compacted next to principal spillway conduit; otherwise, unconsolidated material may be the starting point for piping. From there, removal of soil particles, and subsequent differential settlement of the embankment material, may result in separation of pipe joints allowing stormwater to flow into the surrounding soil. Once piping occurs, the flow passages and seepage increases until failure occurs, eventually resulting in possible embankment failure.

Measures to reduce the potential or control seepage and piping along the principal spillway barrel should be considered for use on TDOT detention basins and shall be used at all permanent pool facilities. The following two sections provide guidance to the designer for the proper application, use, and design of anti-seep collars and drainage diaphragms.

8.04.6.4.1 ANTI-SEEP COLLARS

The purpose of anti-seep collars is to control internal piping or seepage along the outside surface of a principal spillway barrel by extending the flow path of water along barrel, and keeping the hydraulic gradient low. Figure 8-8 provides a representation of the use of anti-seep collars through an earthen embankment. Figure 8A-8 in the Appendix shows anti-seep collars installed on a drop-inlet principal spillway prior to backfilling and compaction of an embankment.

Anti-seep collars shall be used in the saturation zone of an embankment to increase the seepage length along the conduit by 15 percent. The normal saturation zone, or phreatic line, is defined in Section 8.04.5.6. All fill material located within this line should be considered saturated. The length of the principal spillway barrel located in the saturation zone can be computed by Equation 8-2 or by graphically extending a line sloped at 4H:1V from the intersection of the 10-year high water elevation and the inside face of embankment to a point where this line intersects the invert of the conduit.

$$L_S = Y(Z+4)\times\left(1+\frac{S}{0.25-S}\right) \quad (8\text{-}2)$$

Where: L_S = length of barrel in saturated zone, (ft)
Y = depth of water at principal spillway crest, (ft)
Z = embankment side slope (inside facility), (i.e. 3H:1V, Z=3)
S = slope of barrel, (ft/ft)

Multiplying L_S by 0.15 provides the total required increase in seepage length. The increase in seepage length represents the *total* collar projection to be provided using one or more collars. To determine the vertical projection required for a 15 percent increase in seepage length, solve the ratio $(L_S+2nV)/L_S \geq 1.15$ for V. The term ‘n’ represents the number of collars.

Figure 8A-9, provides a simplified graphical method for designing anti-seep collars at a stormwater storage facility. With only L_S and the barrel diameter known, the designer can determine both the number and size of anti-seep collars to use at a project site.

Anti-seep collars shall be used when:

- The proposed embankment height exceeds 10 feet
- Principal spillway barrel exceeds 12 inches

- Embankment material contains a low silt-clay content

The following requirements for anti-seep collars shall apply:

- All anti-seep collars shall be placed within the theoretical zone of saturation. In cases where spacing limits will not allow this, at least 1 collar shall be in the saturated zone.
- Connection between collar and principal spillway pipe shall be watertight
- A minimum of 1 collar should be used for embankments less than 15 feet in height
- A minimum of 2 collars should be used for embankments 15 feet or greater in height
- Collars shall extend a minimum of 2 feet in all directions from around the spillway pipe
- Spacing of collars shall be between 5 and 14 times the required vertical projection of each collar above the pipe, to a distance of not more than 25 feet
- Collars should be equally spaced except where necessary to avoid a pipe joint
- Clearance of 2 feet from a pipe joint shall be maintained unless flanged joints are used
- Collars shall be made of a material compatible with the principal spillway conduit. That is, concrete collars should be applied to concrete barrels, and steel plate should be used where corrugated metal pipe spillway barrel is used. Concrete should be used to form collars for high-density polyethylene outlet conduits. The concrete should be cast-in-place in order to allow it to properly mesh with the external corrugations of the pipe.

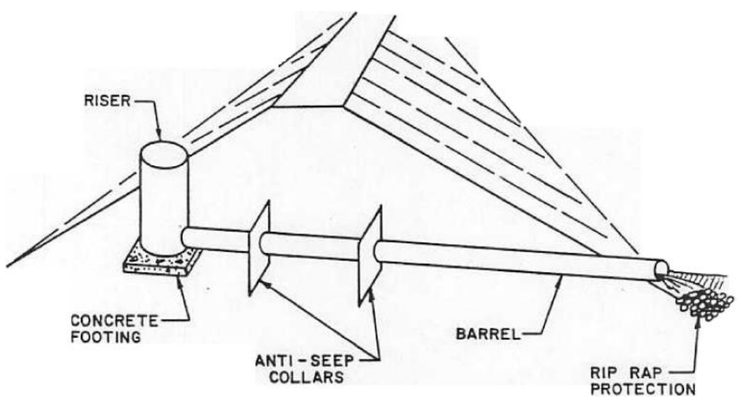


Figure 8-8
Anti-Seep Collars Along Principal Spillway Barrel
Reference: Dept. of Interior, 1982

Designers and engineers continue to use anti-seep collars as the principal means to control piping and seepage along conduits in small structures. The use of anti-seep collars should be acceptable for use in most TDOT stormwater storage basins. However, the designer should consider an alternative method of seepage control for larger structures, special condition sites, and retention (permanent pool) facilities. The U.S. Army Corps of Engineers, N.R.C.S., and the Department of Interior, Bureau of Reclamation no longer recommend the use of anti-seep collars to control piping in large dams and other structures. The use of drainage filters and diaphragms are the recommended alternative for larger structures and are briefly discussed in the following section.

8.04.6.4.2 FILTER AND DRAINAGE DIAPHRAGMS

An alternative method to control seepage along the principal spillway conduit is the use of sand filter and drainage diaphragms. Filter and drainage diaphragms are used to control the transport of embankment fines along a principal spillway by channeling flow through a filter of fine graded material which serves to trap the transported material. In contrast to anti-seep collars, filter diaphragms do not discourage or eliminate piping, nor do they extend the seepage length along the conduit.

Some advantages to using filter and drainage diaphragms instead of anti-seep collars are:

- Embankment fill material adjacent to the principal spillway conduit can be more thoroughly compacted when anti-seep collars are eliminated from the spillway barrel
- Cracks in the embankment fill along the principal spillway conduit will not extend completely through the embankment
- The use of filter diaphragms eliminates the labor intensive construction and use of multiple anti-seep collars along the principal spillway barrel

The disadvantage to using filter and drainage diaphragms is that each site requires detailed engineering design and construction supervision. Additionally, suitable filter material must be on site or imported to the project location. For these reasons some N.R.C.S. offices do not endorse the use of diaphragms on small basin projects.

The following general design criteria should apply when filter and drainage diaphragms are to be used at TDOT stormwater facilities:

- Diaphragms within a homogeneous embankment shall be placed directly downstream of the embankment cut-off trench or downstream of the embankment centerline if no cut-off trench is specified.
- For zoned embankments, the diaphragm shall be located downstream of the core zone and/or cut-off trench.
- Diaphragms should be parallel to the embankment centerline.
- Minimum thickness of drainage diaphragms shall be 3 feet.
- Soil cover over any portion of a filter diaphragm shall be a minimum of 2 feet, measured normal to the nearest embankment surface.
- Diaphragms shall extend both vertically upward and horizontally a minimum of 3 times the circular conduit’s outside diameter, except that;
    - The vertical extension need not be any higher than the maximum potential reservoir water level, and,
    - The horizontal extension need be no further than 5 feet beyond the sides and slopes of any excavation made to install the conduit.
- Diaphragms should extend vertically downward the *lesser* of either, 24 inches below the barrel invert *or* to sound rock. Terminate the diaphragm at bedrock.
- Drainage diaphragms should discharge at the downstream toe of the embankment.
- The outlet for a filter and drainage diaphragm should extend around a pipe surface a minimum of 1.5 times the outside diameter of the conduit.
- In general, for stormwater storage facilities, filter diaphragms should be constructed of sand with at least 15 percent passing the No. 40 sieve and with no more than 10 percent passing the No. 100 sieve.

- Filter and drainage diaphragm design should be performed by a competent geotechnical engineer for each intended location of use.

It should be noted that these are general case design parameters. Detailed design criteria and methods can be found in SCS TR-60, SCS-Technical Note 709 (including Supplement), and SCS National Engineering Handbook, Chapter 26, Part 633. The designer should reference the published documents from the previously listed agencies (Section 8.04.6.4.1) to gain a complete understanding of the design of filter and drainage diaphragms.

8.04.6.5 DEBRIS CONTROL AND SAFETY GRATES

All control structures for TDOT stormwater impoundment facilities should be equipped with a properly designed debris control device, or trash rack, to prevent clogging of the outlet device or principal spillway pipe. This should include providing a trash rack at the small openings typical of extended-detention and retention low flow orifices and weirs.

Trash racks and safety grates are an important aspect of principal spillway design and serve the following functions:

- Debris is trapped away from the primary outlet works to avoid clogging the critical portions of the outlet.
- Debris is trapped in a way that makes removal relatively easy.
- The potential for people or animals being trapped in the confined outlet devices is minimized or eliminated.
- A safety mechanism is provided whereby persons caught in the facility during a storm event will avoid the normally high velocities at the entrance to the outlet works by allowing them a means to climb to safety.

When properly designed, trash racks and safety grates serve their purpose without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1975, Allred-Coonrod, 1991).

Design criteria for trash racks should include the following:

- The openings for trash racks should not exceed one-half the pipe diameter of the principal spillway conduit.
- The average flow velocity through trash rack should not exceed 2 feet per second under the full range of stage and discharge, computed on the basis of the net area of the opening through the rack.
- The clear spacing of bars at large discharge openings should be no less than 6 inches
- Access to the outlet by children or animals can be minimized by limiting bar spacing to a maximum of 1 foot.
- Use of flat grates at permanent pool facilities is discouraged by the Department. For inlet structures with flow over the top, a non-clogging trash rack such as a hood type inlet which allows passage of water from underneath the rack should be used, and should extend a minimum of 8 inches below the weir openings.
- Trash racks, safety grates and mounting hardware should be galvanized or stainless steel and should be installed in a manner to minimize the potential for vandalism.

- Trash racks at pipe or conduit openings (with endwalls) should be sloped at 3H:1V to 5H:1V to allow trash to slide up the rack with rising water levels.
- Trash racks and grates should not be set in concrete. Bolted in place and hinged racks are preferable to aid in maintenance and unclogging.
- Trash racks should have a total opening area equal to or greater than ten times the outlet opening. They should be constructed with a sloping face in order to allow floating debris to rise with the water level in the basin.

The use of trash racks and safety grates should not shift control of the outlet to the rack or grate. Additionally, the grate should not cause headwater in the stormwater facility to rise above design levels. When determining losses and head at a grate, the designer should assume a certain percentage of blockage of the structure in his computations. For TDOT stormwater facilities, a percentage of blockage of 50 percent should be used (a higher percentage should be considered if the upstream watershed is predominately wooded). The following equation can be used for determining losses at a trash rack or safety grate (Metcalf & Eddy, 1972):

$$H_g = K_{g1}\left(\frac{w}{x}\right)^{1.333}\left(\frac{V^2}{2g}\right) sin\,\theta \qquad (8\text{-}3)$$

Where: H_g= head loss through the grate, (ft)

K_{g1} = unitless bar shape factor

= 2.42 for sharp-edged rectangular

= 1.83 for rectangular bars with semi-circular upstream faces

= 1.79 for circular bars

= 1.67 for rectangular bars with semicircular upstream and downstream faces

w = maximum cross-sectional bar width facing flow, (inches)

x = minimum clear spacing between bars, (inches)

V = approach velocity, (feet per second)

θ = angle of the grate with respect to horizontal, (degrees)

g = gravitational acceleration, (ft / sec^2)

8.04.6.6 ANTI-VORTEXING MEASURES

All closed conduit spillways designed for pressure flow are to be equipped with adequate anti-vortex devices to prevent the formation of a flow inhibiting vortex in the water at the entrance to the structure during periods of high flow. An anti-vortex device should be attached to the principal spillway inlet to increase the hydraulic efficiency of the structure when the structure is operating under pressure flow. Figure 8A-10 of the Appendix shows typical applications of steel plate anti-vortex baffles. Most anti-vortex devices can easily be incorporated into a trash rack design. When weir flow control is maintained in the riser for all design storm events (including the 100-year storm), an anti-vortex device will not be required.

8.04.6.7 OUTLET PROTECTION

All principal spillway conduits should be designed with an adequate energy dissipation device at the outlet end of the pipe due to the high energy discharges passing through the spillway. The outlet should be protected to prevent localized scour or erosion at the discharge point. The outlet protection required will depend on the energy (flow velocity) produced from the spillway design discharge. Dissipation could easily be accomplished with the use of a concrete endwall and riprap apron, or may require design of an energy dissipation device. See Chapter 9 of this Manual for further information on the detailed design of energy dissipators. Design procedures for riprap aprons are provided in Section 6.05.5.

8.04.7 EMERGENCY SPILLWAY

An emergency spillway is an open channel, constructed with an embankment, typically consisting of an inlet and outlet channel and a horizontal control section. The intent of an emergency spillway is to safely pass the discharge that exceeds the capacity of a principal spillway, at a non-erosive velocity, to an adequate receiving channel. Selecting, sizing and designing an emergency spillway should be consistent with the potential threat to downstream life and property if the embankment were to fail. Although not always the case, it is generally assumed that once an embankment is overtopped, the embankment will fail. For this reason, an emergency spillway will be provided at stormwater facilities to diminish the potential for catastrophic embankment failure.

8.04.7.1 GENERAL REQUIREMENTS

An emergency spillway, in conjunction with some form of principal spillway, shall be provided at all stormwater storage facilities where impoundments are created by constructing an earthen embankment. The emergency overflow spillway shall be sized to pass the 100-year/24-hour peak flow rate at a maximum flow depth of 1 foot below the settled top of the embankment. Where the entire stormwater detention facility is excavated into natural earth or rock and there is no embankment to overtop (i.e. an excavated “pit” which, in the event of overtopping, would result in sheet flow across natural ground), an emergency spillway may not be required. In this event, the principal spillway should be designed to convey all storm events, including the 100-year/24-hour peak discharge. In the absence of an emergency spillway, the principal spillway should be designed with an inlet which will not clog, such as a hood type inlet with an anti-vortex device.

8.04.7.2 LOCATION

Vegetated emergency spillways should be constructed in undisturbed, “cut”, earth at one or both ends of the embankment as shown in Figure 8A-11 of the chapter Appendix, or over a topographic saddle anywhere on the periphery of the basin. The Department discourages the placement of an emergency spillway on the embankment and the designer should attempt to locate the spillway off the embankment. If this is not practical due to adjacent site constraints or topography, then an armored spillway over the top of the embankment may be considered. When a rigid structural spillway is used over an embankment, such as a chute or drop spillway, the design should be in accordance with the *National Engineering Handbook, Section 11 or Section 14.*

8.04.7.3 COMPONENT LAYOUT AND CROSS SECTION

The typical excavated emergency spillway consists of three components: an inlet channel, control section, and exit channel. The inlet channel should be relatively short and should be sloped toward the reservoir at a rate of not less than 2 percent. The alignment may be straight or should have smooth curves. A large cross-sectional area of flow in the inlet channel, in comparison to the flow area at the control section, facilitates a uniform distribution of flow within the inlet channel and minimizes energy losses. In general, the entrance width of the inlet channel should be 50 percent greater than the bottom width of the control section (see Figure 8A-11).

The control section is the point along the emergency spillway where flow passes through critical depth. The control section should be located close to the intersection of the embankment and the emergency spillway centerlines. The control section should be designed level and perpendicular to flow. The section is normally trapezoidal in shape, cut into natural earth, and lined with a non-erodible material. The side slopes of a vegetated emergency spillway should be limited to 2H:1V (preferably 3H:1V) unless the spillway is cut into rock.

The exit section should have a straight alignment and grade, and at a minimum should retain the same cross sectional properties as the control section. The exit section should extend beyond the toe of the earthen embankment. The slope of the exit channel downstream of the control section should be steep enough to ensure that all significant flows (flows equal to or 25 percent greater than the peak discharge of the hydrograph used to design the basin) will be supercritical, but will not result in erosive velocities detrimental to the soil type and planned vegetation (supercritical flow may be difficult to achieve in certain areas of west Tennessee due to topography). The exit section slope should be designed to be as uniform as possible and should avoid major changes in grade which could induce hydraulic jump.

Excavation for inlet or exit channels can be omitted where natural grades meet minimum slope requirements.

8.04.7.4 HYDRAULIC DESIGN CONSIDERATIONS

The emergency spillway of all stormwater facilities will be designed for the 100-year/ 24-hour post-project peak flow rate from the proposed watershed area. The hydraulic analysis of the facility should include an additional check of the spillways ability to function independently. In the event of complete blockage or failure of the principal spillway device, the emergency spillway must be capable of passing the entire 100-year storm event without causing overtopping of the embankment. The final height of the embankment can be set based upon this computed high water elevation, plus required free-board.

The designer should set the invert of any emergency overflow spillway at a minimum elevation of 0.5 feet above the maximum 10-year high water elevation (principal spillway crest). The overflow spillway should be designed to pass the 100-year/24-hour peak flow at a suggested maximum depth of 1 foot.

The emergency spillway should be modeled as a broad-crested weir. It should be designed to safely pass the 100-year storm with a minimum of storage. The length of the emergency overflow weir can be determined by solving the weir equation for length, since flow and depth are known. Technical guidance and hydraulic design principles for broad-crested weirs is presented in Section 8.05.7.2.2.

For optimum hydraulic performance, the spillway layout should convey uniformly distributed flow across the entire spillway cross-section. The emergency spillway layout should not have flow concentrating features such as short radius curves and non-level sections perpendicular to the anticipated flow. The layout must ensure a predictable stage verses discharge relationship. To accomplish this, a control section should be used such that the inlet channel flow will be subcritical and the exit channel flow will be supercritical.

8.04.7.5 PERMISSIBLE VELOCITIES

The emergency overflow spillway should be protected from erosion by establishing vegetation or properly armoring the structure. The erosion protection may consist of properly selected vegetation, riprap, concrete, or other non-erodible surface. See Chapter 5 for the procedures for selecting vegetative lining materials for channel design. The procedure for selecting riprap for slope protection is presented in Section 5.04.7.1 of this Manual. Structural linings such as concrete will require special design and must be approved for use by the Design Manager. The maximum permissible velocity for a vegetated spillway should be selected using Table 8A-6 of the chapter Appendix.

8.04.8 STORAGE FACILITY LOCATION

The stormwater storage facility shall be located outside of the highway clear zone wherever possible. Permanent pool facilities will not be allowed within the highway clear zone. In cases where it is not possible to meet these requirements, adequate safety measures shall be incorporated into the roadway design. Facilities should additionally be located to minimize the introduction of off-site drainage which is typically not highway related. Drainage to the facility should originate as much as possible within the highway right-of-way.

The designer should attempt to locate all stormwater storage facilities *outside* of any FEMA identified flood plain or floodway. Where unavoidable, the designer should examine the adequacy of the facility to operate effectively during the 10-year storm event. Additionally, the structural stability of the stormwater facility components should be evaluated during the passage of the 100-year storm on the flood plain. The designer should study the possible impacts of the proposed basin on the 100-year floodplain and water surface elevations in conformance with FEMA requirements.

TDOT stormwater storage facilities shall *not* be located within a live-active perennial stream.

8.04.9 PERMANENT POOL FACILITIES

Use of permanent pool facilities will be limited, and should be considered only when special circumstances or requirements govern their use. When a retention basin is required to be used for stormwater quantity, quality, or both, the following general design criteria should be considered:

- Applicable criteria as established in Section 8.04.1 through Section 8.04.9
- Minimum hydraulic residence time of at least 2 to 4 weeks
- Storm storage above the permanent pool should be detained for 24 to 60 hours
- Basin should contain wetland vegetation to occupy a minimum of 25 percent of the surface area to improve removal of contaminants and reduce the formation of algae
- A 4 to 6-foot deep sediment forebay should be provided at each inlet. The forebay is intended to capture sediment deposition in an easily accessible area, and allow the heavier particulates to settle out prior to entering the permanent pool. The sediment forebay shall be separated (isolated) from the permanent pool by a stabilized overflow

spillway at the elevation of the permanent pool. The sediment forebay should be designed to contain 0.1 inches of runoff per impervious acre of contributing drainage area or sized at 10 percent of the required storm detention volume. Figure 8A-12 of the Appendix shows the layout of a typical sediment forebay in relation to the permanent pool facility. Figure 8A-2 shows a constructed sediment forebay at a permanent pool facility.

- Fixed sediment marker installed in sediment forebay to measure deposition
- Use of a clog resistant outlet device such as a perforated riser or v-notch weir should be considered for design
- Outlet device should be a staged riser, located within the embankment for maintenance accessibility
- Emergency drain device should be provided for permanent pool facilities to completely drain the basin
- Include applicable public safety features into final design of facility (Section 8.06.8)

The retention storage volume sizing procedure is similar to that for detention basins. For a retention reservoir, the reservoir will retain all of the runoff for the design storm. The designer will need to determine the required volume which will be retained. This will be based on the downstream or regulatory constraint that requires a volume of runoff to be retained in the basin. For water quality, it is required that the first ½ inch of runoff from the contributing impervious area be retained to capture the pollutants occurring in that initial runoff. See Section 8.04.3.2 for additional information on first flush requirements. This volume will be placed at the bottom of the reservoir. This will set the invert elevation of the primary control structure within the stormwater storage basin with all the retained storage located below this elevation.

8.04.10 UNDERGROUND DETENTION

The use of underground detention systems can add significant cost to a roadway project, and their use shall require approval of the design manager. Relatively expensive to construct, underground concrete vaults should be used primarily to control small flows in areas where system replacement costs would be high. Less expensive single or multi-pipe CMP or HDPE systems are typically used to control significant volumes of runoff from parking lots adjacent to the right-of-way, and in medians, where they can be replaced or maintained if necessary. The designer should also consider the following:

- Underground detention vaults and tanks must meet structural requirements for overburden support and traffic loading if appropriate
- The minimum diameter of pipe used for underground storage shall be 36 inches. The minimum outlet pipe from the system shall be 18 inches
- For a pipe system, the pipe segments shall be continuously sloped a minimum of 0.25 percent toward the outlet
- If environmental issues downstream necessitate achieving a level of water quality improvement, design measures must be taken to trap and store sediments in locations where cleanout and maintenance can be easily performed. This generally requires that some type of water quality inlet or other stormwater treatment BMP be installed upstream from the underground detention facility
- Significant water quality improvements should not be expected in underground detention facilities. They should mainly be used for providing storage to limit downstream effects due to high peak flow rates.

The construction cost and required maintenance of these systems are major considerations for design. The systems must be designed to have easy access for inspection and maintenance. In addition, underground systems will be considered confined spaces that require additional safety requirements for inspection and maintenance as per OSHA safety regulations.

For additional detailed design procedures and guidance on underground stormwater storage tanks, pipe systems, and vaults, the designer is directed to the following documents for additional design criteria:

- *Underground Detention Tanks as a Best Management Practice*, Northern Virginia Planning District Commission, 1992.
- *Alexandria Supplement to the Northern Virginia BMP Handbook*, Alexandria Department of Transportation and Environmental Services, 1992.
- *Rainwater Catchment Systems as a Best Management Practice*, Northern Virginia Planning District Commission, 1992.
- *Guide to the Selection and Design of Stormwater Best Management Practices (BMP's)*, Tennessee Water Resources Research Center, University of Tennessee, March 2003.
- *Design and Construction of Urban Stormwater Management Systems,* The Urban Water Resources Research Council of ASCE and the Water Environment Federation. ASCE, 1992.

SECTION 8.05 – DESIGN PROCEDURES

8.05.1 INTRODUCTION

This section provides general procedures for the design of stormwater storage facilities including an overview on volume estimation, discharge calculations, and routing computations. The design of a stormwater impoundment involves a variety of interrelated and complex factors including the relationships between stage and storage and stage and discharge which are necessary to perform the basic routing techniques. The design of a detention or retention basin normally begins with a level of overall system planning to establish a generalized design framework necessary to accomplish the desired goal. Once this has been accomplished, hydrological and routing computations can be performed to determine design flow rates from a stormwater basin that meet or exceed the objectives of the facility.

8.05.2 DESIGN DATA REQUIREMENTS

For proper storage design and routing calculations, the following data is required for analysis:

- Inflow hydrographs for the design storms required in Section 8.04.2
- Stage-storage curve for the proposed stormwater facility
- Stage-discharge curve for outlet control structures
- Allowable or desired release rates

Typically, only the inflow hydrographs are known at the beginning of design computations; therefore, a trial and error design procedure is required to complete the development the proposed facility.

8.05.3 GENERAL DESIGN PROCEDURE

The following steps should be used as the general procedure for sizing and designing a stormwater storage basin. Various steps in the design procedure will be described in greater detail in subsequent sections of this Manual.

Step 1: Develop and compute the inflow hydrographs for the 2-year, 10-year, and 100-year storm events using hydraulic procedures in Chapter 4 and methods found in Chapter 3 of *HEC-22* or SCS *Urban Hydrology for Small Watersheds (TR-55)*. The pre-development and post-development hydrographs are required for the 2 and 10-year storm events, while the 100-year storm event requires computing only the post-developed hydrograph. Each hydrograph shall be computed for the 24-hour storm duration.

Step 2: Compute a preliminary estimate of the required storage volume for the proposed facility. Storage volume estimating is an important task since the required volume necessary to achieve peak reduction is unknown and proper estimating will reduce the number of trials involved in the storm routing. Section 8.05.4 provides techniques to assist the designer with making initial volume estimates.

Step 3: Develop the layout and grading plan for the proposed facility consistent with the requirements of Sections 8.04.4 and 8.04.5. The maximum required storage volume obtained from Step 2 should be selected as the target volume to use for initial design of the basin.

Step 4: Using the proposed basin geometry, develop a stage-storage relationship for the facility. See Section 8.05.5 for further details on developing a stage-storage curve.

Step 5: Select and size the outlet structure(s) and their arrangement. Multiple outlet devices may be required and should be provided at appropriate levels within the basin. Trial and error analysis will be required to achieve the best outlet arrangement necessary to meet the level of control desired. The outlet structure should be sized to convey the maximum allowable discharge at the peak stage for the estimated volume calculated in Step 2. Multi-stage riser design procedures are provided in Section 8.05.8.

Step 6: Compute stage-discharge relationship for the outlet arrangement selected in Step 5. Details for developing a stage-discharge curve are provided in Section 8.05.6.

Step 7: Perform calculations by routing the inflow hydrographs developed in Step 1 through the proposed facility using methods presented in Section 8.05.9. If the post-developed peak discharges for the 2 and 10-year storms exceed the pre-developed peaks, adjustments must be made, to either, the basin geometry, storage volume, outlet arrangement, or manipulation of all three. Repeat Steps 3 to 7 until the discharge control criteria have been met.

Step 8: Design a secondary outlet (emergency spillway) consistent with the design criteria of Section 8.04.7. Perform routing calculations using the post-developed, 100-year/24-hour inflow hydrograph utilizing all outlet devices to determine the maximum water level in the facility. The basin performance under extreme conditions, such as partial or total blockage of the principal spillway, must be checked by performing routing calculations utilizing only the emergency spillway and the 100-year/24-hour storm. Freeboard requirements can now be established for the proposed facility based on the final computed water surface elevation.

Step 9: The potential for downstream flooding problems as a result of routing the storms through the facility should be evaluated by comparing the recession limb of both the pre-developed and post-developed hydrographs. If the maximum difference in hydrograph time and discharge rates increases significantly; a more detailed watershed analysis showing control of downstream impacts may be required.

Step 10: Evaluate the outlet velocity at the principal spillway. Provide energy dissipation or stabilization, as necessary. Proper design methods for energy dissipators can be found in Chapter 9 of this Manual.

8.05.4 PRELIMINARY DETENTION VOLUME COMPUTATIONS

8.05.4.1 STORAGE VOLUME ESTIMATES

Designing a stormwater storage facility is a trial and error process. The designer must select the minimum facility size with the most effective and proper outlet control devices by performing several iterations of routing computations. Each iteration will require the facility size (stage-storage relationship) and the outlet configuration (stage-discharge relationship) be evaluated and adjusted for performance against the requirements of the watershed. To accomplish the necessary or desired peak reduction of stormwater discharge, an estimate of the required volume of storage is an important first step for minimizing the trial and error process involved in the routing procedure. Procedures for preliminary detention calculations provide a simple method that can be used to develop initial storage estimates required to attenuate peak flows. Acceptable routing techniques should be used for final storage facility design.

8.05.4.1.1 HYDROGRAPH METHOD

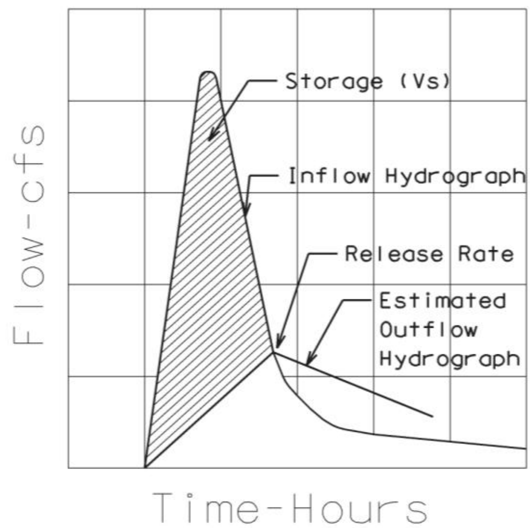


Figure 8-9
Hydrograph Method for Estimating Storage

The hydrograph method requires the designer to develop the inflow hydrograph for the stormwater basin, which is the watershed runoff to the facility. With the inflow hydrograph calculated and plotted, and a maximum release rate assigned, an estimated discharge curve can be estimated and plotted as shown in Figure 8-9. The peak of the estimated outflow hydrograph should always intersect the recession limb of the computed inflow hydrograph. The shaded area between the curves represents the initial estimate of storage volume which must be provided. Note, at this point the outflow hydrograph has been estimated by sketching an assumed outflow curve for the facility, with a peak discharge equal to or less than the desired facility outflow. Detailed storm routing, as described in Section 8.05.9, must be performed to establish the final outflow hydrograph for the facility.

8.05.4.1.2 TRIANGULAR HYDROGRAPH METHOD

For small drainage areas, a preliminary estimate of the storage volume required to attenuate the peak flow may be obtained from a simplified design procedure which replaces the actual inflow and outflow hydrographs with standard triangular shapes as shown in Figure 8-10. The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph. This volume is expressed as:

$$V_s = 0.5t_i(Q_i - Q_o) \qquad (8\text{-}4)$$

Where: V_s = estimate of storage volume, (ft^3)
Q_i = peak inflow rate, (ft^3/s)
Q_o = peak outflow rate, (ft^3/s)
t_i = duration of basin inflow, (seconds)
t_p = time to peak of the inflow hydrograph, (seconds)

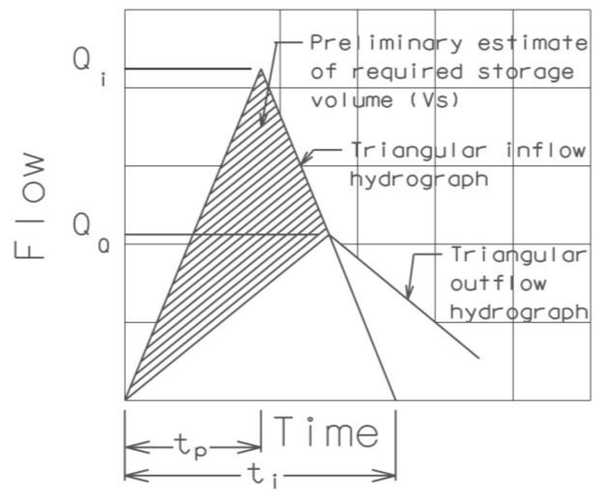


Figure 8-10
Triangular-Shaped Hydrograph Method

The triangular hydrograph method has been found to compare favorably with more complete design procedures involving reservoir routing. Any consistent units may be used for Equation 8-4.

8.05.4.1.3 MODIFIED RATIONAL METHOD (CRITICAL STORM DURATION)

The Modified Rational Method uses the critical storm duration for estimating the maximum required storage volume, V_s, for a stormwater detention basin. Unlike the conventional rational method which was meant for peak discharges and ignores the runoff volume produced by the storm event, the modified rational method evaluates different storm durations to determine which event will require the most storage volume with respect to the allowable release rate (the pre-project discharge). The storm duration that generates the greatest runoff volume is termed the critical storm, which consequently requires the most storage volume. This design storm duration will be the one that maximizes the storage volume for any given event.

The modified rational method will produce a family of hydrographs representing storms of different duration. The modified rational method assumes that the runoff hydrographs created are trapezoidal in shape, except for the triangular shaped hydrograph created for the post-project time of concentration, T_c. Rainfall averaging periods (durations), T_d, representing incremental time periods longer than T_c, are selected to create the trapezoidal hydrographs with peak discharges based on the intensity of the averaging period. The averaging period producing the greatest amount or required storage is the one with the most area between the inflow and outflow curves. The family of curves produced by using the modified rational method is shown in Figure 8-11.

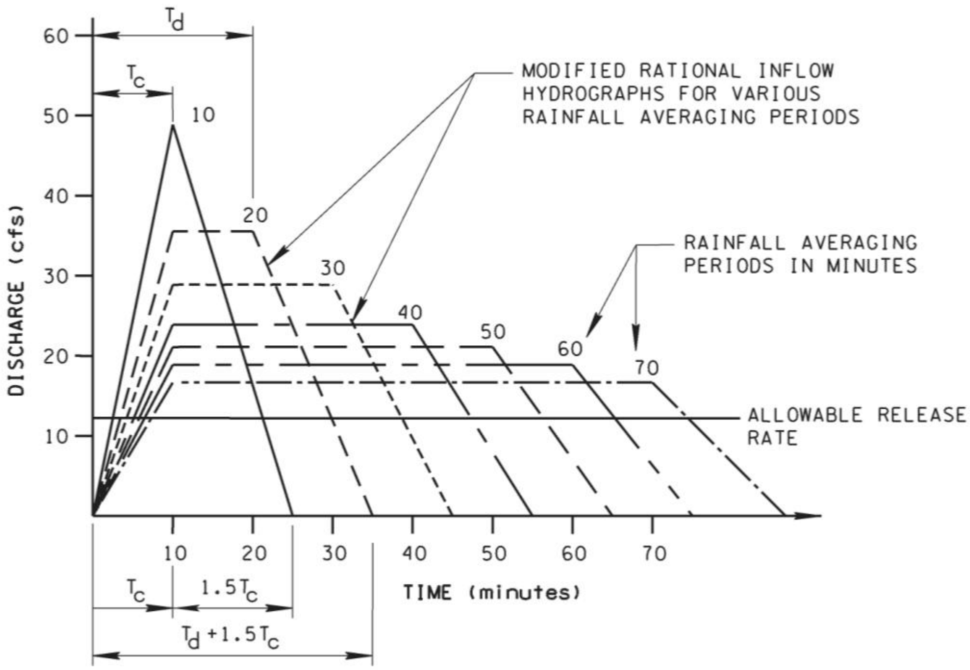


Figure 8-11
Modified Rational (Critical Storm Duration) Hydrographs

When estimating storage requirements using the modified rational method, the peak runoff rate produced by the storm event is first calculated using the conventional rational method as shown in Equation 8-5.

$$Q_i = CiA \tag{8-5}$$

Where: Q_i = peak inflow rate, (ft^3/s)
C = watershed runoff coefficient, (unitless)
i = design storm intensity, (in/hour)
A = watershed area, (acres)

With the peak storm discharge now calculated, the detention storage volume for each rainfall averaging period can be calculated using Equation 8-6. Averaging periods selected may be arbitrarily chosen; however, the designer should consider selecting time periods which coincide with times on the IDF-curve used for the project (see IDF-curves, Chapter 4).

$$V_s = Q_i T_d - Q_o\left(\frac{T_d + T_c}{2}\right) \tag{8-6}$$

Where: V_s = required storage volume, (ft^3)
Q_i = peak inflow rate, (ft^3/s)
T_d = duration of design storm, (seconds)
Q_o= allowable outflow rate, (ft^3/s)
T_c = time of concentration of watershed, (seconds)

The modified rational method has been shown to underestimate (10 to 20 percent) the required storage volume and should be used only for estimating purposes. This method assumes that the rainfall averaging period is equal to the actual storm duration, and the runoff that occurs before and after the averaging period is neglected. Its use is also generally limited to watersheds of 30 acres or less with times of concentration of 30 minutes or less. Additionally, detailed storage routing should be performed on the stormwater basin as the project plans develop. The Modified Rational Method produces a straight line approximation of the basin outflow; whereas, the actual shape of the outflow curve will vary depending on the outlet device configuration.

8.05.4.1.4 SCS (TR-55) METHOD

The SCS Tabular Method is a quick manual method for estimating required storage volumes and is based on average storage and routing effects of many structures. Estimating volume by this method does not require the generation of a hydrograph; rather, only the calculated peak rates of discharge, allowable basin release rate, and post-developed runoff be determined. A dimensionless figure relating peak outflow to peak inflow discharge, Q_o/Q_i, and storage volume to runoff volume, V_S/V_r, was developed and is illustrated in Figure 8-12.

The plotted curves on Figure 8-12 are based on the four NRCS developed 24-hour synthetic rainfall distributions (Types I, IA, II, and III) which were created from available National Weather Service duration-frequency data and local storm data for various regions of the country. Of the 4 distributions, Type IA is the least intense and Type II is most intense short duration rainfall. A rainfall distribution is used to describe what portion of the total rainfall depth falls during each portion of the total storm duration. Rainfall distributions are usually expressed as a graph of the percentage of total storm depth versus the percentage of total storm duration. Type II rainfall distribution applies to all of Tennessee (for additional information, see SCS 210-VI-TR-55, June 1986).

The procedure for using Figure 8-12 when estimating required storage volume, V_S is as follows:

1. Determine inflow, Q_i, and outflow, Q_o, discharges
2. Compute the ratio Q_o/Q_i
3. For the design storm, compute the inflow runoff volume, V_r, from Equation 8-7,

$$V_r = K_r Q_D A_m \tag{8-7}$$

Where: V_r = inflow volume of runoff, (acre-feet)
K_r = 53.33 (conversion factor for in-mi^2 to acre-feet)

Q_D= direct runoff depth, (inches)
A_m= watershed area, (square miles)

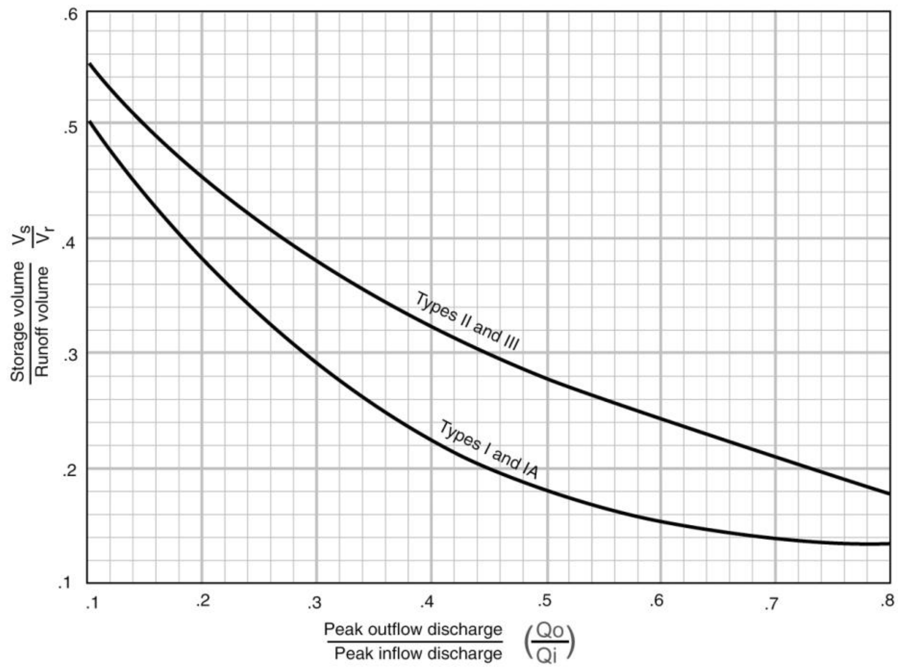


Figure 8-12
SCS Detention Basin Routing Curves
(Use Type II Rainfall Distribution Curve for Tennessee)

4. From Figure 8-12, determine the ratio V_s/V_r
5. The storage volume, V_s, can then be determined by Equation 8-8

$$V_s = V_r\left(\frac{V_s}{V_r}\right) \tag{8-8}$$

The SCS tabular procedure for storage volume estimating should not be used if an error in storage volume of 25% will not be acceptable. Therefore, it should only be used for preliminary design estimates of large detention structures. Other methods for calculating detention storage volume should be used for verification and final design of the stormwater storage facility.

8.05.4.1.5 ALTERNATIVE REGRESSION EQUATION METHOD

When a specified reduction in peak flow is required, an estimation of the required volume of storage may be obtained using the following equation by Wycoff & Singh (1986):

$$\frac{V_s}{V_r}=\frac{1.291\left(1-\frac{Q_o}{Qi}\right)^{0.753}}{\left(\frac{t_b}{t_p}\right)^{0.411}} \tag{8-9}$$

Where: V_s = storage volume, (in)
V_r = runoff volume, (in)
Q_i = peak inflow rate, (ft^3/s)
Q_o = peak outflow rate, (ft^3/s)
t_b = time base of the inflow hydrograph (hours), determined as the time from the beginning of rise, to a point on the recession limb where the flow is percent of the peak
t_p = time to peak of the inflow hydrograph (hours)

To apply this method for estimating storage volume, the following steps should be followed:

Step 1: Determine the volume of storm runoff in the inflow hydrograph, V_r, including the allowable peak outflow rate, Q_o, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

Step 2: Calculate a preliminary estimate of the ratio, V_s/V_r, using the input data from Step 1 and Equation 8-9.

Step 3: Multiply the volume of runoff, V_r, times the ratio, V_s/V_r, calculated in Step 2 to obtain the storage volume required to avoid exceeding the allowable peak discharge rate.

8.05.4.2 PEAK FLOW REDUCTION

When the available volume is known, and an estimate of the peak discharge is desired, a preliminary estimate of the peak flow reduction for the known volume can be computed by solving Equation 8-9 in terms of discharges. The equation then becomes:

$$\frac{Q_o}{Q_i}=1-0.712\left(\frac{V_s}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546} \tag{8-10}$$

Where: All variables are the same as in Equation 8-9.

A preliminary estimate of the potential peak flow reduction for a known storage volume can then be determined by the following method:

Step 1: Determine the volume of storm runoff in the inflow hydrograph, V_r, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, the time to peak of the inflow hydrograph, t_p, and the storage volume, V_s.

Step 2: Calculate a preliminary estimate of the potential peak flow reduction for the known storage volume using Equation 8-10, above.

Step 3: Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated in Step 2, to obtain the estimated peak discharge rate for the selected storage volume.

8.05.5 STAGE-STORAGE CURVE

A stage-storage volume curve should be developed in one foot increments (or less) from the proposed bottom of the basin to three feet above the design high water elevation in the storage facility or the maximum embankment elevation. Figure 8-13 depicts a sample stage-storage curve. The curve represents the relationship between the storage volume and the depth of water at a particular elevation within the basin. After initial layout and grading of the facility has been performed based on estimated required storage volume, the designer can compute the volume of storage at any stage within the facility by applying the geometric formulas found in the following sections which are expressed as a function of head, H.

For open basins, the average end area should be calculated using horizontal slices at every 1-foot change in elevation (or some other easy to use increment). For enclosed systems or roadside ditches, the average end area should be calculated using vertical sections spaced no more than 100 feet apart. Sections for the average end area should be incorporated wherever there is a significant change in facility shape or structure size.

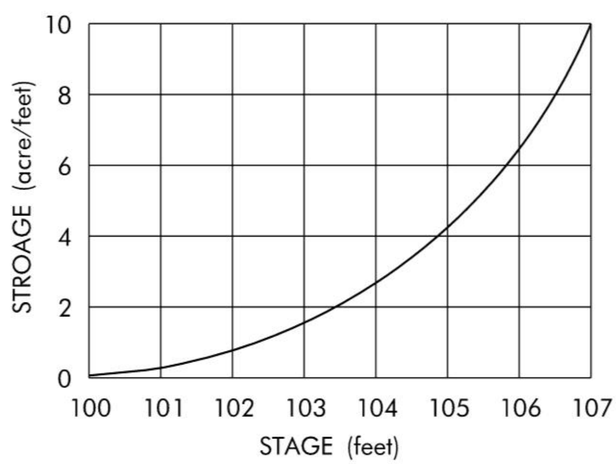


Figure 8-13
Sample Stage-Storage Curve

8.05.5.1 NATURAL BASIN VOLUMES

The storage volume for natural basins in irregular terrain may be computed using a topographic map and the double-end area method, frustum of a pyramid, or circular conic section formulas. Figure 8-14 shows the terms of the double end area method. The double-end area method formula is expressed as:

$$V_{1,2} = \left(\frac{A_1 + A_2}{2}\right)d \qquad (8\text{-}11)$$

Where: $V_{1,2}$ =storage volume between elevations 1 and 2, (ft^3)
A_1 =surface area at elevation 1, (ft^2)
A_2 =surface area at elevation 2, (ft^2)
d =change in elevation between A_1 and A_2, (ft)

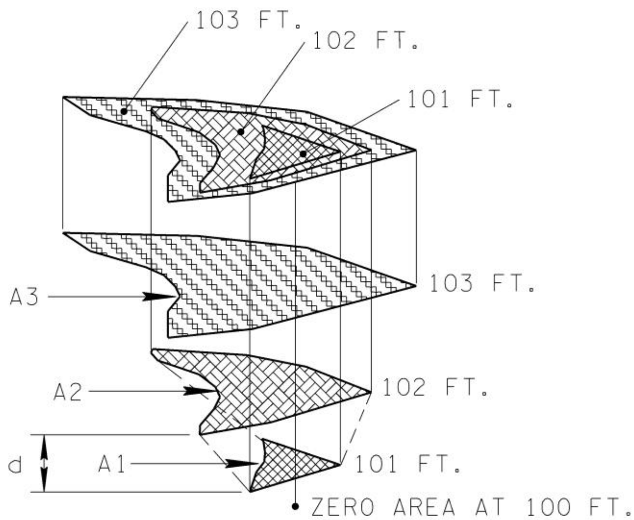


Figure 8-14
Double-End Area Method

The frustum of a pyramid is shown in Figure 8-15 and can be expressed by the equation:

$$V = d\,\frac{A_1 + (A_1 A_2)^{0.5} + A_2}{3} \qquad (8\text{-}12)$$

Where: V =volume of frustum of pyramid, (ft^3)
A_1 =surface area at elevation 1, (ft^2)
A_2 =surface area at elevation 2, (ft^2)
d =change in elevation between A_1 and A_2, (ft)

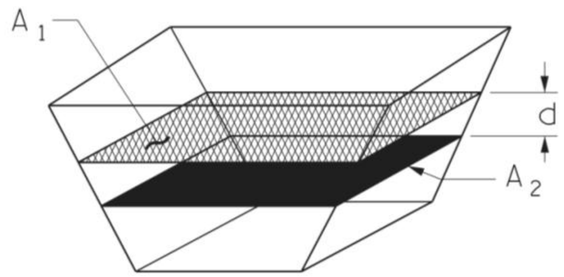


Figure 8-15
Frustum of Pyramid Method

The volume of a trapezoidal basin, as represented in Figure 8-16, can be computed by dividing the basin into rectangular and triangular components and applying the prismoidal formula expressed as:

$$V = LWD + (L + W)ZD^2 + \frac{4}{3}Z^2D^3 \qquad (8\text{-}13)$$

Where:
- V =volume of trapezoidal basin, (ft^3)
- L =length of basin at base, (ft)
- A =width of basin at base, (ft)
- D =depth of basin, (ft)
- Z =side slope factor, ratio of horizontal to vertical (i.e. 3H:1V, Z=3)

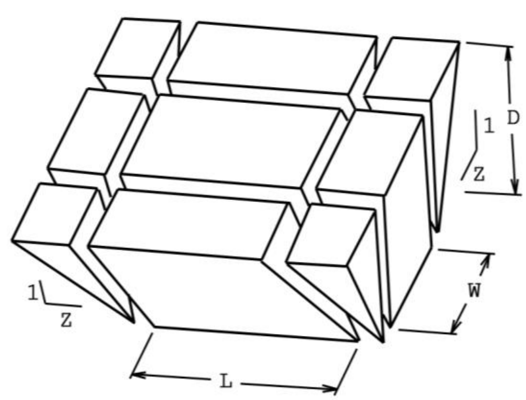


Figure 8-16
Trapezoidal Basin

8.05.5.2 ARTIFICIAL BASIN VOLUMES

The volume of an artificial rectangular basin (i.e. an underground vault) on a slope can be computed by dividing the basin into triangular and rectangular components and applying the following two equations:

$$V = L \times W \times D \qquad \text{(rectangular)} \qquad (8\text{-}14)$$

$$V = 0.5W \frac{D^2}{S} \qquad \text{(triangular)} \qquad (8\text{-}15)$$

Where: V = available volume at a specific depth, (ft^3)
D = depth of ponding, (ft)
L = basin length at base, (ft)
W = basin width at base, (ft)
S = basin slope, (ft/ft)

Figure 8-16 depicts the terms of Equations 8-14 and 8-15. Obviously, in cases where the facility is not provided with a bottom grade, the volume will be easily computed from the rectangular geometry of the facility. In most cases, underground vaults should be designed with a bottom slope to avoid deposition of sediment and facilitate cleaning of the structure.

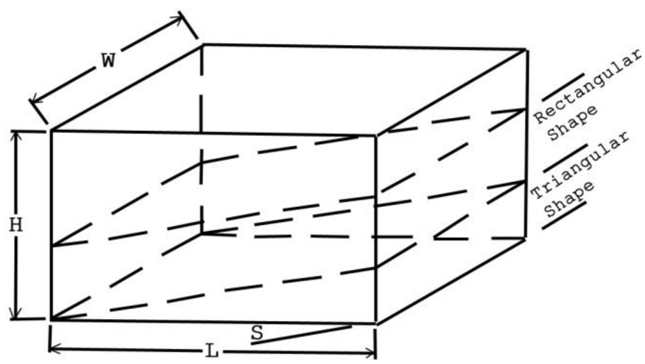


Figure 8-17
Rectangular Basin

For underground pipe and conveyance system detention, the volume of a circular pipe storage facility on a uniform slope can be computed by the ungula of a cone formula expressed as:

$$V = H \frac{\frac{2}{3}a^3 \pm cB}{r \pm c} \qquad (8\text{-}16)$$

Where: V = volume of stormwater in pipe, (ft^3)
H = wetted length of pipe, (ft)
B = wetted cross sectional area of base, (ft^2)
a = ½ of the water surface width, (ft)

r = radius of the pipe, (ft)
c* = distance from the center of pipe to the water surface, (ft)

*Use + (positive ‘c’-value) when the water surface is above the center of the pipe
Use – (negative ‘c’-value) when the water surface is below the center of the pipe

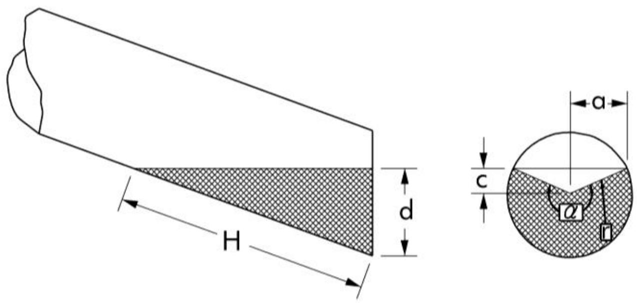


Figure 8-18
Ungula of Cone

In Equation 8-16, the value of “a” can be calculated using:

$$a = \left(r^2 - c^2\right)^{0.5} \qquad (8\text{-}17)$$

and, α in Figure 8-18 is computed by the formula:

$$\alpha = 2\,sin^{-1}\left(\frac{a}{r}\right) \qquad (8\text{-}18)$$

Table 8A-8 can be used to assist the designer in determining area B in the cross section of a circular conduit flowing part full.

It should be stressed that the ungula of a cone formula is for a circular sloping storm system on a uniform slope. If this is not the case, the storage volume of each segment of the system on a uniform slope should be computed using the prismoidal formula. The sum of the computed pipe segment volumes should then be calculated to determine the total available storage volume.

The prismoidal formula can be used to compute the volume of a sloping storm pipe (or ditch) in a linear stormwater storage system which is sloping toward a positive outlet. The prismoidal formula for this application is expressed as:

$$V = \frac{L}{6}\left(A_1 + A_2 + 4M\right) \qquad (8\text{-}19)$$

Where:
V = volume of stormwater in pipe (or ditch), (ft^3)
L = wetted length of pipe, (ft)
A_1 = wetted cross sectional area of lower end of pipe, (ft^2)

A_2 = wetted cross sectional area of upper end of pipe, (ft^2)
M = wetted cross sectional area of midsection of pipe, (ft^2)

Figure 8-19 shows the terms of the prismoidal formula.

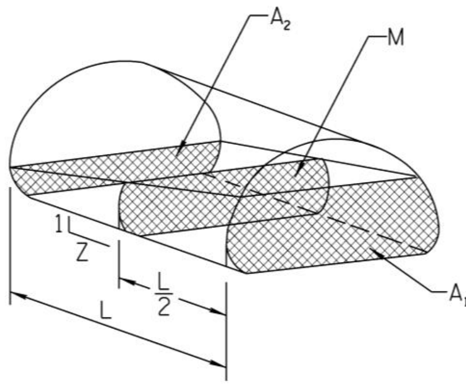


Figure 8-19
Definition Sketch for Prismoidal Formula

8.05.6 STAGE-DISCHARGE CURVE

The stage-discharge curve is a graphical representation of the relationship between the depth of water in a storage facility and the discharge from the facility. A typical stormwater storage facility should have both a principal spillway and an emergency overflow spillway. The principal spillway should be designed to sufficiently convey all design discharges without the use of the emergency spillway. Principal spillways typically consist of a single pipe culvert through an embankment or a multi-stage riser and pipe configuration with the riser containing a series of orifices, weirs, or other outlet works. Multi-stage risers are typically required to control multiple frequency storms.

Regardless of the spillway configuration, a composite stage-discharge curve must be developed for the proposed stormwater basin by considering the discharge rating relationship for each component of the outlet works and combining them into one curve. A typical composite stage-discharge curve for a stormwater facility is shown in Figure 8-20.

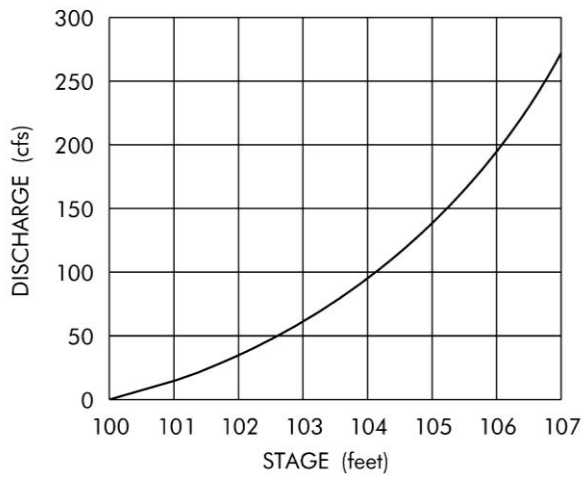


Figure 8-20
Sample Stage-Discharge Curve

The following sections provide the designer with the basic relationship of outflow for commonly used outlet structures and the computational procedures for determining the outflow based on head.

8.05.7 OUTLET HYDRAULICS

8.05.7.1 ORIFICES

An orifice is the most common type of outlet control structure used to regulate discharge from a stormwater facility. Although round is the most often used shape, orifices can be square, rectangular or triangular. The discharge rate through an orifice is dependent upon the height of the water surface above the orifice, the opening size, and edge treatment of the orifice. Regardless of the orifice shape, the orifice will act as a weir initially, until the orifice is completely submerged. Until the orifice becomes submerged, the discharge through the opening at the lower stages is calculated using the broad-crested weir equation (see Section 8.05.7.2.2). Once submerged, the equation for determining discharge through a single orifice can be computed from the standard orifice equation.

$$Q = C \times A \times (2gH)^{0.5} \qquad (8\text{-}20)$$

Where: Q = discharge, (ft^3/s)
C = orifice coefficient, (Varies, 0.60 recommend for use)
g = gravitational acceleration, (ft / sec^2)
H = orifice head, (ft)
A = flow area, (ft^2)

Studies by Brater and King (1976) found that the orifice coefficient varies based on size, shape, and head, but a recommend average coefficient of 0.6 should typically be used for stormwater analysis, provided the orifice is square-edged with uniform entrance conditions.

Beveling the orifice edge or field coring a concrete riser can have a considerable impact on the orifice coefficient.

A sharp-edged orifice under ideal conditions can provide coefficients of:

- C=0.66 when orifice diameter is equal or greater than the thickness of riser
- C=0.80 when orifice diameter is less than the thickness of riser

For orifices that are to be field-cut into corrugated metal pipe resulting in a ragged edge, the designer should use a C-value of 0.4. (FHWA, HEC-22)

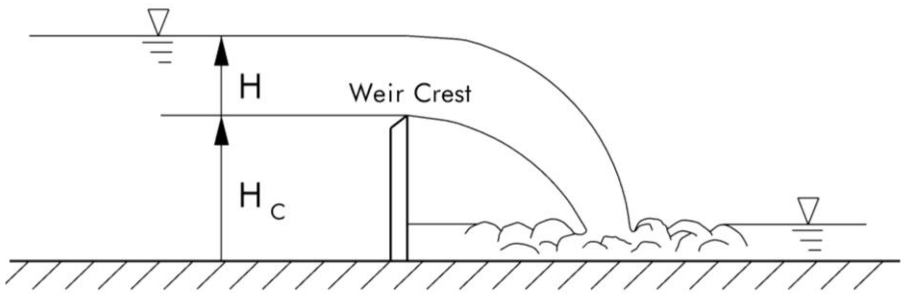
For Equation 8-20 to be correctly applied, the flow area must be fully submerged. The orifice head is computed as the difference in elevation between the water surface and the centroid of the orifice (for free-fall exit conditions) or the controlling tailwater elevation (submerged outlet conditions); whichever is greater. To calculate the flow through multiple orifices, the designer shall sum the discharge through each individual orifice.

Figure 8A-13 of the chapter appendix shows the relationship between weir flow and orifice flow for a typical application of an orifice in the wall of an outlet structure.

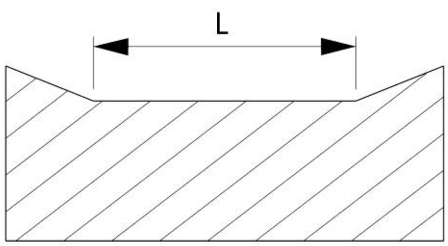
8.05.7.2 WEIRS

8.05.7.2.1 SHARP-CRESTED WEIRS

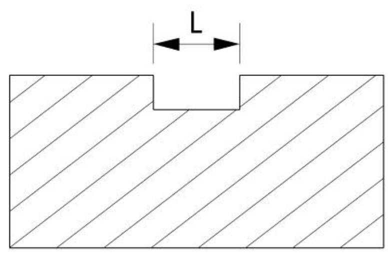
A sharp crested weir allows a nappe to form beneath the discharge and therefore provides more free flow characteristics. Sharp crested weirs are typically used for flow measurement because of the stable stage-discharge relationship they provide. Sharp crested weirs can have no end contractions or may have both end contracted.



Profile of a Sharp Crested Weir



Sharp Crested Weir, No End Contractions



Sharp Crested Weir, Two End Contractions

Figure 8-21 Typical Sharp-Crested Weirs

A sharp-crested weir with no end contractions is shown in Figure 8-21. The flow over the top edge of a riser pipe is typically treated as flow over a sharp-crested weir with *no* end contractions. The equation to compute discharge for this configuration is:

$$Q = \left(3.27 + 0.4\frac{H}{H_c}\right)LH^{1.5} \tag{8-21}$$

Where: Q = discharge, (ft^3/s)
H = head above weir crest excluding velocity head, (ft)
H_C = height of the weir crest above the channel bottom, (ft)
L = horizontal weir length, (ft)

The discharge over a sharp-crested weir with *two* end contractions can be determined by the equation:

$$Q = \left(3.27 + 0.4\frac{H}{H_c}\right)(L - 0.2H)H^{1.5} \tag{8-22}$$

Where: The variables are the same as in Equation 8-21.

A sharp-crested weir will be affected by submergence (see Figure 8-22, below) when the tailwater rises above the weir crest elevation. When this occurs, the result will be that the discharge over the weir will be reduced. To account for the submergence, and the reduction in discharge, the free-fall discharge obtained by either Equation 8-21 or Equation 8-22 shall be adjusted using the following equation developed by Brater and King:

$$Q_s = Q_f\left(1-\left(\frac{H_2}{H_1}\right)^{1.5}\right)^{0.385} \tag{8-23}$$

Where: Q_s = submerged flow, (ft^3/s)
Q_f = unsubmerged free flow, (ft^3/s)
H_2 = downstream head above crest, (ft)
H_1 = upstream head above crest, (ft)

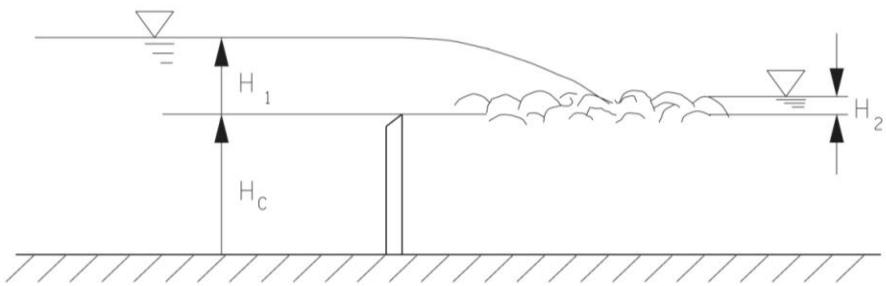


Figure 8-22
Submerged Sharp-Crested Weirs

8.05.7.2.2 BROAD-CRESTED WEIRS

The most common type of weir structure associated with stormwater storage is the broad crested weir as illustrated in Figure 8-23. Broad crested weirs can be defined as a raised channel control crest section with a nearly horizontal crest sufficiently long in the flow direction that the nappe is supported at least for a short distance. The water surface draws down onto the weir until critical depth forms. The flow regime over an emergency spillway of a detention storage facility can be considered to be replicating flow over a broad crested weir.

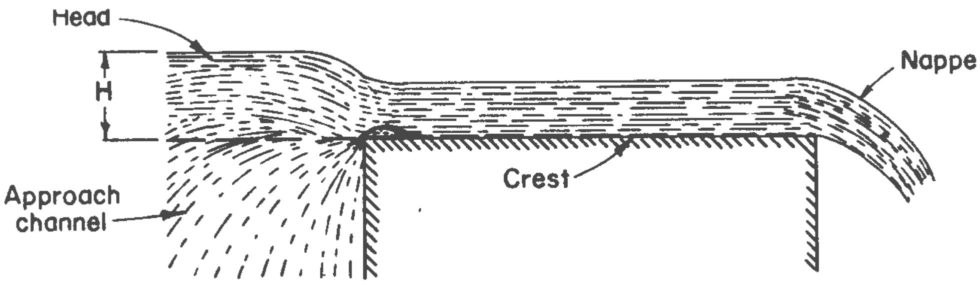


Figure 8-23
Broad-Crested Weir

Equation 8-24 is used to compute discharge over a broad crested weir is expressed as:

$$Q = C \times L \times H^{1.5} \tag{8-24}$$

Where: Q = discharge, (ft^3/s)
C = broad-crested weir coefficient, (2.61 to 3.08)
L = length of weir crest, (ft)
H = head over weir crest, (ft)

H is the head of water above the weir crest at some distance upstream usually measured a minimum distance of 4H upstream from the weir. Typically, there is no defined approach channel for a detention basin, therefore, the velocity head is generally assumed to be zero. With that, H in Equation 8-24 becomes the difference in elevation between the spillway crest elevation and the water surface elevation within the detention basin.

Broad-crested weir coefficients range from 2.61 to 3.08. The coefficient for H = 1 foot and a width of 10 feet would be 2.68. 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 head loss due to friction, flow will pass through critical depth at the weir crest; therefore, giving a maximum coefficient, C of 3.08. For sharp corners on a broad-crested weir, a minimum coefficient of 2.6 should be used. A table of weir coefficients for various combinations of head and width is shown in Table 8A-1 of the chapter Appendix.

8.05.7.2.3 V-NOTCH WEIRS

The V-notch weir design causes small changes in the discharge to have a large change in depth; therefore allowing greater accuracy in head measurement than with a rectangular weir. The discharge through a V-Notched weir as shown in Figure 8-24 can be calculated from the following equation developed by Brater and King (1976):

$$Q = 2.5 \tan\left(\frac{\theta}{2}\right) H^{2.5} \tag{8-25}$$

Where: Q = discharge, (ft^3/s)
Θ = angle of V-notch, (degrees)
H = head on apex (lowest point) of v-notch, (ft)

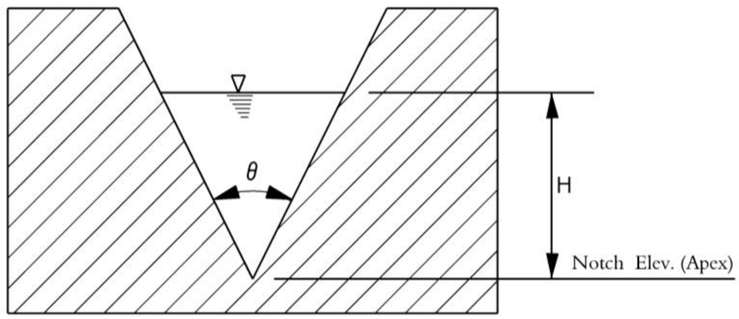


Figure 8-24
V-Notch Weir

8.05.7.2.4 PROPORTIONAL WEIRS

The use of a proportional weir may significantly reduce the required stormwater storage volume for a given site. With the proportional weir, a linear head-discharge relationship is achieved by allowing the discharge area to vary non-linearly with head. Figure 8-25 depicts a simple proportional weir. The proportional weir may be more difficult to design and construct than other weirs; therefore, its use should be limited.

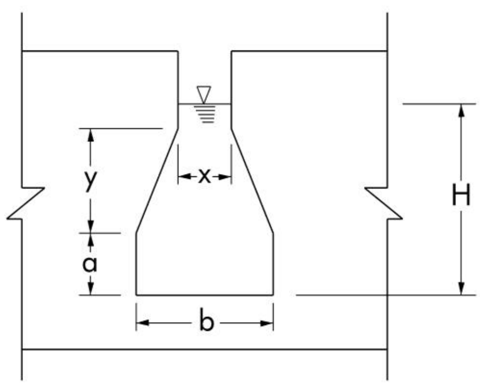


Figure 8-25
Proportional Weir

The equations used to calculate the flow over a proportional weir are:

$$Q = 4.97a^{0.5}b\left(H - \frac{a}{3}\right) \tag{8-26}$$

and,

$$\frac{x}{b} = 1 - \frac{1}{3.17}\arctan\left(\frac{y}{a}\right)^{0.5} \tag{8-27}$$

Where: Q = discharge, (ft^3/s)
H = head over weir crest, (ft)
Dimensions a, b, h, and y are shown in Figure 8-25

8.05.7.3 PIPES AND CULVERTS

Where a single pipe or culvert will be used as the outlet control structure for the storage facility, the design procedures in Chapter 6 or those outlined in *HDS-5* should be followed. The use of culverts alone, should be limited since the culvert is acting as the primary release structure and, in the event of blockage, the only alternative overflow point is the emergency spillway. Also, attenuating the peak storm runoff for the 2-year through 10-year design storms may be difficult with a stand-alone pipe or culvert installation. Due to compaction limitations, which can potentially lead to seepage and piping along the outer edge of a culvert, multiple pipe barrels through an embankment will not be allowed by the Department as the primary outlet structure unless special conditions necessitate their use.

Culvert design and analysis should be performed using the methods outlined in Chapter 6 of this Manual. Additionally, the designer shall review FHWA's *HDS-5* to gain an understanding of the hydraulics of culvert flow. For design purposes, if a culvert is used as a “stand alone” outlet control device in a detention basin, the culvert will be required to operate under inlet control. In lieu of using the nomographs in *HDS-5*, the following two equations can be used to determine the discharge from a culvert flowing full under inlet control (i.e. not subjected to backwater). It is left to the user to solve the equation for Q, the discharge. Table 8A-3 of the chapter Appendix provides inlet control coefficients to be used in the following equations.

The weir flow equation used to compute discharge for a culvert with an <u>unsubmerged</u> entrance is:

$$\frac{HW}{D} = \frac{H_c}{D} + K\left(\frac{Q}{AD^{0.5}}\right)^M \tag{8-28}$$

Where: HW = headwater depth above the culvert invert, (ft)
D = inside diameter of the culvert barrel, (ft)
H_c = specific head at critical depth, (ft)
Q = culvert discharge, (ft^3/s)
A = full cross-sectional area of culvert barrel, (ft)
S = slope of culvert barrel, (ft/ft)
K = unsubmerged Coefficient (see Appendix Table 8A-3)
M = unsubmerged Coefficient (see Appendix Table 8A-3)

The orifice flow equation used to compute discharge for a culvert with a <u>submerged</u> entrance is given below. Substitute +0.7S for the slope correction factor instead of -0.5S when a mitered inlet is used.

$$\frac{HW}{D} = C\left(\frac{Q}{AD^{0.5}}\right)^2 + Y - 0.5S \quad (8\text{-}29)$$

Where:
- HW = headwater depth above the culvert invert, (ft)
- D = inside diameter of the culvert barrel, (ft)
- H_c = specific head at critical depth, (ft)
- Q = culvert discharge, (ft^3/s)
- A = full cross-sectional area of culvert barrel, (ft)
- S = slope of culvert barrel, (ft/ft)
- C = submerged coefficient (see Appendix Table 8A-3)
- Y = submerged coefficient (see Appendix Table 8A-3)

Equation 8-28 (un-submerged) applies for values of $Q/AD^{0.5}$ up to 3.5. For Equation 8-29 (submerged), the equation applies for values of $Q/AD^{0.5}$ of 4.0 and greater. For values of $Q/AD^{0.5}$ between 3.5 and 4.0, linear interpolation shall be used from the two boundary conditions to find the value of HW of the culvert.

8.05.8 MULTI-STAGE RISER DESIGN

Controlling both the 2-year/24-hour and the 10-year/24-hour storm events may require the design of a multi-stage control structure. Typically this is accomplished by providing a drop-inlet principal spillway consisting of a multi-stage riser with a configuration of orifices and weirs at differing elevations. By providing this arrangement, the designer will be able to control the discharge from the facility for multiple storm events.

The designer should attempt to provide the most hydraulically efficient and economical outlet structure possible which will meet the desired flow reduction criteria of the basin and which will minimize the size of the reservoir. Many iterative routing attempts are typically required to achieve these desired goals. For each routing attempt, the basin trial size and outlet works must be tested for hydraulic performance as multiple storms are routed through the facility. A stage-storage curve and stage-discharge curve will be created for each of the routing attempts and should be revised as the designer makes adjustments to either the basin volume or the outlet devices and then re-routes the storms through the facility.

The following procedure provides guidance on the proper design of a multi-stage stormwater riser. It should be noted that one or more steps may be omitted depending on site specific requirements or the method of storm routing (hand or computer solution).

Step 1: Determine the allowable release rates. The allowable release rates from a proposed stormwater storage facility should be determined based on the criteria provided in Section 8.04.2 of this Manual. This will require that the pre-developed and post-developed hydrographs of the watershed be computed. In many cases the pre-developed runoff hydrograph will provide a peak rate of discharge which will set the maximum allowable discharge rate from the basin. In other cases, downstream conditions and capacities of the receiving conveyance system may govern the allowable release rate.

Step 2: Determine the Water Quality Volume (only where required). Compute the required water quality volume, V_{wq}, necessary to meet the "first flush" requirements of Section 8.04.3.2. If water quality measures are not required, proceed to Step 3.

Step 3: Estimate the Required Storage Volume. To reduce the number of routing iterations to be performed at a later time, the designer should estimate the required stormwater storage using one of the methods described in Section 8.05.4. For each storm event, the post-developed runoff hydrograph, peak rate of runoff, and the allowable outflow will be required to compute a reasonable estimate of the required storage volume. Where required, the computed water quality volume (Step 2) should be added to the total required storm storage volume. Note that the computed volume is only an estimate and is intended to provide the designer with a basis for beginning the facility design.

Step 4: Develop a Stage-Storage Curve for the Facility. A stage-storage curve (see Figure 8-13) should now be generated based on the proposed geometry and grading of the basin. The basin geometry should be consistent with the requirements of Section 8.04.4, and its location determined by considering site geometry, site topography, available land area, and subsurface conditions. The basin should be sized by trial and error to provide the required storage calculated in Step 3, measured from the absolute lowest elevation in the facility. During the basin grading process, the designer should also initially consider freeboard requirements by providing a preliminary grading design which will allow for additional grading of the embankment and/or surrounding areas to meet these requirements.

Step 5: Design the Water Quality Outlet Device (where required). When required, a water quality outlet device should be designed to meet the drawdown requirements discussed in Section 8.04.3.2. Water quality outlet devices can be designed as a single small orifice, perforated pipe, or other acceptable device which meets the drawdown requirements. With V_{wq} computed from Step 2, the average hydraulic head, H_{avg}, acting on the device can be computed from the stage-storage curve using the elevation corresponding to the required, V_{wq}, and the invert of the device. The average discharge from the water quality device to meet drawdown time requirements can be computed by the equation:

$$Q_{avg} = \frac{V_{wq}}{t_{dd}(3600)} \tag{8-30}$$

Where: Q_{avg} = average discharge, (ft^3/s)
V_{wq} = water quality volume, (ft^3)
t_{dd} = draw down time, (hrs)
Note: 3600 is used to convert drawdown time in hours to seconds

If an orifice is used as the outlet device, the orifice equation can be rearranged to solve for area, and further solved for diameter using Q_{avg} and H_{avg} in the rearranged equation, given as Equation 8-31. Additional computations and storm routing should be performed on the trial orifice size with adjustments being made as necessary to meet drawdown time requirements. Where a perforated pipe or other device is used, the designer should seek other reference material for computing the discharge through such devices.

$$A = \frac{Q}{C\left(2gH\right)^{0.5}} \tag{8-31}$$

Where: Q = discharge, (ft^3/s)
C = orifice coefficient, (Varies, 0.60 recommend for use)
g = gravitational acceleration, (32.2 ft / sec^2)
H = orifice head, (ft)
A = flow area, (ft^2)

Step 6: Design the 2-year Storm Control Device. With the allowable release rate, inflow hydrograph, and stage-storage relationship known, a preliminary 2-year outlet structure can be designed for the riser. From the stage-storage curve, approximate the maximum water surface corresponding to the 2-year storage requirements. This will provide the maximum head, H_{max}, to apply to the orifice or weir equations when developing the stage-discharge curve. If water quality is required, the maximum head will be computed as the 2-year water surface elevation minus the elevation corresponding to the water quality volume, V_{wq}; otherwise, the head is the approximate water surface elevation minus the bottom of the reservoir. (Note, when water quality measures *are* required, the 2-year storm storage computations should begin at the V_{wq} elevation).

With allowable discharge known from Step 1, and rearranging the orifice or weir equations depending on the structure used, the dimensions of the outlet control device can be calculated. The weir equation will apply to an orifice until the structure is fully submerged. The designer shall apply the weir equation at lower stages of discharge until the water surface is above the orifice (see Figure 8A-13 of the Appendix). A stage-discharge curve can now be developed for the 2-year control device by calculating the corresponding discharge for each elevation used on the curve.

Step 7: Evaluate the 2-year Storm Control Device Performance. Verification should be made that the maximum release rate from the 2-year control device meets the requirements for the given facility. The designer should route the 2-year/24-hour storm through the basin using the methods described in Section 8.05.9 or by using an acceptable computer program. Minor adjustments to the trial structure size may be required or additional storage volume should be added to the facility to meet the release criteria.

Step 8: Design the 10-year Storm Control Device. Using procedures similar to Step 6, the 10-year outlet control structure invert should be set a minimum of 0.1 feet above the computed 2-year maximum water surface elevation. Determine the approximate 10-year maximum water surface elevation from the stage-storage curve developed in Step 4. Estimate the head, H, on the structure by subtracting the invert elevation of the structure from the estimated maximum water surface elevation. Also determine the discharge through the 2-year control device for the estimated 10-year water surface elevation. The maximum discharge through the 10-year control device would then be equal to the 10-year basin allowable release rate minus the discharge through the 2-year control device.

With the maximum discharge determined above, the size of the 10-year control structure can be determined by rearranging the orifice or weir equations and solving for the geometric properties. With an approximate size determined, a stage-discharge curve can now be developed for the 10-year control device by calculating the corresponding discharge for each elevation used on the curve.

Step 9: Evaluate the 10-year Storm Control Device Performance. Verification should be made that the maximum release rate from the 10-year control structure meets the discharge requirements for the given facility. The designer should route the 10-year/24-hour storm through the basin using the methods described in Section 8.05.9 or by using an acceptable computer program. Minor adjustments to the trial structure size may be required or additional storage volume should be added to the facility to meet the release criteria.

Step 10: Evaluate Hydraulic Performance of the Riser and Outlet Barrel. Multi-stage riser structures require a hydraulic analysis be performed by the designer to ensure all outlet structures will operate as intended. To optimize the hydraulics of the outlet structures and the riser-barrel system, several iterations may be required as part of the evaluation. The discharge through a drop-inlet spillway can be classified as being either riser flow control or barrel flow control. Riser flow control can be further categorized into riser-weir flow control or riser-orifice flow control. Likewise, barrel flow control can be further described as barrel flow with inlet control or barrel flow with pipe control. The classifications can be illustrated by Figure 8A-14 of the Appendix and are discussed in the following paragraphs.

<u>Riser Flow Control</u>

As storm discharge increases beyond a 10-year event, water will begin to pass over the top of the riser. At this point, hydraulically, the riser is acting as a weir. At this stage, the structure is in riser-weir flow control and the discharge is computed using the weir equation (see Section 8.05.7.2.2). As the water surface elevation continues to increase over the riser crest, the flow regime will transition to riser-orifice flow control, whereas the riser is acting hydraulically as a submerged orifice. The elevation at which the riser transitions from riser-weir flow to riser-orifice flow must be computed during the design of the structure, and is usually during conditions of high hydraulic head. The following steps should be considered necessary for checking the riser flow control regime:

- Using the inside circumference or perimeter of the structure, the flow over the riser should be computed using the weir equation
- The orifice equation (Section 8.05.7.1) should be used to determine the discharge through the riser opening. Area, A, in the orifice equation will be the interior area of the opening at the top of the riser
- The controlling flow at any given stage will be the smaller of the two computed values

<u>Barrel Flow Control</u>

With barrel inlet flow control, the capacity of the outlet pipe is dependent upon the pipe diameter, entrance shape and the headwater over the pipe measured from the water surface elevation to the pipe invert. Hydraulically, the outlet pipe will be acting as a submerged orifice, whereby the discharge through the entrance can be computed using the orifice equation or the inlet control nomographs contained in FHWA's HDS-5. Use of a particular nomograph will be dependent on the type of pipe material and the shape of the entrance. To calculate the discharge during barrel inlet flow control, the designer should:

- Determine the barrels entrance condition

- Determine HW/D for each elevation specified on the stage-discharge curve
- Determine the hydraulic capacity of the barrel using the orifice equation or the inlet control nomographs from *HDS-5*

When barrel pipe control exists, the capacity of the outlet pipe is dependent upon the pipe slope, length, internal roughness, and possible tailwater at the pipe exit. Full flow within the pipe will exist for all or part of the barrel length. To calculate the discharge during barrel pipe flow control, the designer should:

- Determine the discharge for each elevation specified on the stage-discharge curve using the pipe flow equation:

$$Q = A\left(\frac{2gH}{1K_m K_p L}\right)^{0.5} \tag{8-32}$$

Where: Q = discharge, (ft^3/s)
A = flow area of Barrel, (ft^2)
g = gravitational acceleration, (32.2 ft / sec^2)
H = elevation head differential, (ft)
L = barrel Length, (ft)
K_m = coefficient of minor losses, ($K_e + K_b$)
K_e = entrance loss coefficient, (See Table 6A-6, Chapter 6 Appendix)
K_b = bend Loss Coefficient, (use 0.5 for riser-barrel system)
K_p = coefficient of pipe friction, where: $K_p = 5087(n^2) / D^{1.333}$
n=Mannings roughness, D= inside pipe diameter. (USDA, NEH-5)

H is measured from the water surface elevation in the basin to the centerline of the pipe or the tailwater elevation, whichever is greater.

- Compare the outlet control discharges with the inlet control discharges for the barrel. The controlling flow at any given stage is the smaller of the two discharges.

Conditions of barrel flow control are desirable because the flow regime will minimize or eliminate surging, vibration, and cavitational forces acting on the structure. These adverse conditions will occur when the riser is flowing full and restricting flow to the outlet conduit, whereby the barrel will not be experiencing full flow conditions. If the riser flow transitions from weir flow to orifice flow, before barrel flow control begins, the negative conditions noted above will occur. The multi-staged riser MUST be designed so that the barrel flow control will govern prior to the riser transitioning from weir to orifice flow.

Step 11: Design the Emergency Spillway. An emergency spillway should be provided at all stormwater facilities to prevent overtopping of the embankment and the potential for subsequent failure. The emergency spillway will pass significant volumes of discharge with little effect on head. The design analysis is similar to that of the 10-year storm and should be in conformance with the design criteria established in Section 8.04.7. Additional design data for earth spillways can be found in Table 8A-4 of the chapter Appendix. Where an emergency spillway will not be provided, the designer shall size the principal spillway to accommodate the peak discharge resulting from the 100-year/24-hour storm event.

Step 12: Compute Total Discharge from the Facility. Combine the computed flows from the previous steps and generate a composite stage-discharge curve. At some elevation, the total flow from ALL outlet devices will exceed the capacity of the outlet conduit. The total outflow from the facility will be the lesser of the total discharge through all outlet structures, or the controlling flow though the barrel and riser. The designer should verify that the total computed discharge is in conformance with allowable release rates for the design storms.

Additionally, the designer should verify that the release rates for each storm event are not set *too* low, whereby unnecessary additional storage will be provided over that which is necessary (see Section 8.04.2). The designer should also determine the maximum high water elevation and ensure that the freeboard criteria provided in Section 8.04.5 are met.

Step 13: Design Outlet Protection for Anticipated Discharges. Design necessary outlet devices or energy dissipation measures at the discharge points of the facility (see Section 8.04.6.7). With the discharges known for the full range of design storms, energy dissipation can be designed to prevent localized scour and reduce the velocity of the concentrated discharge. Design of such measures shall be in conformance with the procedures presented in Chapters 6 and 9 and may range from a simple rip-rap apron to more complex concrete structures.

Step 14: Perform Buoyancy Calculations on Riser. When the water surface in the basin is above the foundation of the riser and the ground is saturated, the riser is subject buoyant forces relative in strength to the displaced water volume. The riser will float when the weight of the structure (downward force) is less than the buoyant force exerted by the water (upward force). As a general rule, the downward force should be at least 1.25 times the upward force. Since the weight of concrete (approx. 150 lb/ft^3) is over twice the weight of water (approx. 62.4 lb/ft^3), adverse buoyancy effects can easily be overcome by adding more concrete to the footing and burying the riser and base deeper, whereby increasing the downward earth load on the extended footing (unit weight of soil approx. 120 lb/ft^3).

The following procedure should be used to perform buoyancy calculations:

- Compute the total upward force (buoyant force, F_b) being exerted on the riser. This buoyant force is computed as the total volume displacement of the structure, including the base (footing), times the unit weight of water. This may be computed simply by using Equation 8-33. The volume must be calculated using the outside dimensions of the structure.

$$F_b = \left(V_{riser} + V_{ftg}\right)\left(62.4\ lb/ft^3\right) \quad (8\text{-}33)$$

- Determine the total downward resisting force, F_r, being exerted by the structure. This force is the total volume of the structure walls and footing times the unit weight of the concrete. Additional weight from trash racks, anti-vortex baffles, and the weight of the soil bearing on the footing may be added to the total weight. The opening for the principal spillway pipe and all orifices and weirs shall be deducted from the computed concrete volume. F_r may be computed using Equation 8-34.

$$F_r = Weight_{conc} + Weight_{soil} + Weight\,of\,all\,other\,components \qquad (8\text{-}34)$$

Step 15: Provide Seepage and Piping Countermeasures. Seepage and piping control should be provided for the outlet pipe passing through an embankment. The designer may use anti-seep collars or filter and drainage diaphragms to adequately control the flow of water along the outlet conduit. The use and design of either type of countermeasure shall be in accordance with the design criteria discussed in Section 8.04.6.4 and by referencing other technical documents.

8.05.9 ROUTING COMPUTATIONS

Routing computations shall be performed at all required stormwater facilities to be incorporated as part of a TDOT design project. Storm routing is the process of tracing the flow of stormwater through a hydrologic system or reservoir, and can be described as being either hydraulic or hydrological. Hydraulic routing uses distributed system methods whereby the flow is computed as a function of both space and time throughout the system being analyzed.

Hydrological routing methods calculate flow as a function of time, only, and are typically adequate to evaluate stormwater storage facilities. Based on the continuity equation, the goal of hydrologic routing through a stormwater reservoir is to create an outflow hydrograph resulting from the combined effects of the storage available and the outlet device(s) hydraulics. In simple terms, the continuity equation relates the inflow, I, outflow, O, and rate of change in storage, S, as shown by the following equation:

$$I - O = \frac{\Delta S}{\Delta t} \qquad (8\text{-}35)$$

Where: I = inflow rate, (ft^3/s)
O = outflow rate, (ft^3/s)
$\Delta S / \Delta t$ = Storage change over time, (ft^3/s)

When performing reservoir routing computations, watershed hydrographs have been generated, therefore the inflow will be known. However, Equation 8-35 cannot be solved directly for outflow because both outflow and storage are unknown. A second relationship is necessary to further relate I, O, and S, which when coupled with the continuity equation, will provide the designer with a solvable mathematical relationship consisting of two equations and two unknowns. The Storage Indication method described in the following section provides this required second correlation by relating the storage and outflow to the water surface (level pool) within the reservoir.

8.05.9.1 STORAGE INDICATION METHOD

The Storage Indication method (also referred to as “level-pool” method) of storm routing is the most common method of reservoir routing and is typically suitable for highway drainage projects. The Storage Indication routing technique is based on the continuity equation for unsteady flow (Equation 8-35), assumes a horizontal water surface within the reservoir, and is used for calculating an outflow hydrograph from a detention basin given the inflow hydrograph and storage-discharge characteristics.

For storm routing by this method, storage, S, is a non-linear function of the outflow, O, and can be determined by relating available facility storage, and its outflow, to the water surface elevation in the reservoir. When a facility's water surface is known and considered horizontal, the total storage and outflow are a function of the pool depth at any given time. The outflow can be computed from the head acting on the outlet device(s) at a given time using the hydraulic principles relating head and discharge for various outlet works presented in Section 8.05.7. The following procedure should be used to perform reservoir routing by the Storage Indication Method:

Step 1: Develop an inflow hydrograph, a stage-storage curve, and a stage-discharge curve for the proposed detention facilities. A complete inflow hydrograph, not just the peak discharge, must be generated. Example stage-storage and stage-discharge curves are presented in Figures 8-13 and 8-20, respectively.

Step 2: Select a time interval for routing (time step), Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$). Generally, a routing interval of one-tenth the time to peak will be adequate. Normally, for small watersheds, the routing interval will be less than an hour, usually from 0.25 to 0.5 hours.

Step 3: Use the stage-storage and stage-discharge data from Step 1 to develop storage characteristic curves that provide values of $S \pm O\Delta t / 2$ versus stage. An example of storage characteristic curves is shown in Figure 8-26, a tabulated example of storage characteristics curve data is shown in Table 8-2.

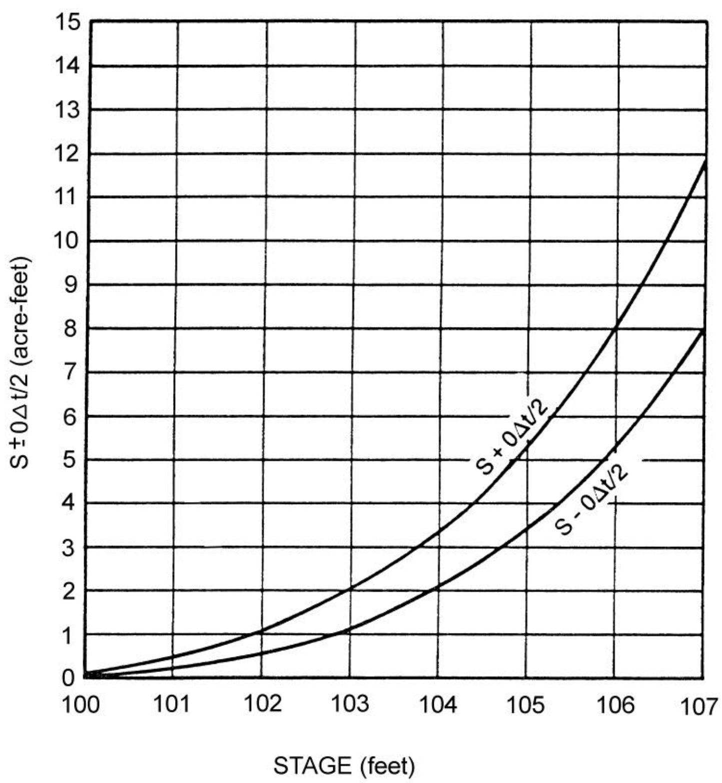


Figure 8-26
Storage Characteristic Curves (*example, only*)

STAGE – H (FEET)	STORAGE – S (ACRE-FEET) [A]	DISCHARGE-Q (CFS) [B]	DISCHARGE-Q (AC-FT / HOUR)[B]	S_1 - $O_1\Delta T$ / 2 (ACRE-FEET)	S_1 + $O_1\Delta T$ / 2 (ACRE-FEET)
100	0.05	0.00	0.00	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98

[a] Storage determined from stage-storage curve
[b] Discharge determined from stage-discharge curve

Table 8-2
Storage Characteristic Curve Data (*example-only*)

Step 4: For a given time interval, inflows I_1 and I_2 are known from the previously generated inflow hydrographs. Given that the depth of storage or stage, H_1, at the beginning of that time interval, $S_1 – O_1\Delta t / 2$ can be determined from the appropriate storage characteristics curve (e.g., Figure 8-26).

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

$$S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \quad (8\text{-}36)$$

Where: S_2 = storage volume at time 2, (ft^3)
O_2 = outflow rate at time 2, (ft^3/s)
Δt = routing time period, (seconds)
S_1 = storage volume at time 1, (ft^3)
O_1 = outflow rate at time 1, (ft^3/s)
I_1 = inflow rate at time 1, (ft^3/s)
I_2 = inflow rate at time 2, (ft^3/s)

Step 6: Enter the appropriate storage characteristics curve at the value of $S_2 + O_2\Delta t / 2$ determined in Step 5 and read off a new depth of water, H_2.

Step 7: Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve (see sample curve Figure 8-20).

Step 8: Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1, and H_1 equal to the previous I_1, O_2, S_2, and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin. The values obtained in Steps 4 through 8 should be tabulated for convenience, as shown in Table 8-3. The designer should then verify that the maximum discharge from the reservoir is less than the allowable release rate, and that the discharge is not considerably less than the allowable release rate. Should this occur, a different outlet size, arrangement, or basin shape should be considered and the computations repeated.

1	2	3	4	5	6	7	8
Incremental Time (hours) [a]	**Inflow (cfs)**	**$(I_1+I_2)\Delta t/2$ (ft^3)**	**Stage[b], H_1 (feet)**	**$S_1 - O_1\Delta t/2$ (ft^3)**	**$S_2 + O_2\Delta t/2$ (ft^3)**	**Stage, H_2 (feet)**	**Outflow, O_2 (cfs)**
From Inflow Hydrograph		*Use Columns 1 and 2*	*From Stage-Storage Curve*	*From Storage Char. Curve at H_1*	*Col 3 + Col. 5 or Equation (8-34)*	*From Storage Char. Curve at Col. 6 Value*	*From Stage-Discharge Curve*
0.00	0	0	0.00	0	0	0.00	0.0
0.17	38	11400	0.00	0	11400	0.65	9.7
0.33	125	48900	0.65	5560	54460	2.21	40.9
0.50	190	94500	2.21	29920	124420	3.58	81.5
0.67	125	94500	3.58	75503	170003	4.23	106.2[c]
0.83	70	58500	4.23	106313	164813	4.17	103.2
1.00	39	32700	4.17	102900	135600	3.76	87.3
1.17	40	23700	3.76	83214	106914	3.30	72.5

[a] Convert time to seconds for unit consistency

[b] Depth at the beginning time interval from column 3

[c] Peak discharge at maximum elevation in basin. Compare with allowable.

Table 8-3
Storage Indication Hydrograph Routing Worksheet
(Values in Table Shown for Example Only)

Step 9: Plot the inflow and outflow hydrographs. The peak outflow from the facility should always coincide with a point on the receding limb of the basins inflow hydrograph. A sample plot of both routed storm hydrographs is shown in Figure 8-27. In the figure, target flow represents the maximum allowable release rate from the facility. The shaded area represents the volume of stormwater to be stored in the reservoir and released over time. The time distribution of the storm runoff is altered by the storage facility, even though the total volumes of the inflow and outflow hydrographs are the same.

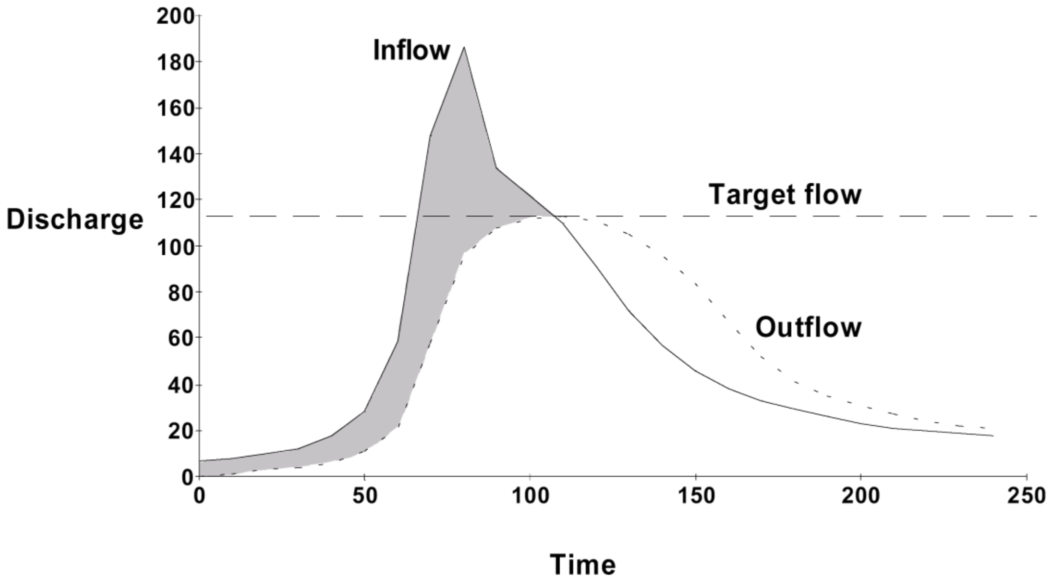


Figure 8-27
Routed Hydrographs (*example, only*)
Reference: USACE, HEC-HMS Tech. Ref. Manual, (2000)

From the previous steps, it can be seen that the routing procedure for a storage facility is a trial and error process that can be cumbersome and time consuming. This is especially true as the designer attempts to create the most efficient and cost effective basin shape and outlet arrangement at the proposed facility. Computer programs are available (and recommended) to assist the designer with reservoir design and routing computations. The programs allow the user the ability to easily change outlet works, basin shapes, and re-route the storm hydrographs through the facility in a matter of minutes, whereas hand computations may take in excess of hours. Most programs will plot computed hydrographs and provide the user with tabulated reports for project documentation. Section 8.07 provides a brief discussion and overview of available computer models for stormwater storage design.

SECTION 8.06 – MAINTENANCE AND CONSTRUCTION CONSIDERATIONS

8.06.1 INTRODUCTION

A stormwater storage facility will prove effective only if it is designed correctly, constructed properly, and maintained regularly. The proper maintenance of a detention basin is necessary to obtain as good of a performance as was anticipated from the design. A facility that is difficult to maintain will not provide the required storage and water quality benefits. In addition, the maintenance of all elements of the storage facility should be considered as an entire entity rather than maintaining each element separately.

To assure acceptable performance and function, detention storage facilities requiring frequent inspection and maintenance are discouraged by the Department. Maintenance can become a substantial annual cost statewide; therefore, it is important to consider and incorporate maintenance concerns into the initial design of the facility. This potentially large annual expense should be thoroughly considered in the planning and design phase of the stormwater storage facility. Maintenance costs can be kept to a minimum by careful design of the basin, outlet structures, spillways, and any adjoining amenity areas. This requires a firm understanding of hydrological, hydraulic, and structural design principals in addition to operational and maintenance requirements. Proper design and construction should focus on the reduction or elimination of potential maintenance requirements.

8.06.2 GENERAL INSPECTION CRITERIA

Inspection of a storage facility is an essential aspect of a detention basin maintenance plan. Annual inspection of detention facilities may reveal minor issues which need correcting or could uncover serious structural problems which could compromise the overall integrity of the facility. Inspection is vital for early detection of potentially serious and dangerous conditions which could be developing over extended time periods at a stormwater storage basin. Early detection and repair are necessary to protect the integrity of the facility and avoid possible costly reconstruction activities. Proper inspection is important and cannot be adequately performed by driving by at 30 miles per hour and seeing if the basin is there.

To aide with inspection activities, an access road to the basin and a turn-around facility should be considered during the design of the facility. These measures should be designed with the anticipation that heavy equipment may utilize the access in the future. Access roads and turnaround areas should be properly graded and compacted to sustain heavy equipment traffic. In addition, access to the bottom of the basin should be provided for maintenance vehicles and personnel. This can easily be accomplished by limiting the facility side slope adjacent to the access road to 4H:1V. When detention basin areas are not accessible directly from the road right-of-way, an access easement shall be provided to the facility.

8.06.3 EMBANKMENTS

Proper design, construction and inspection of a stormwater storage facility embankment are necessary to insure that the facility is functioning as planned. All storage facility earth embankments should be visually inspected annually for signs of erosion, burrowing animals, and seepage. This inspection should be done after mowing of the embankment so that the slope is not obscured from inspection by tall grass and brush. The earth embankment should be inspected for signs of erosion after any storm event where water has flowed over the emergency spillway.

The engineer should design a facility embankment which will minimize the need for costly maintenance and repairs. Carefully selected features during the design of the facility may minimize or eliminate the need for future repair activities. In general, embankments should be designed with a minimum top width of 8 feet and side slopes a minimum of 3H:1V or flatter. Where safety is a concern and fencing is not practical, side slopes of 4H:1V or flatter should be considered. Freeboard of 1 foot or more should be provided above any expected high water elevation in the facility (Section 8.04.5.3). If right-of-way is available, shallow basins with large surface areas should be considered instead of short deep basins, thus minimizing the embankment height. A homogeneous embankment with seepage control zones or a zoned embankment shall be designed for structures greater than 15 feet in height.

For permanent pool facilities, embankment protection measures should be considered to guard against erosion caused by wave action. Bank protection should be in the form of protective linings such as rip-rap, engineered vegetation, soil stabilization, and in extreme cases, armoring or paving.

8.06.4 CONTROL STRUCTURES

Litter and debris blocking the outlet control structure is a common problem with most stormwater storage facilities. Control structures should be inspected monthly and after any storm event greater than two inches for accumulated debris. The primary outlet, principal spillway, and emergency overflow structures should be inspected for damage, clogging and sediment accumulation. The drawdown device of an extended detention facility should also be inspected for clogged orifices, or excessive sediment buildup.

All control structures should be designed to minimize the potential for maintenance and repair. Considering potential maintenance problems associated with a control structure is important throughout the design process. Small outlet pipes, which may frequently block, are discouraged. A principal spillway pipe which minimizes or eliminates joints should be considered for use under the embankment. Drawdown devices and perforated risers should be protected with filter socks and gravel filter barriers to reduce the potential for clogging from sedimentation.

A trash rack should be provided at all control structures (may be omitted for some weirs) to minimize the likelihood that the outlet will be blocked by debris. Wherever possible, the minimum opening size for an outlet structure should be four inches (does not apply to perforated risers). The trash rack should be designed to capture all litter that is larger than one half the size of the smallest dimension of the outlet structure. The trash rack should have a total opening area that is equal to or greater than ten times the outlet opening. Trash racks should be constructed with a sloping face, which will allow floating debris to rise with the water level in the basin.

Access should be provided to the outlet control structure when the stormwater storage facility is in operation to clean litter and other floating debris from the control structure and trash rack; however, this may not always be feasible. The access should be such that maintenance personnel do not have to enter the water to clean the structure. At a minimum, the control structure should be reachable by a long handled rake or other device to remove the debris. For underground storage, this may require more than one access manhole at the control structure.

8.06.5 SEDIMENTATION AND DEBRIS CONTROL

Sediment removal is an important maintenance activity for a stormwater storage facility. Sediment will accumulate over time in any stormwater storage facility. Accumulated sediment in a basin reduces the desired storage volume of the basin and also may negatively impact the pollution removal efficiency of the basin. Sediment must be limited or controlled for a detention facility to operate as designed. Measures taken during design can minimize the effects of sedimentation on the facility. Where sedimentation does occur, proper maintenance is essential for the facility to continue to function as designed.

The facility designer should consider certain design elements which, over the life of the basin, will minimize the need for maintenance resulting from excessive sediment load. Excessive sediment can cause the drawdown device to become partially clogged or possibly rendered completely inoperable. Standing water which is unable to “bleed down” from the facility due to a clogged drawdown device will result in the facility becoming an eyesore and a breeding ground for mosquitoes. Sediment control devices at the inlet of the facility should be considered. Energy dissipation devices such as rip-rap aprons at the terminus of inlet ditches and storm pipes will minimize the potential for erosion of the facility bottom. A forebay or plunge pool will serve as an initial sediment trap for the storm runoff. A forebay will facilitate maintenance activities by isolating the sediment deposition in an accessible area. As a minimum, the forebay should be designed to approximately 10 percent of the required detention volume of the main basin. A sediment forebay is vital and recommended where permanent pool facilities are planned due to difficulty in cleaning or dredging sediment from a live pool.

For storage facilities where access for maintenance will be difficult or infrequent, the storage facility should be oversized by 10 percent to ensure that sufficient volume is available. In areas where extensive sediment accumulation can be expected due to surrounding land uses, the facility should be oversized by 25 percent to ensure that sufficient volume is available. If extensive sediment loads are estimated throughout the life of the facility, the designer should consider installing a permanent sediment marker within the basin indicating depth of sediment requiring removal.

The basin will need to be cleaned periodically to maintain the desired design stormwater storage volume. Cleaning of accumulated sediment from sediment control devices may be as frequent as every 5 to 10 years. As discussed in Section 8.04.4, access to the stormwater storage facility for removing accumulated sediment should be incorporated into the facility design.

Where underground storage is used, access will be required to periodically flush the accumulated sediment from the facility. Access will be required to each pipe and the header pipe if multiple pipes are used. The access must be large enough for maintenance personnel and their equipment to safely enter the facility. The bottom of the underground facility should have sufficient slope so that the material can be flushed with water to a sump near the outlet. The sump should have a sufficient opening to allow easy removal of the accumulated debris.

Debris must be removed from a detention basin so it does not become an impediment to the normal operation of the facility during a storm event. Floating debris such as litter, tree limbs, lumber, and leaves can be controlled by installing trash racks at outlet structures. Prior to mowing, special attention should be given to removing debris from around the inlet of the control structures. To facilitate cleanout and reduce the potential for debris accumulation, trees and shrubs should be cleared from the immediate detention facility area, and in no terms should hardwood trees be specified or planted on or near the embankment area.

8.06.6 INSECT & ODOR CONTROL

A properly designed stormwater management facility can provide effective quantity and quality control of highway runoff. A poorly designed or constructed facility can potentially become a nuisance to the public and a maintenance problem for the Department. Perpetual standing water or saturated areas within or adjacent to the facility may result in an unmanageable public eyesore and most likely will become a habitat for the proliferation of mosquitoes and other insects. A poorly drained basin or one with saturated side slopes will promote unwanted vegetation growth and hinder inspection, mowing and maintenance operations. In addition, standing water may result in the growth of algae which can produce unwanted odors.

The design of proposed stormwater storage facilities should include measures to avoid potential constant saturated areas. Standing water or otherwise soggy conditions within the facility can be minimized by incorporating the following countermeasures into the basin design:

- Top of embankment shall be graded in a manner to avoid the possibility of standing water or puddles. Insure positive drainage by crowning or sloping the top of embankment with a grade of approximately 5 percent
- Bottom of detention facility should be sloped toward the outlet structure to minimize the potential for standing water. Bottom slope shall be a minimum of 0.5% to a maximum of 2%.
- Constructing a concrete, low flow pilot (trickling) ditch from the inlet to the outlet (see Figure 8A-3). Due to short circuiting possibilities, this method should not be used where water quality is an objective unless protective measures are taken.
- Underdrain system to water table or outlet device where necessary at detention or extended detention facilities
- At permanent pool facilities, consider a 1 foot drop-off between the normal water surface and the safety bench (see Figure 8-7)

In addition to these measures, areas adjacent to the facility shall be graded to insure positive drainage away from or toward the basin as necessary.

Within permanent pool facilities, wetland vegetation may be encouraged to improve the aesthetic qualities of the facility and reduce the growth of floating algae by limiting the amount of nutrients available for phytoplankton. Wetland vegetation should be small species with a low potential of obstructing the principal spillway or other outlets. The Ecology Section of the Natural Resources Office in the Environmental Division should be contacted for assistance.

8.06.7 WEED AND GRASS MAINTENANCE

Weed growth and grass maintenance can easily be controlled by providing a facility with slopes which will allow conventional tractor mowers or bush hogging operations to be accomplished with relative ease. Side slopes should be no steeper than 3H:1V to provide safe and easy access for mowing and inspection operations. Interior and exterior corners of the basin shall be rounded to provide a safe and acceptable turning radius for maintenance equipment.

Regular mowing encourages the growth of a thicker and more erosion resistant ground cover and minimizes the potential for the facility to become an eyesore to the public. Prior to mowing the facility, debris removal and litter pickup should be performed as part of the annual maintenance plan. Access to the storage facility should be provided as described in Section 8.04.4. Where fencing is present, an access gate must be provided for routine mowing operations.

8.06.8 PROTECTIVE TREATMENT FOR STORMWATER FACILITIES

Public safety around a stormwater storage basin should be a foremost consideration when designing a proposed detention basin. Accordingly, a detention basin must be designed with public safety in mind when the facility is in operation. Grading, fencing, signage, and safety grates should all be considered during the design process. The engineer should weigh the necessity and benefit of protective devices against the cost of such safety features. An evaluation should be performed to study the relative risks and dangers of a detention facility and develop a design which will provide the best balance of safety measures at the site. The following is a list of protective treatments to consider when evaluating a design:

- Grading – Minimize side slopes as much as practical. If fencing is necessary but not feasible, consider lessening maximum side slopes to 4H:1V or flatter. Provide a flat safety bench around the edge of a permanent pool facility consistent with Section 8.04.4.4
- Signage – Signs such as “No Trespassing” may be considered. Signs may have a negative impact by attracting unwanted attention of area children.
- Guardrails – Consider installing guardrail adjacent to roadways where errant vehicles could enter the facility, especially if facility is designed as a permanent pool facility or the dry basin is designed to have a stormwater depth greater than 3 feet
- Grates and Trash Racks – Open grates bolted to the top of riser structures should be considered to prevent entry. Grates will prevent persons or large debris from being swept into a long or submerged drainage system
- Fencing – Perimeter fencing will typically not be required at most stormwater detention basins. Fencing may limit or obstruct maintenance operations, can create an attractive nuisance to the public, and has been proven <u>not</u> to provide a formidable barrier to deter unauthorized entry; rather, it tends to draw attention to the facility. However, fencing should be considered, and may be required in the following circumstances:

  - The design stormwater stage within the basin is greater than 3 feet deep for a normal storm event
  - The facility will have side slopes equal to 2H:1V, or steeper
  - The facility has been designed as a permanent pool facility and is adjacent to a residential area, school, park, etc…
  - The facility is in close proximity to a school, daycare, recreational sports complex, park, shopping center or other facilities where children will normally be present
  - Facility will be adjacent to or along a pedestrian school route
  - Site is located within a municipality which will provide maintenance for the facility at the completion of the roadway project and has requested the fence installation at the basin

Protective fencing at stormwater detention facilities will typically not be warranted within a limited access highway right-of-way and will not be required along a controlled access route except under special circumstances.

SECTION 8.07 – ACCEPTABLE SOFTWARE

8.07.1 INTRODUCTION

The computer applications for stormwater storage facility design described in the following sections may be used for the hydraulic analysis and design. The Tennessee Department of Transportation does not endorse any specific software package. Further, TDOT does not warrant the results that may be obtained from any of these computer programs. Evaluation of the results given by any specific computer program and the stormwater facility design are entirely the responsibility of the user.

Models use the computational power of the computer to expedite the time consuming and repetitive computations normally associated with stormwater routing. The models analytical power and computational speed gives them superior advantages over manual calculations.

The designer is advised that no computer method is completely reliable, and the results from any software package should be carefully evaluated. A clear understanding of the various hydraulic methods involved in storage facility design is therefore necessary to successfully apply any of these programs. The software packages described below should be used unless a special circumstance on a project requires the use of other software. The TDOT Design Manager should approve the use of any other software for these special circumstances. The designer should refer to the software manuals for further guidance on how to apply the reservoir routing methods provided for in the selected software.

8.07.2 NRCS TR-20 PROJECT FORMULATION HYDROLOGY

Technical Release 20 (TR-20) was developed by the NRCS to conduct hydrographic analysis of watersheds. The TR-20 program is capable of computing direct runoff hydrographs for a large number of subareas within a watershed and is able to add these hydrographs together and route them through a network of channels or reservoirs. Other capabilities include an option to input a hydrograph computed by another source and the ability to “split” a hydrograph by diverting water at given location.

The program offers a variety of options regarding the input of rainfall data and is able to model hypothetical storm events (such as the NRCS Type II rainfall distribution discussed in Chapter 4) as well as actual rainfall events recorded by a gauging station. It is also capable of applying differing rainfall data to different portions of the watershed. TR-20 is considered a single-event model because it is not able to account for any drying of soil between consecutive rainfall events. However, the program is capable of processing a large number of individual rainfall events in a single run. For example, the program can compute runoff for 2-year, 10-year and 100-year, 24-hour events on a given watershed in a single run.

TR-20 uses the Storage Indication Method to route flood hydrographs through a reservoir such as a detention basin. It does not provide procedures for calculating time-of-concentration, deriving stage-storage tables, or for calculating stage-discharge relationships for hydraulic devices. All such calculations must be generated by other means and then entered into TR-20 by the user.

An important limitation of TR-20 is that it cannot analyze interconnected basins. The outflow from a basin or reservoir is assumed to be steady-state. Thus, the stage-discharge rating curve which is entered for a given reservoir or basin is constant for both the rising and falling limbs of the hydrograph, and the tailwater depth is a function only of the flow through the outlet works. Where a basin discharges into a channel, this assumption is adequate. However, this assumption is not adequate where other factors, such the storage in a downstream basin, can affect the tailwater elevation.

Originally developed for use on mainframe computers, TR-20 still operates in “batch” mode on computers using the Windows™ operating system. Input is prepared as a text file which is then run by the program to produce an output file. Since the output file can contain a detailed printout of flow rate (or elevation) versus time for every cross section in the model, the size of the output file can be quite substantial. The program offers options which may be used to reduce the size of the output file and to control the type of information printed out for each individual cross section.

8.07.3 NRCS TR-55 URBAN HYDROLOGY FOR SMALL WATERSHEDS

The NRCS (formerly the Soil Conservation Service-SCS) TR-55 model was originally created in 1975 as a simplified DOS-based model to simulate small urban watershed runoff hydrographs and calculate runoff volume, peak discharges, and preliminary storage volume requirements for stormwater storage facilities. The model was revised in 1986 by adding rainfall distributions other than Type II. The program became a standard for analyzing the changes to peak runoff in urbanizing areas.

WinTR-55 was created by NRCS hydraulics experts to provide a user-friendly, fully integrated Windows based version of the original model. WinTR-55 is a public domain program which was approved for distribution in July 2002. The program is a single event rainfall runoff small watershed hydraulic model. The model generates hydrographs that are routed downstream through channels or stormwater reservoirs. The hydrograph procedures, including generation and storage routing, are derived from the TR-20 model.

The model only uses the SCS hydrograph methodology for estimating runoff from simulated rainfall events of not less than 24 hours. The model can be used for initial detention basin sizing, but a final analysis using other methods is suggested. Where the facility contains a complex storage or structure configuration requiring a complex rating relationship, the user is recommended to use other routing methods such as TR-20.

Some of the capabilities and limitations of the WinTR-55 model pertinent to creating stormwater storage facilities applicable to linear projects include:

- 1-10 subareas may be analyzed with a maximum drainage area of 25 square miles
- Time of concentration must be between 0.1 hours and 10 hours
- Storage-Indication Method is utilized in stormwater structure routing procedure (see Section 8.05.9.1)
- Structure types may be either pipes or weirs and the program has the ability to compute up to 3 trial sizes simultaneously for each run of the model
- Standard rainfall duration of 24 hours for 8 predefined rainfall distributions, including NRCS Type II applicable to Tennessee

The WinTR-55 model assumes full-flow pipe characteristics at any given stage. This is a reasonable assumption for a combination riser and pipe principal spillway layout but may not be a valid assumption where a large straight pipe is selected as the only outlet device. The user also has the ability to analyze broad crested or v-notch weirs and can view output in easy to read tables and graphs.

8.07.4 U.S.A.C.E. HEC-1 MODEL

The HEC-1 model was developed by the Hydraulic Engineering Center of the US Army Corps of Engineers over 30 years ago and is still a widely used, public domain model for major drainage system analysis. It is used to analyze surface runoff through a watershed during and after a storm event. The model will generate rainfall runoff hydrographs using various available methods. The model user can then perform storage routing at stormwater reservoirs.

The HEC-1 model is a DOS-based program that models a watershed catchment as an interconnected system of sub-basins, channels and reservoirs using a variety of rainfall runoff simulation methods including the SCS curve number method (Section 4.04.5). The designer must prepare an input data file for the watershed prior to running the program using any type of ASCII text editor.

Concerning storage basins, the models limitations include its inability to account for tail water effects at the discharge point. Additionally, the HEC-1 model user is unable to accurately model multistage stormwater facility riser structures. The designer must compute the stage-storage-discharge curve manually for each outlet device. With this limitation, the designer may have difficulty meeting the requirements for multiple design storm frequencies and may find the program to be somewhat cumbersome for the routing process and may want to consider other acceptable software models.

8.07.5 U.S.A.C.E. HEC-HMS MODEL

The US Army Corps of Engineers' Hydraulic Modeling System (HMS) has been designed to simulate the precipitation runoff process of watershed systems, and is a public domain program developed by a team of HEC staff and private consultants. It is intended to supercede or replace the DOS-based HEC-1 model. The program was not developed to simply add a graphical user interface to the aging HEC-1 model, but rather was designed to improve the quality of simulation results. HEC-HMS is a user-friendly, Windows based Hydraulic Modeling System. The program includes a variety of mathematical models for simulating runoff, evapotranspiration, infiltration, base flow, and open channel and reservoir routing. The program features a completely integrated work environment including database, data entry utilities, computational engine, and results reporting tools.

The program features most of the watershed runoff and routing capabilities as the HEC-1 model, and contains new capabilities unavailable to the HEC-1 user, including continuous hydrograph simulation over long periods of time and distributed runoff computation. Both US customary and metric units are available for input of data or output results. The user can automatically convert the model units, if necessary. Previously created HEC-1 data files can be imported directly into HEC-HMS.

To model detention with HEC-HMS, the storage-discharge relationship for the proposed detention basin must be provided by the user. This relationship must be generated manually by hand or by using spreadsheets to simplify repetitive, trial-and-error calculations. With the HEC-HMS model, the user will develop control, meteorological, and basin components for each modeling run. The basin outlet may be designed as a single culvert through an embankment, or may consist of separate conduits of various sizes, or several inlets to a chamber or manifold which then leads to a single outlet barrel. The model uses the level-pool routing model (known as the Modified Puls routing model) to determine the outflow from a specified basin. The model computes the basin outlet rating as the outflow relates to the water surface in the impoundment based on the user supplied weir, orifice, or pipe formulas.

The detention model capabilities included in HEC-HMS are appropriate for simulating the performance of any configuration of outlets and basin. The major limitation to the program is that the model assumes that the outlet will be inlet controlled (i.e. basin outflow is a function of upstream water surface elevation). This should be the case for most stormwater basins required on Department projects. If backwater effects are present at the facility outfall, the designer should not use the HMS model, instead, an unsteady flow network model such as UNET (USACE, 1997) or other proprietary software should be used.

8.07.6 GEOPAK DRAINAGE

Geopak Drainage contains the ability to generate and manipulate rainfall runoff hydrographs and perform reservoir routing at storage facilities. The computational method contained in the programs routing tools is based on NRCS methodologies. The software user is able to create hydrographs using NRCS TR-20 or the Rational Method and route the hydrographs through reaches and stormwater facilities.

For stormwater reservoir routing, the stage-storage relationship may be developed by utilizing a Triangular Irregular Network (TIN) file and existing or proposed surfaces created from the TIN, or the relationship may be manually supplied by the designer. Similarly, for computing a stage-discharge relationship, the program allows the user to manually input the stage-discharge relationship or the software will compute the relationship based on the designer's selection of output device (culvert, broad crested weir, v-notch weir, orifice, etc…).

8.07.7 COMMERCIAL PROGRAMS

Many proprietary software models are available and widely used by design professionals in the design of stormwater storage facilities. Although typically somewhat expensive, commercial software typically contains value-added features such as graphical interfaces and links to CADD systems. Additionally, training, user and technical support is readily available for the commercial software user. The programs generally offer more graphics and report generating options and more structure modeling selections than do the public domain models.

Modeling techniques of commercial software should be based on approved public domain computational procedures (NRCS, etc…). Additionally, the software developers expand on the basic hydraulic principals to provide typically more user-friendly modeling software that contains features and capabilities beyond that of public domain software. The programs typically represent a sizable investment for the potential user, but because of hydraulic modeling and programming staff experts, the software manufactures can provide superior products which have been largely embraced by the private sector design community.

SECTION 8.08 - APPENDIX

8.08 APPENDIX

8.08.1 FIGURES AND TABLES

Extended Detention Basin Operational

Extended Detention Basin Empty

Figure 8A-1
Typical Extended Detention Basin
Reference: Virginia DCR (1999)

Figure 8A-2
Typical Retention Basin with Sediment Forebay
Reference: Virginia DCR (1999)

Figure 8A-3
Typical Dry Detention Basin with Concrete Trickle Ditch
Location: U.S. 41A /SR-112, Nashville, TN

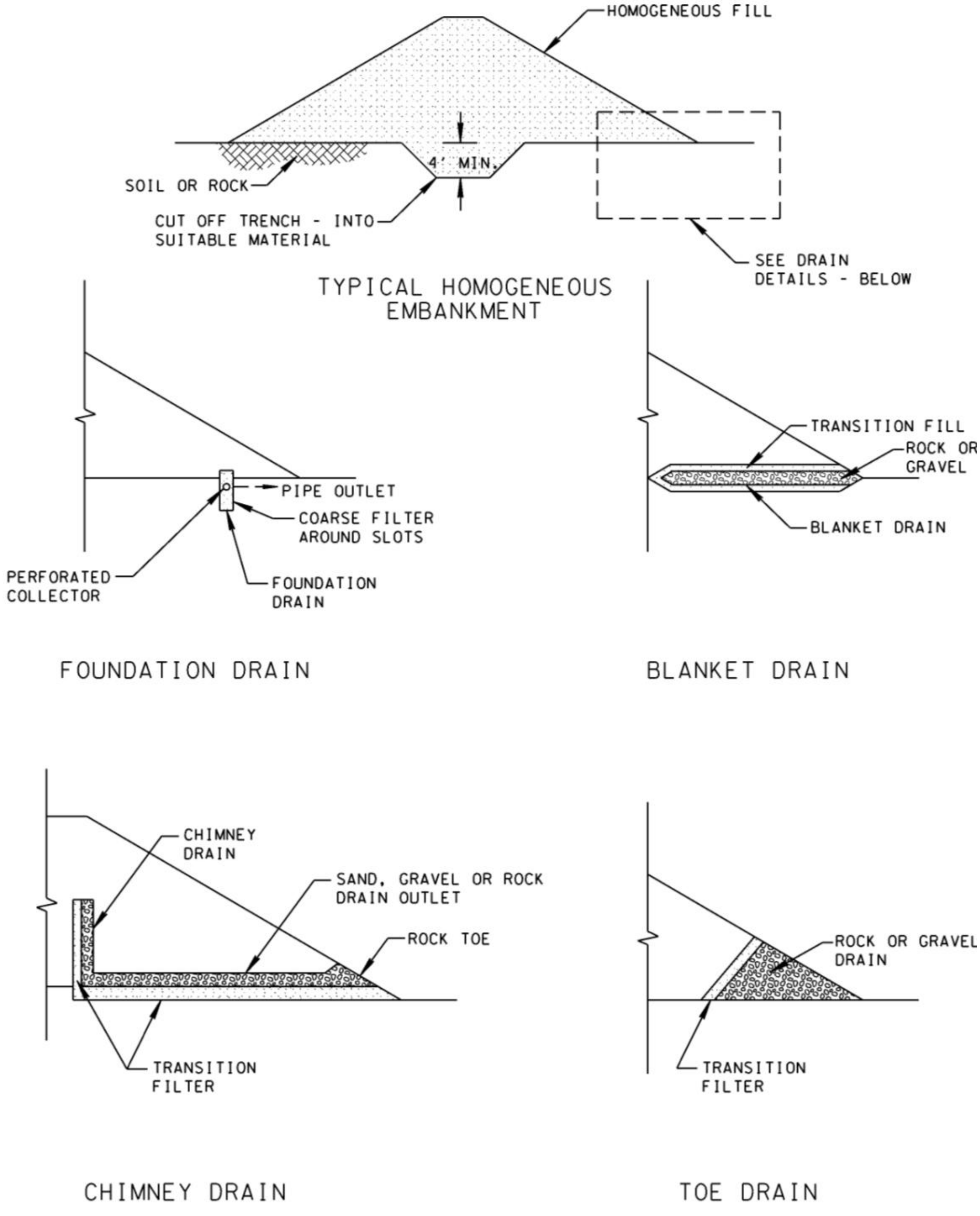


Figure 8A-4
Homogeneous Embankment with Seepage Controls
Reference: Virginia DCR (1999)

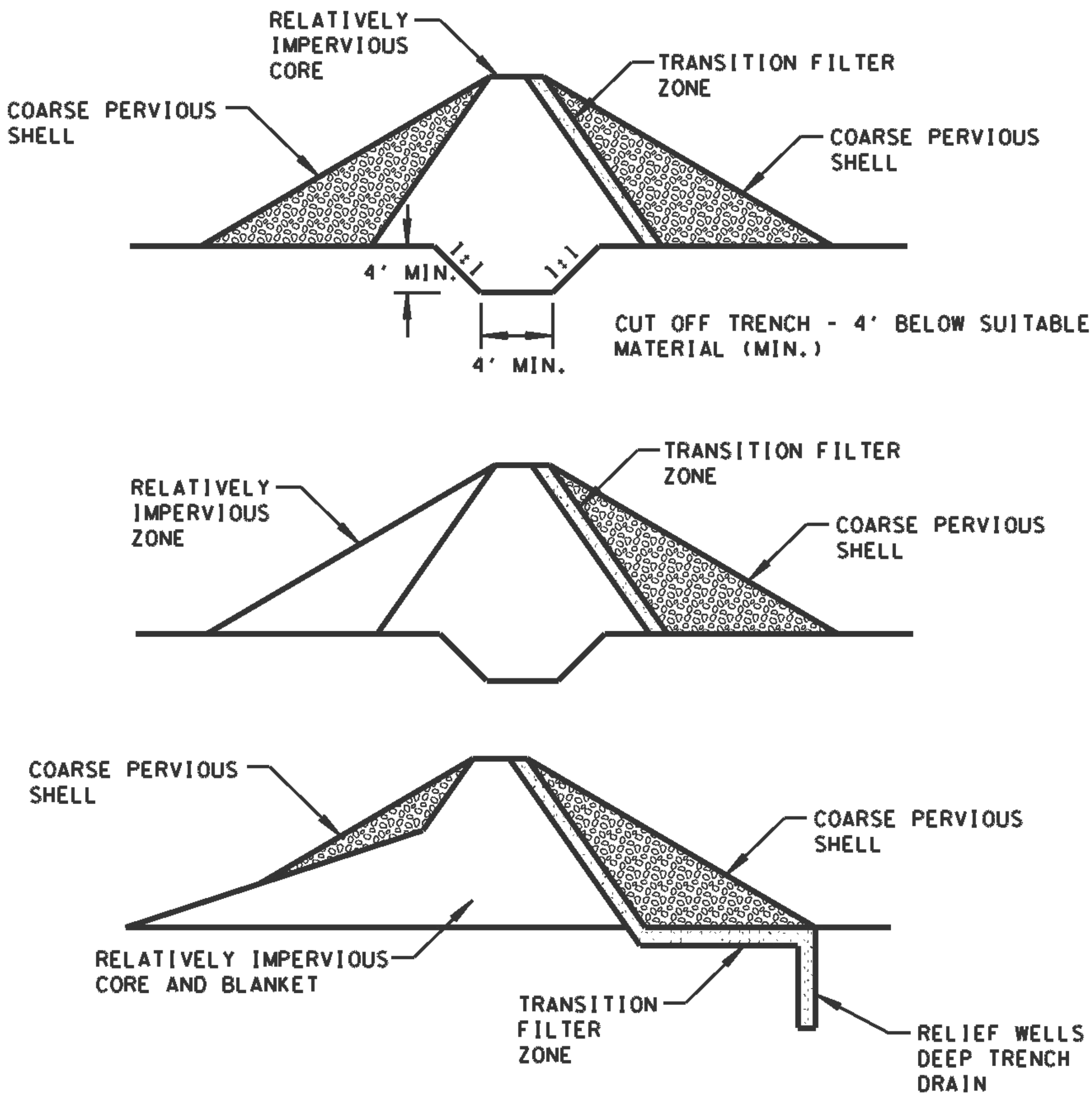


Figure 8A-5
Zoned Embankment
Reference: Virginia DCR (1999)

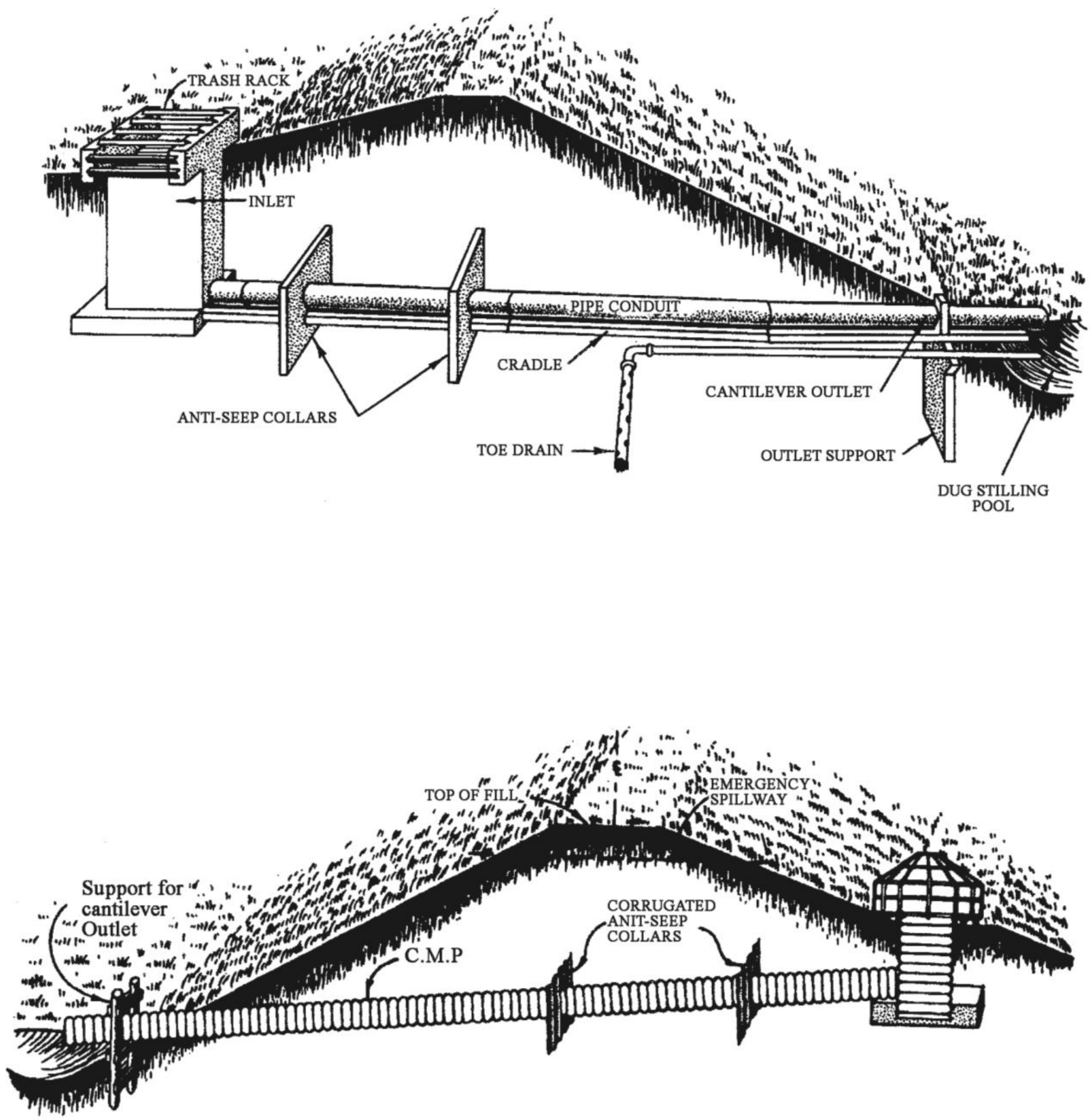


Figure 8A-6
Typical Drop Inlet Spillways
Reference: NRCS, Engineering Field Manual, Chapter 6

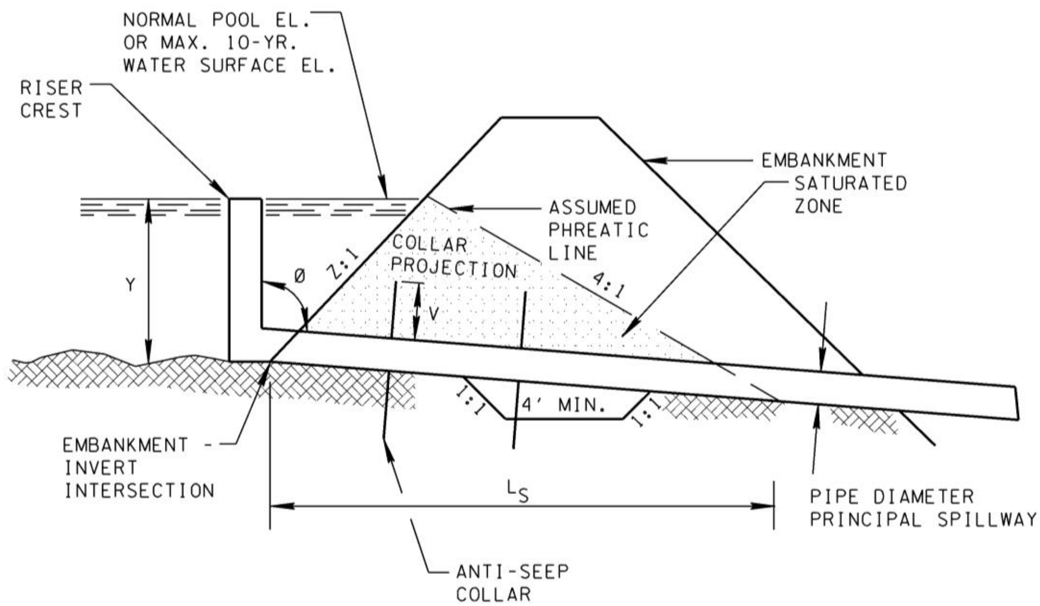


Figure 8A-7
Phreatic Line through Embankment
Reference: GaSWCC (1990)

Figure 8A-8
Anti-Seep Collars on Drop-Inlet Principal Spillway
Reference: USDA, Agricultural Handbook 590 (1997)

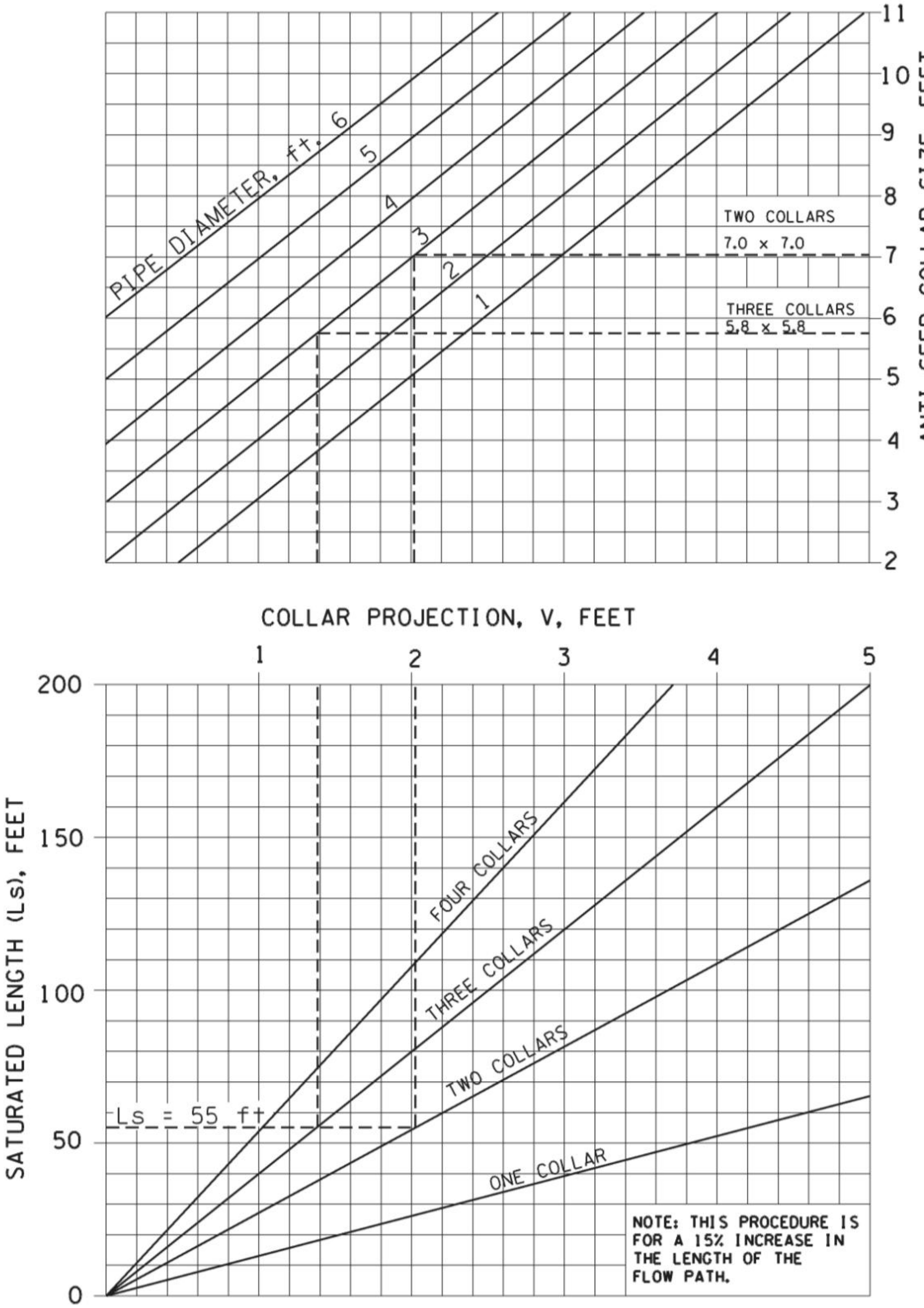


Figure 8A-9
Anti-Seep Collar Design Chart
Reference: Adapted from OHIO D.O.T. (2004)

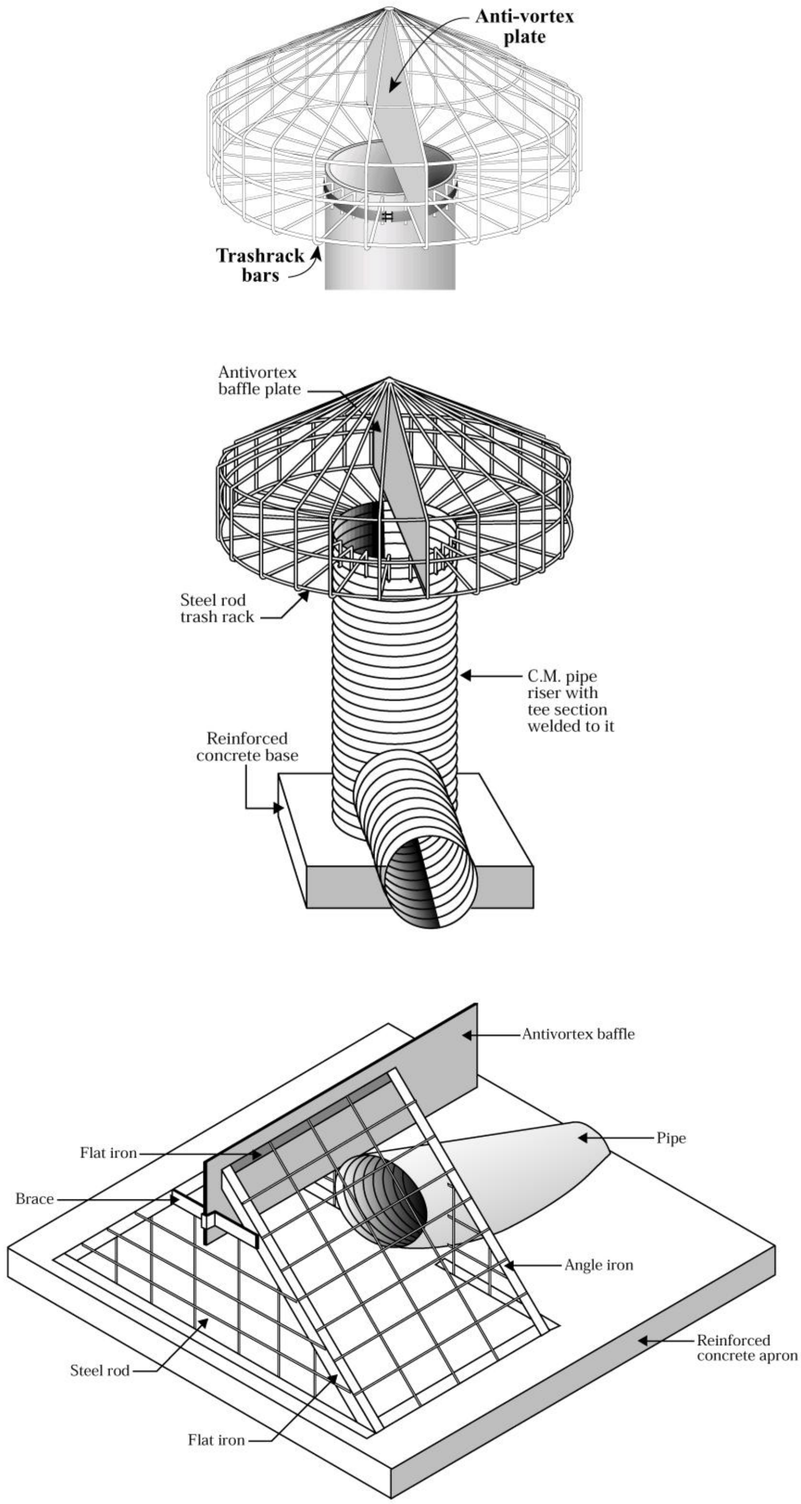
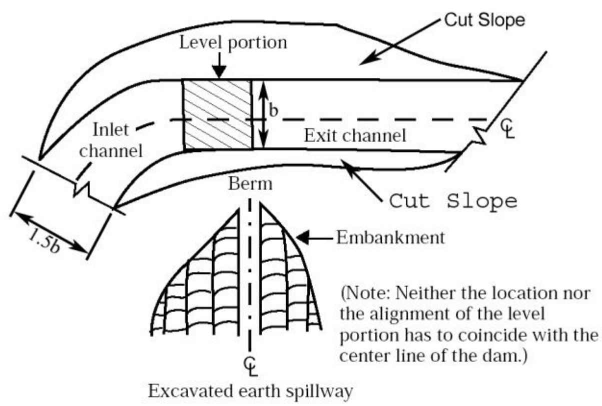
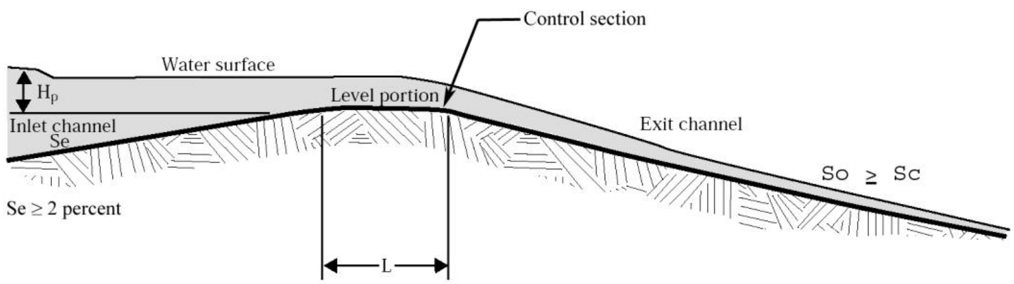


Figure 8A-10
Principal Spillways with Trash Rack and Anti-vortex Device
Reference: USDA, Agricultural Handbook 590 (1997)



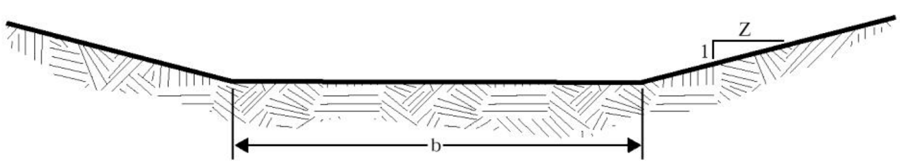
Plan view of earth spillways



Profile along centerline

Definition of terms:

H_p = depth of water in reservoir above crest
L = length of level portion min.
b = bottom width of spillway
So = slope for exit channel
Se = slope of inlet channel
Sc = critical slope



Cross section of level portion (control section)

Figure 8A-11
Typical Excavated Earth Spillway
Reference: Adapted from USDA, Agricultural Handbook 590 (1997)

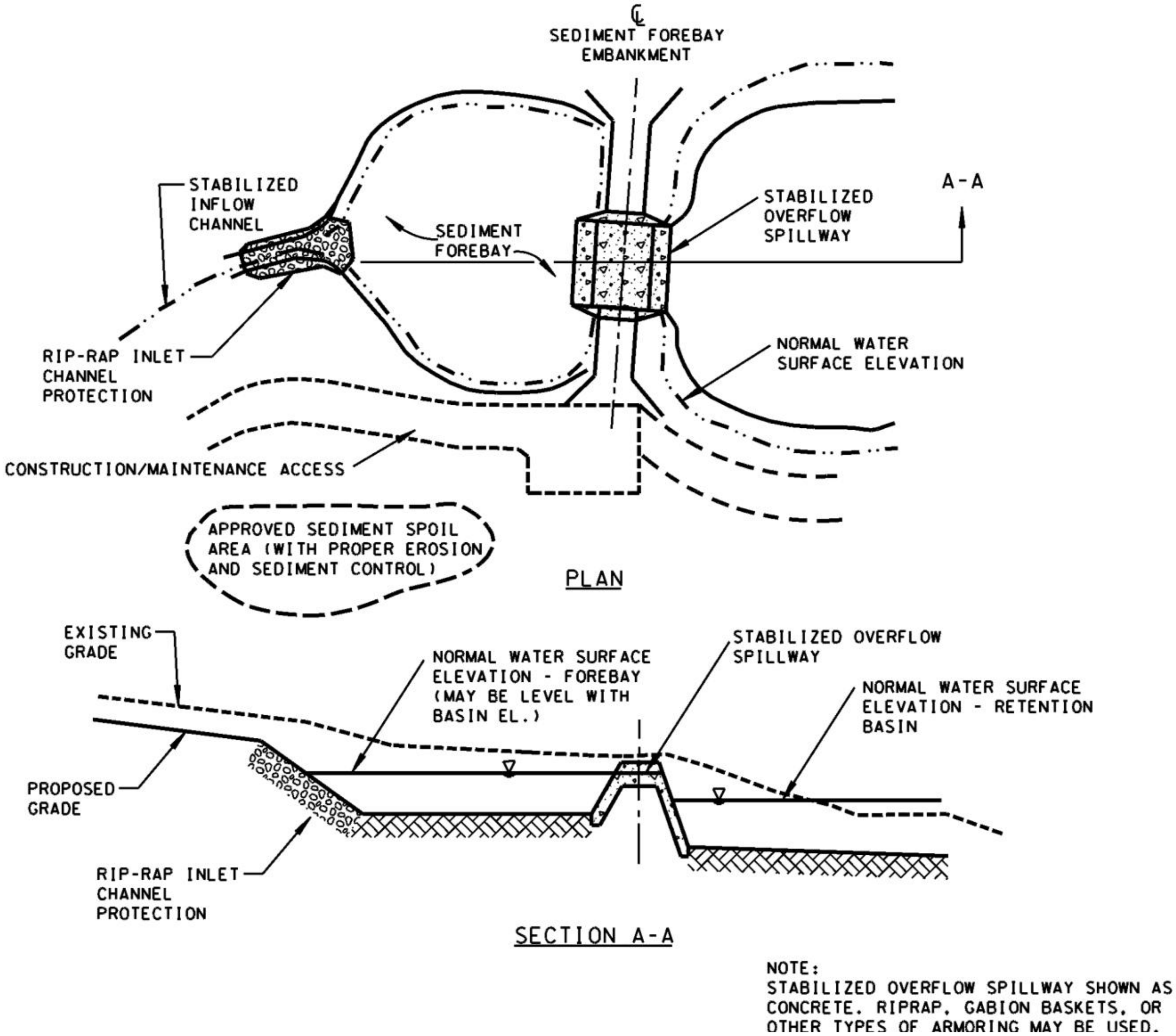


Figure 8A-12
Typical Sediment Forebay for Retention Basins
Reference: Virginia DCR (1999)

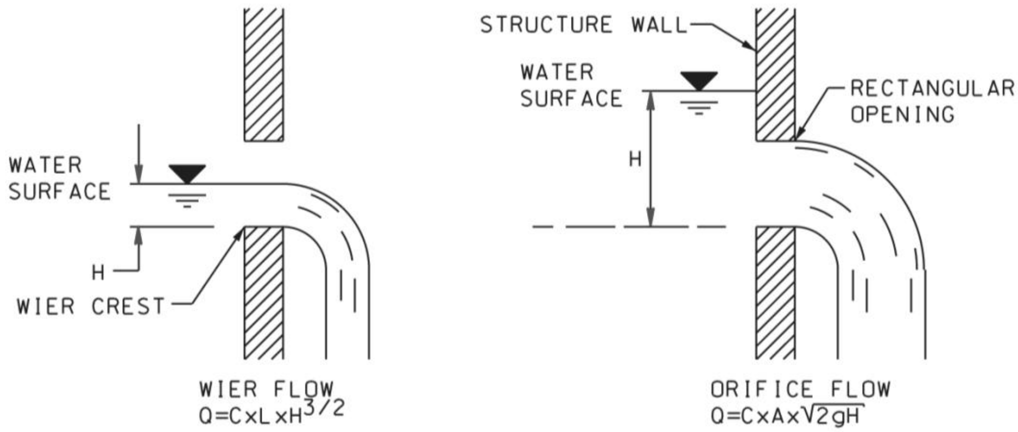
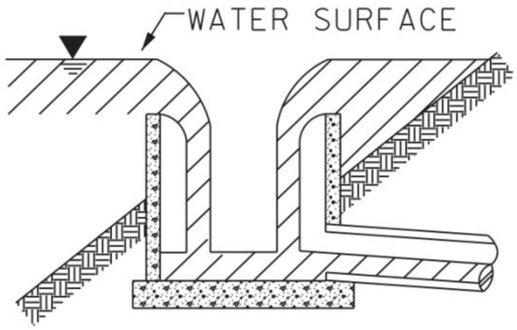
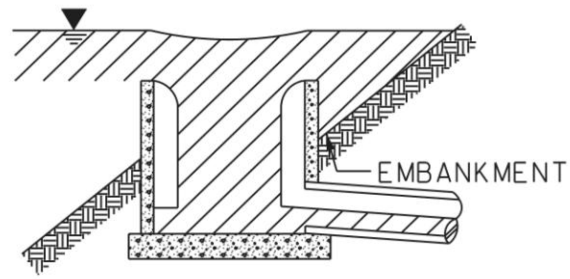


Figure 8A-13
Weir and Orifice Flow at Structure Outlet

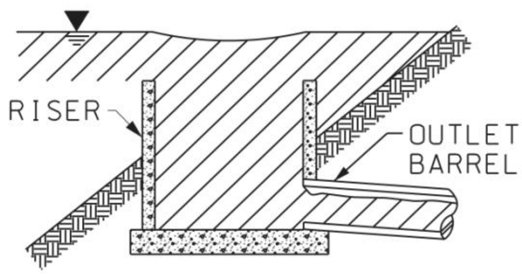


A. Weir Flow
(Unsubmerged)

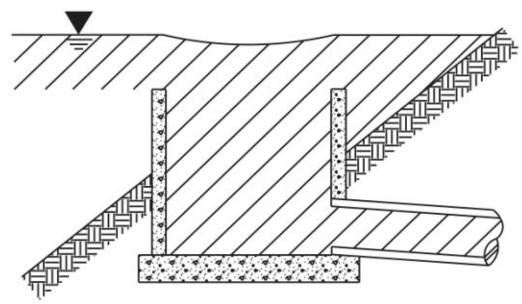


B. Orifice Flow
(Submerged orifice)

Riser Flow Control



C. Barrel Inlet Flow Control
(Controlled at barrel entrance)



D. Barrel Pipe Flow Control
(Controlled along barrel length)

Barrel Flow Control

Figure 8A-14
Riser Flow Diagrams
Reference: NRCS, Engineering Field Manual, Chapter 6

Measured Head H[a] (ft)	Weir Crest Breadth (feet)										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.20	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.40	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.60	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.80	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.00	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.20	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.40	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.60	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.80	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.00	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.50	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.00	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.50	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.00	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.50	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.00	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.50	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[a] Measured at least 2.5H upstream of weir

Table 8A-1
Broad Crested Weir Coefficient C-Values
as a Function of Weir Crest Breadth and Head
Reference: Brater and King (1976)

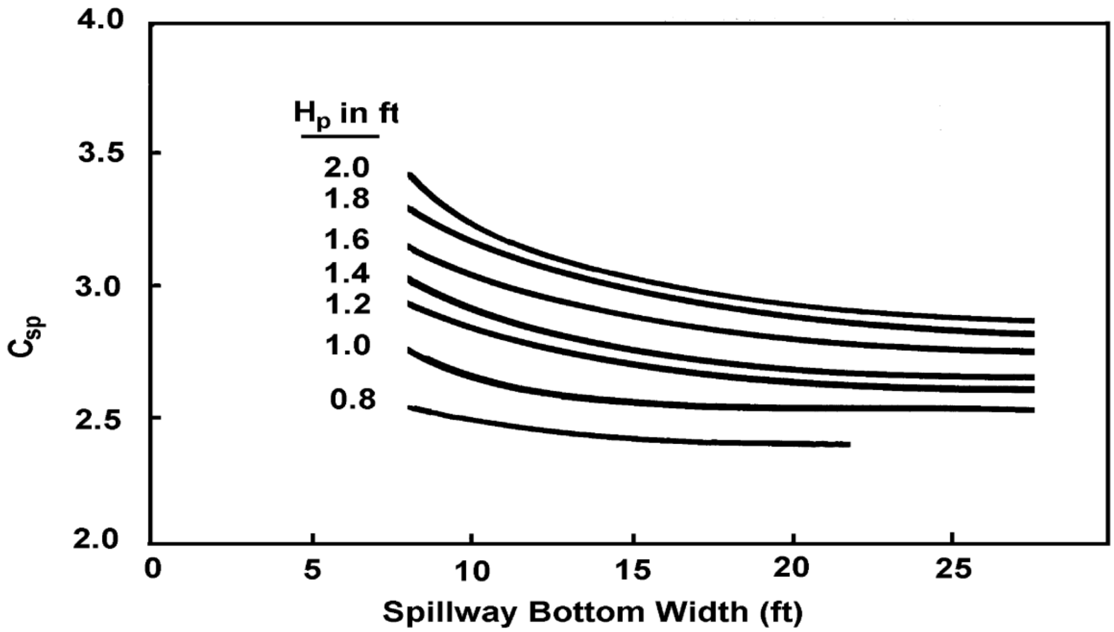


Table 8A-2
Discharge Coefficients for Emergency Spillways
Reference: USDOT, FHWA, HEC-22 (2001)

Coefficients for Circular Culverts Under Inlet Control						
FHWA Chart #	Nomograph Scale	Inlet Edge Description	Unsubmerged		Submerged	
			K	M	C	Y
Chart 1 Concrete	1	Square Edge with Headwall	0.0098	2.00	0.0398	0.67
	2	Groove End with Headwall	0.0078	2.00	0.0292	0.74
	3	Groove End Projecting	0.0045	2.00	0.0317	0.69
Chart 2 Circular CMP	1	Headwall	0.0078	2.00	0.0379	0.69
	2	Mitered to Slope	0.0210	1.33	0.0463	0.75
	3	Projecting	0.0340	1.50	0.0553	0.54

Table 8A-3
Coefficients for Circular Culverts under Inlet Control
Reference: USDOT, FHWA, HDS-5 (1965)

Stage (Hp) in Feet	Spillway Variables	Spillway Bottom Width (b) in Feet																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	37
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	39	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	41
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	17	20	24	26	32	35	39	43	47	51	53	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	47	47	48	48	48	48	48	48	48	48	49	49	49	49	49	49	49
1.0	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
	S	3.1	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	X	51	51	51	51	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	55	55	55	55	55	56	56	56	56	56	56	56	56	56	56
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	58	58	59	59	59	59	59	59	60	60	60	60	60	60	60	60	60
1.3	Q	32	38	46	53	56	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	62	62	62	63	63	63	63	63	63	6	63	64	64	64	64	64	64
1.4	Q	34	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	65	66	66	66	66	67	67	67	67	67	67	68	68	68	68	68	69

Table 8A-4

Design Data for Earth Spillways *(Continued-next page)*

Reference: USDA, Agricultural Handbook 590 (1997)

Stage (Hp) in Feet	Spillway Variables	Spillway Bottom Width (b) in Feet																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	168	178
	V	4.8	4.9	4.9	5	5	5	5	5	5	5	5	5	5	5	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	69	69	70	70	71	71	71	71	71	71	71	72	72	72	72	72	72
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	72	74	74	75	75	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	78	79	80	80	80	80	80	80	80	80	80	80	80	80	80	80
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	80	82	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	84	85	86	87	88	88	88	88	88	88	88	88	88	88	88	88	88
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	88	90	91	91	91	91	92	92	92	92	92	92	92	92	92	92	92
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6	6	6	6	6	6	6	6	6	6
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	92	93	95	95	95	95	95	95	95	96	96	96	96	96	96	96	96
2.2	Q	84	100	116	131	146	163	177	194	210	224	238	253	269	288	301	314	330
	V	5.9	5.9	6	6	6	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.2	6.2	6.2	6.2
	S	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	96	98	99	99	99	99	99	100	100	100	100	100	100	100	100	100	100
2.3	Q	90	108	124	140	158	175	193	208	226	243	258	275	292	306	323	341	354
	V	6	6.1	6.1	6.1	6.2	6.2	6.2	6.2	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3
	S	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	100	102	102	103	103	103	104	104	104	105	105	105	105	105	105	105	105
2.4	Q	99	116	136	152	170	189	206	224	241	260	275	294	312	327	346	364	378
	V	6.1	6.2	6.2	6.3	6.3	6.3	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4
	S	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	105	105	106	107	107	108	108	108	108	108	109	109	109	109	109	109	109

Table 8A-4 *(continued)*
Design Data for Earth Spillways
Reference: USDA, Agricultural Handbook 590 (1997)

Orifice Diameter		Orifice Area	
Inches	Feet	Square Inches	Square Feet
0.50	0.042	0.1963	0.0014
0.75	0.063	0.4418	0.0031
1.00	0.083	0.7854	0.0055
1.50	0.125	1.7671	0.0123
2.00	0.167	3.1416	0.0218
2.50	0.208	4.9087	0.0341
3.00	0.250	7.0686	0.0491
3.50	0.292	9.6211	0.0668
4.00	0.333	12.5664	0.0873
4.50	0.375	15.9043	0.1104
5.00	0.417	19.6350	0.1364
5.50	0.458	23.7583	0.1650
6.00	0.500	28.2743	0.1963
6.50	0.542	33.1831	0.2304
7.00	0.583	38.4845	0.2673
7.50	0.625	44.1786	0.3068
8.00	0.667	50.2655	0.3491
8.50	0.708	56.7450	0.3941
9.00	0.750	63.6173	0.4418
9.50	0.792	70.8822	0.4922
10.00	0.833	78.5398	0.5454
10.50	0.875	86.5901	0.6013
11.00	0.917	95.0332	0.6600
11.50	0.958	103.8689	0.7213
12.00	1.000	113.0973	0.7854

Table 8A-5
Standard Orifice Sizes and Areas

Permissible Velocity [1] (ft/s)				
Vegetative Cover	Erosion Resistant Soils[2]		Easily Erodible Soils[3]	
	Slope of Exit Channel		Slope of Exit Channel	
	0-5 %	5-10%	0-5 %	5-10%
Bermuda Grass Bahiagrass	8	7	6	5
Buffalograss Kentucky Bluegrass Smooth Bromegrass Tall Fescue Reed Canary Grass	7	6	5	4
Sod Forming Grass-Legume Mixtures	5	4	4	3
Lespedeza Weeping Lovegrass Yellow Bluestem Native Grass Mixtures	3.5	3.5	2.5	2.5

[1] Increase values 25 percent when the anticipated average use of the spillway is not more frequent than once in 10 years

[2] Those soils with high clay content and high plasticity. Typical soil textures are silty clay, sandy clay, and clay.

[3] Those soils with a high content of fine sand or silty and lower plasticity or non-plastic. Typical soil textures are fine sand, silt, sandy loam, and silty loam.

Table 8A-6
Permissible Velocities for Vegetated Spillways
Reference: USDA, SCS-TP-60 (1985)

Degree of Retardance	Maximum Permissible Velocity v (ft/s)	Unit Discharge q (cfs/ft)	Depth of Water Above Spillway Crest, H_p (ft) / Length of Level Section, L (ft)				Slope Range (%)	
			25	50	100	200	Min.	Max.
A	3	3	2.3	2.5	2.7	3.1	1	11
	4	4	2.3	2.5	2.8	3.1	1	12
	4	5	2.5	2.6	2.9	3.2	1	7
	5	6	2.6	2.7	3.0	3.3	1	9
	6	7	2.7	2.8	3.1	3.5	1	12
	7	10	3.0	3.2	3.4	3.8	1	9
	8	12.5	3.3	3.5	3.7	4.1	1	10
B	2	1	1.2	1.4	1.5	1.8	1	12
	2	1.25	1.3	1.4	1.6	1.9	1	7
	3	1.5	1.3	1.5	1.7	1.9	1	12
	3	2	1.4	1.5	1.7	1.9	1	8
	4	3	1.6	1.7	1.9	2.2	1	9
	5	4	1.8	1.9	2.1	2.4	1	8
	6	5	1.9	2.1	2.3	2.5	1	10
	7	6	2.1	2.2	2.4	2.7	1	11
	8	7	2.2	2.4	2.6	2.9	1	12
C	2	0.5	0.7	0.8	0.9	1.1	1	6
	2	1	0.9	1.0	1.2	1.3	1	3
	3	1.3	0.9	1.0	1.2	1.3	1	6
	4	1.5	1.0	1.1	1.2	1.4	1	12
	4	2	1.1	1.2	1.4	1.6	1	7
	5	3	1.3	1.4	1.6	1.8	1	6
	6	4	1.5	1.6	1.8	2.0	1	12
	8	5	1.7	1.8	2.0	2.2	1	12
	9	6	1.8	2.0	2.1	2.4	1	12
	9	7	2.0	2.1	2.3	2.5	1	10
	10	7.5	2.1	2.2	2.4	2.6	1	12
D	2	0.5	0.6	0.7	0.8	0.9	1	6
	3	1	0.8	0.9	1.0	1.1	1	6
	3	1.25	0.8	0.9	1.0	1.2	1	4
	4	1.25	0.8	0.9	1.0	1.2	1	10
	4	2	1.0	1.1	1.3	1.4	1	4
	5	1.5	0.9	1.0	1.2	1.3	1	12
	5	2	1.0	1.2	1.3	1.4	1	9
	5	3	1.2	1.3	1.5	1.7	1	4
	6	2.5	1.1	1.2	1.4	1.5	1	11
	6	3	1.2	1.3	1.5	1.7	1	7
	7	3	1.2	1.3	1.5	1.7	1	12
	7	4	1.4	1.5	1.7	1.9	1	7
	8	4	1.4	1.5	1.7	1.9	1	12
	8	5	1.6	1.7	1.9	2.0	1	8
	10	6	1.8	1.9	2.0	2.2	1	12
E	2	0.5	0.5	0.5	0.6	0.7	1	2
	3	0.5	0.5	0.5	0.6	0.7	1	9
	3	1	0.7	0.7	0.8	0.9	1	3
	4	1	0.7	0.7	0.8	0.9	1	6
	4	1.25	0.7	0.8	0.9	1.0	1	5
	5	1	0.7	0.7	0.8	0.9	1	12
	5	2	0.9	1.0	1.1	1.2	1	4
	6	1.5	0.8	0.9	1.0	1.1	1	12
	6	2	0.9	1.0	1.1	1.2	1	7
	6	3	1.2	1.2	1.3	1.5	1	4
	7	2	0.9	1.0	1.1	1.2	1	12
	7	3	1.2	1.2	1.3	1.5	1	7
	8	3	1.2	1.2	1.3	1.5	1	10
	8	4	1.4	1.4	1.5	1.7	1	6
	10	4	1.4	1.4	1.5	1.7	1	12

Table 8A-7
H_p and Slope Range for Discharge, Permissible Velocity, and Spillway Crest Length Based on Degree of Retardance
Reference: USDA, Agricultural Handbook 590 (1997)

D/d	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.00	0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	0.614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.674	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

Let: D/d = Depth of Water / Dia. Of Pipe, and C_a = Values in Table

Then $B = C_a d^2$

Table 8A-8
Values of C_a Used for Determining Term
B in the Ungula of Cone Formula
Reference: USDOT, FHWA, IP-82-17, Vol. 1 (1982)

8.08.2 EXAMPLE PROBLEMS

8.08.2.1 EXAMPLE PROBLEM #1: STORAGE ESTIMATE BY MODIFIED RATIONAL METHOD

GIVEN:

An existing 2-lane State Route in Memphis, Tennessee is to be widened to a proposed 5-lane section with curb and gutter and sidewalk on one side of the route. The project is currently in the preliminary design phase of development. At one location along the route, the outfall of the proposed storm sewer system will discharge to an existing ditch which continues beyond the right-of-way to an existing residential area. The proposed ditch travels between two homes to a pipe and headwall where it enters the storm sewer system of the subdivision. The off-site storm sewer system is not capable of handling additional flow resulting from the proposed improvements; therefore, the designer will be required to include a detention basin with the plans.

Site Conditions – Example Problem #1					
	Drainage Area (acres)	Runoff Coefficient (weighted)	Time of Concentration (minutes)	2-year Discharge (ft^3/s)	10-year Discharge (ft^3/s)
Pre-Project	6.3	0.41	18	9.59	12.0
Post-Project	6.3	0.60	10	17.8	22.1

Table 8A-9
Hydrologic Analysis and Computed Peak Discharge for Modified Rational Method Example

FIND:

Obtain a preliminary estimate of the required storage to attenuate the 2 and 10-year storms to meet the Department’s stormwater storage requirements using the Modified Rational Method.

SOLUTION:

As described in Section 8.05.4.1.3, the designer should determine the peak discharge for the 2-year and the 10-year storm events using the Rational Method as presented in Chapter 4 of this Manual. The results of the hydrologic site analysis and the computed peak discharges are given in Table 8A-9. Rainfall intensities used for analysis were taken from the Jackson IDF-curves for the respective storms as presented in Figure 4A-5.

Step 1: The 2-year post-project triangular hydrograph is developed based on the computed time of concentration, T_c. This provides the peak inflow rate at time=T_c. See Figure 8A-15 for the graphical representation of this hydrograph.

Step 2: Select rainfall intensity averaging periods which are incrementally longer than the time of concentration to develop the family of trapezoidal shaped hydrographs. With the post project T_c of 10 minutes, the designer should choose time increments which are conveniently read from the IDF-curve for this site (any time period is acceptable and selection is purely arbitrary). Incremental time periods of 10 (the time of concentration), 15, 20, 30, 40, 60, and 120 minutes are arbitrarily chosen for ease in reading rainfall intensities from the 2 and 10-year IDF-curves for Jackson. The shortest period selected should be the time of concentration.

Step 3: For the 2-year storm, determine the rainfall intensities, i, and peak inflow rates, Q_i, to the storage basin for each of the storm durations chosen in Step 2. A table summarizing theses values is shown in the following table. The peak inflow rate, Q_i, is calculated using the Rational Method given as Equation 8-5.

Averaging Period – T_d (minutes)	Averaging Period – T_d (seconds)[1]	Rainfall Intensity (inches/hour)	Peak Inflow – Q_i (cfs)
10	600	4.72	17.84
15	900	3.96	14.97
20	1200	3.55	13.42
30	1800	2.73	10.32
40	2400	2.39	9.03
60	3600	1.71	6.46
120	7200	1.05	3.97

[1] Rainfall averaging period converted to seconds for consistency of units

Table 8A-10
Table of Values for Modified Rational Method Example,
2-Year Storm Event

Step 4: Compute the required storage volume, V_s, using Equation 8-6 and the values contained in Table 8A-10 of Step 3.

For an averaging period, T_d = 10 minutes = T_c, the required storage is:

$$V_s = Q_i T_d - Q_o\left(\frac{T_d + T_c}{2}\right)$$

$$V_s = (17.84)(600) - 9.59\left(\frac{600+600}{2}\right) = 4{,}950 \text{ ft}^3$$

Note in the computations that Q_o is the allowable outflow for this site, which is the pre-project peak discharge rate for this location. Q_o may be less than this if other downstream factors warrant a lesser value or require restricting the discharge even more.

The estimate of required storage volume at T_d =15 minutes is computed as:

$$V_s = (14.97)(900) - 9.59\left(\frac{900+600}{2}\right) = 6{,}279 \text{ ft}^3$$

The following equations show the calculations for determining required storage volume for the 20, 30, 40, 60, 120 minute rainfall intensity averaging periods (durations):

$$V_s = (13.42)(1200) - 9.59\left(\frac{1200+600}{2}\right) = 7{,}472 \text{ ft}^3 \qquad \text{(20-minutes)}$$

$$V_s = (10.32)(1800) - 9.59\left(\frac{1800+600}{2}\right) = 7{,}067 \text{ ft}^3 \qquad \text{(30-minutes)}$$

$$V_s = (9.03)(2400) - 9.59\left(\frac{2400+600}{2}\right) = 7{,}297 \text{ ft}^3 \qquad \text{(40-minutes)}$$

$$V_s = (6.46)(3600) - 9.59\left(\frac{3600+600}{2}\right) = 3{,}131 \text{ ft}^3 \qquad \text{(60-minutes)}$$

$$V_s = (3.97)(7200) - 9.59\left(\frac{7200+600}{2}\right) = -8{,}824 \text{ ft}^3 \text{ use } 0.0 \qquad \text{(120-minutes)}$$

The following table summarizes the Modified Rational computations for the 2-year storm event.

2-year Modified Rational Critical Storm Duration				
Averaging Period – T_d (minutes)	**Averaging Period – T_d (seconds)[1]**	**Rainfall Intensity (inches/hour)**	**Peak Inflow – Q_i (cfs)**	**Req'd Storage Volume – V_s (ft^3)**
10	600	4.72	17.84	4,951
15	900	3.96	14.97	6,279
20	1200	3.55	13.42	7,472
30	1800	2.73	10.32	7,067
40	2400	2.39	9.03	7,297
60	3600	1.71	6.46	3,131
120	7200	1.05	3.97	0.0

[1] Rainfall averaging period converted to seconds for consistency of units

Table 8A-11
Completed Table of Values for Modified Rational Method Example,
2-Year Storm Event

Note in the required storage column that the maximum storage required occurs at the critical storm duration of 20 minutes, not when the discharge peaks at the time of concentration of 10 minutes. This illustrates that when sizing a stormwater storage facility, the runoff volume is of greater consequence than the rainfall intensity.

Step 5: Using Equation 8-6, compute the required storage volume, V_s, for the 10-year storm event. Repeat Steps 2 through 4 above for the 10-year storm. The allowable outflow, Q_o, for this run will be the pre-project peak runoff rate of 12.0 cfs. The following table summarizes the Modified Rational computations for the 10-year storm event. Note in the required storage column that the maximum storage required occurs at the critical storm duration of 40 minutes, not when the discharge rate peaks at the time of concentration of 10 minutes. Note also that the maximum required storage occurs for a 40 minute duration storm for the 10-year storm, but occurs at 20 minutes for the 2-year storm.

Averaging Period – T_d (minutes)	**Averaging Period – T_d (seconds)**	**Rainfall Intensity (inches/hour)**	**Peak Inflow – Q_i (cfs)**	**Req'd Storage Volume – V_s (ft^3)**
10	600	5.84	22.08	6,045
15	900	4.93	18.64	7,772
20	1200	4.48	16.92	9,506
30	1800	3.57	13.49	9,890
40	2400	3.16	11.93	10,637
60	3600	2.33	8.81	6,507
120	7200	1.44	5.44	0.0

Table 8A-12
Completed Table of Values for Modified Rational Method Example,
10-Year Storm Event

The greatest volume of runoff is generated by the storm having a critical duration of 40 minutes, even though this storm does not produce as large a peak rate of runoff as the 10 minute duration storm (this site's T_c). The basin should initially be designed with adequate volume to detain the 40 minute duration storm. This volume should be considered a reasonable number for preliminary estimating purposes. Additional volume may need to be provided at the preliminary design stage of plans development due to the underestimate of required volume typical of the Modified Rational Method.

The Modified Rational method runoff hydrographs for the 10-year storm computations are shown in the figure below.

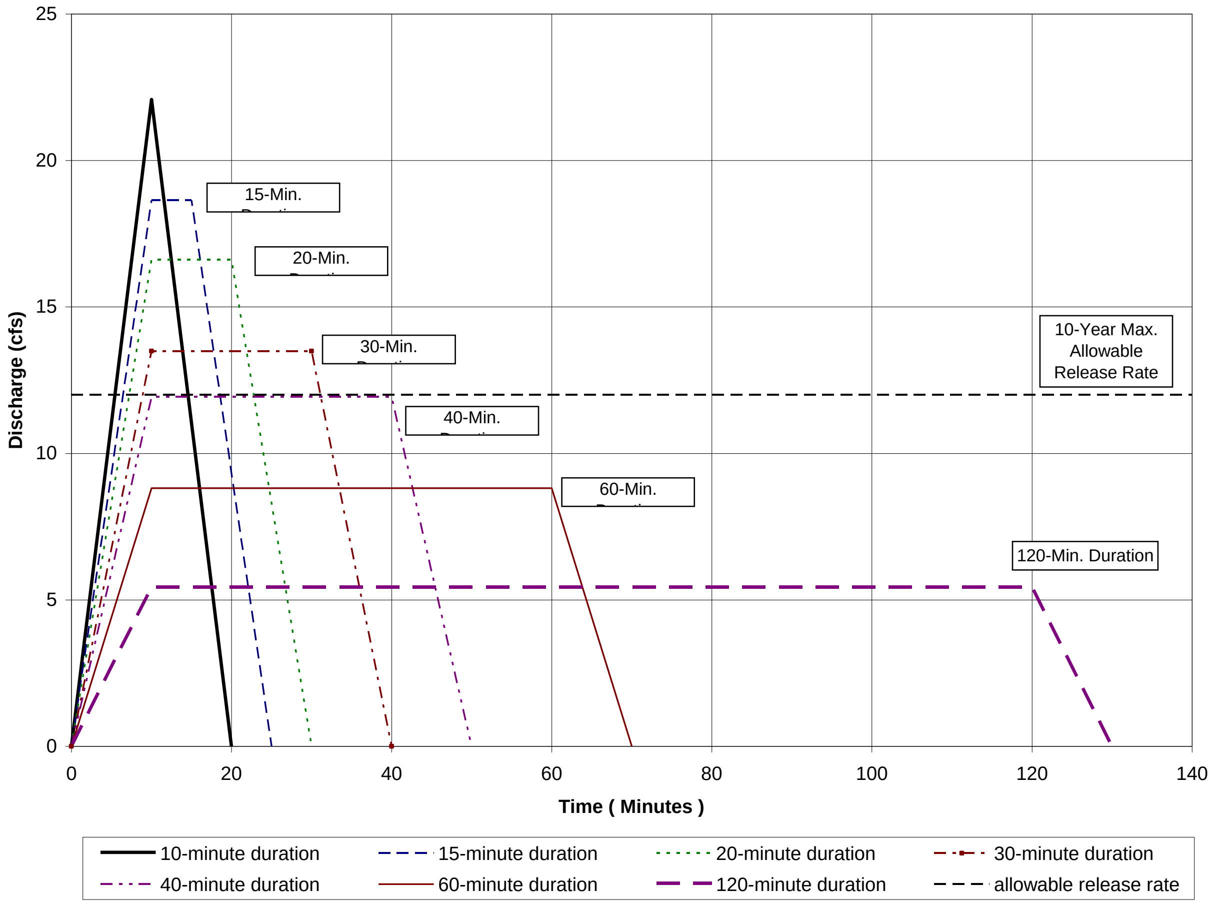


Figure 8A-15
Modified Rational Method Hydrographs for Example Problem #1
10-Year Storm Event

8.08.2.2 EXAMPLE PROBLEM #2: WATER QUALITY OUTLET DESIGN

GIVEN:

A proposed new 4-lane divided highway is to be constructed in the watershed of a small unnamed tributary to a 303d listed stream in Greene County. To minimize potential negative impacts to the aquatic characteristics of the stream, the design manager will be responsible for providing a detention facility which will capture the first-flush of runoff from the proposed roadway and adjacent areas. The detention basin will attenuate stormwater from a 26 acre watershed comprised of 10 acres of pervious, open areas and 16 acres of impervious surface including the roadway, shoulders, sidewalk, and some adjacent off-site parking that drains directly to the project storm sewer.

The stage-storage curve provided in Figure 8A-16 has been developed by the designer based on her proposed plan for grading the basin. Please note that in Figure 8A-16 Stage on the x-axis means water surface elevation.

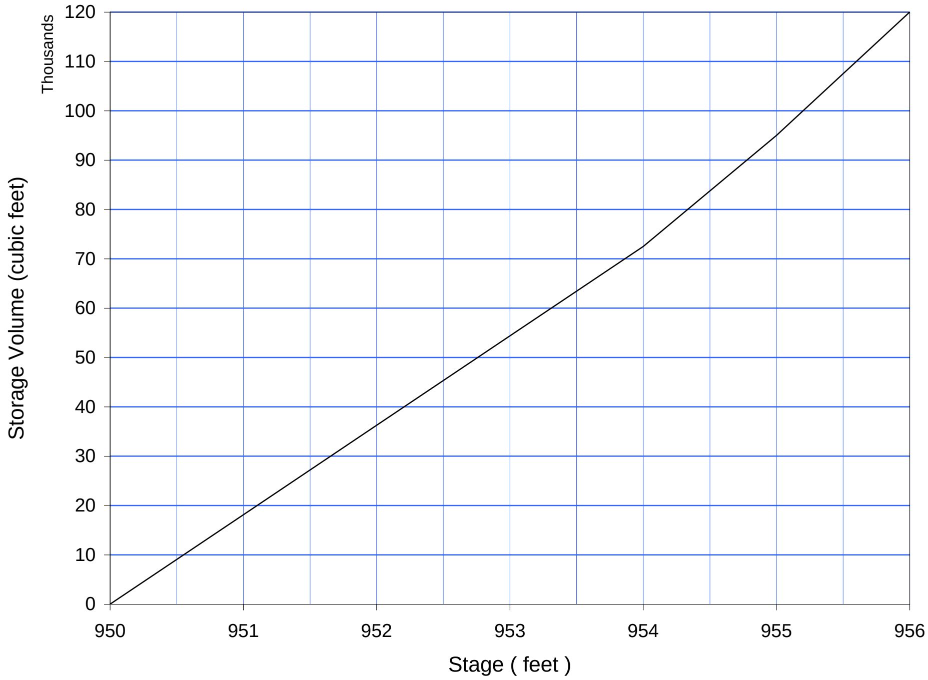


Figure 8A-16
Stage-Storage Curve, Greene County, TN
Note: Stage means water surface elevation within basin

FIND:

Size a single orifice to serve as the water quality outlet device for the stormwater storage facility.

SOLUTION:

The guidelines for designing a *basic* water quality measure can be found in Section 8.04.3.2 of the chapter text. To control sediment and other pollutants, the designer should provide an extended detention basin meeting required drawdown time limits to allow settling of anticipated suspended solids and other pollutants.

Step 1: Determine the Required Water Quality Volume, V_{wq}

The required first flush volume necessary to provide water quality storage in the facility can be computed using Equation 8-1 of Section 8.04.3.2.

$$V_{wq} = \frac{0.5 inch}{12 \frac{inch}{foot}} \times A_{imp}$$

Where: A= 16 acres x 43560 ft^2/acre = 696,960 ft^2

$$V_{wq} = \frac{0.5''}{12} \times 696{,}960 = 29{,}040 \text{ ft}^3$$

Step 2: Determine the Average Discharge Resulting from the Drawdown Time Constraints

From Section 8.04.3.2, the drawdown time for a detention facility should range between 24 and 48 hours. The designer chooses an average time of 36 hours as her target to compute the average discharge rate over this time. Using Equation 8-30, the average required discharge rate over the selected 36 hour period is computed as:

$$Q_{avg} = \frac{V_{wq}}{t_{dd}(3600)}$$

$$Q_{avg} = \frac{29{,}040}{36(3600)} = 0.224 \text{ ft}^3\text{/s}$$

Step 3: Determine the Average Hydraulic Head Expected on the Drawdown Orifice

From the stage-storage curve of Figure 8A-16, the water surface elevation (stage) corresponding to the water quality volume computed in Step 1 can be determined by entering the graph on the y-axis at a volume of 29,040 ft^3, extending horizontally to the point of intersection with the curve, and then projecting down to the x-axis. From the figure, a stage of approximately 951.6 is used.

The average hydraulic head is computed by:

$$H_{avg} = \left(\frac{951.6 - 950}{2}\right) = 0.8 \text{ ft}$$

Where: 950 is the lowest available elevation in the basin (the basin floor)

Step 4: Determine the Required Orifice Size

Equation 8-31 in the Chapter text represents the orifice equation rearranged to solve for area. Using this equation, the orifice area is computed as:

$$A = \frac{Q}{C(2gH)^{0.5}}$$

$$A = \frac{0.224}{0.6(2(32.2)0.8)^{0.5}} = 0.052 \text{ ft}^2$$

Where: 0.6 is the orifice coefficient

The orifice diameter can then be calculated:

$$A = \pi\frac{d^2}{4}$$

Solving for d, yields an orifice diameter of 0.257 feet, which is 3.08 inches. The designer should use 3 inches for design.

Step 5: Determine the Actual Drawdown Time Based on Final Orifice Size

With the orifice area computed as 0.052 ft^2, the designer could also obtain the approximate orifice diameter using Table 8A-5 of the chapter Appendix. Solving the standard orifice equation (Equation 8-20) using a 3 inch diameter orifice, and solving for t_{dd} in Step 2, above, yields an actual draw-down time of 38 hours under ideal conditions, which is within the desired time constraints.

The bottom of the orifice should be set at the basin elevation corresponding to a volume of 29,040 ft^3. For this site, and from the stage-storage curve, this elevation is 951.6. The designer should consider structural measures and other design features to prevent clogging of the device.

8.08.2.3 EXAMPLE PROBLEM #3: EMERGENCY SPILLWAY & BUOYANCY ANALYSIS

GIVEN:

A new roadway project in Rhea County is in the final stages of the design process. The expected increase in stormwater runoff will be attenuated with the use of a detention basin located downgrade of the entire project. The proposed project will result in a 100-year peak storm discharge of 43 cfs. The proposed detention basin will have the following characteristics:

Emergency Spillway Crest Elevation = 859.25
Riser Crest Elevation = 858.75
4' Square (inside), Staged Concrete Riser with 8" thick walls
6" Diameter 2-year orifice at Elevation = 850
10" Diameter 10-year orifice at Elevation = 853
8-foot square x 18" thick concrete footing 1' below grade
18" RCP principal spillway pipe
Weight of reinforcing steel, trash rack, anti-vortex baffle, bolts, etc. = 280 pounds

The emergency spillway for the proposed facility will be cut into undisturbed soil. The spillway will have a 25 foot long level section, an exit channel with a 6% slope, and it is expected to maintain a fair stand of tall fescue. Figure 8A-17 provides a typical section through the drop inlet spillway and embankment for the project site. The designer wants to evaluate the facility based on complete failure (blockage) of the principal spillway device.

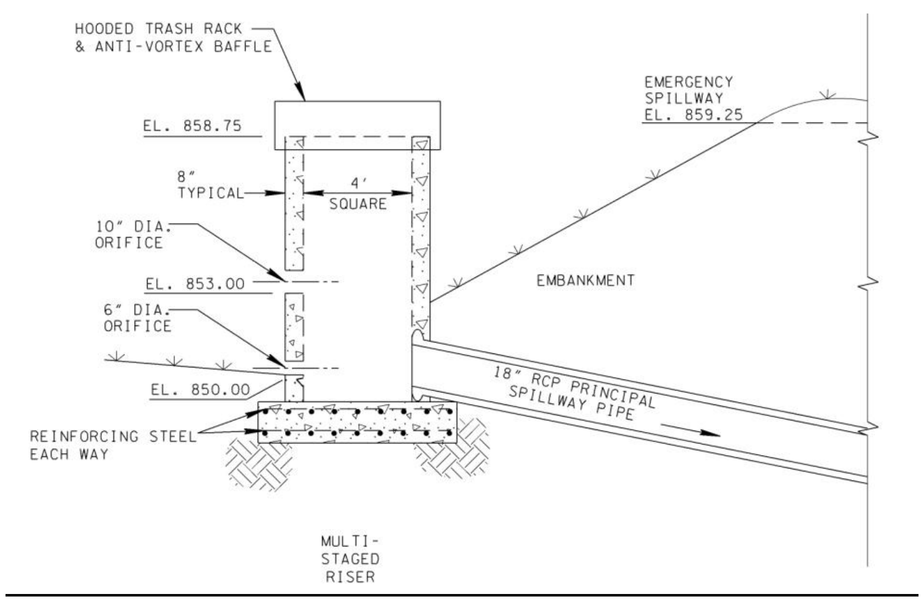


Figure 8A-17
Spillway Layout for Example Problem #3
Not to scale

FIND:

a) Permissible velocity, required level-section width, and final top of embankment elevation
b) Verify that floatation will not occur during complete failure of the principal spillway

SOLUTION:

Part a)

Step 1: **Determine the Permissible Velocity of the Emergency Spillway**

Entering Table 8A-6 knowing the slope of the exit channel and the proposed vegetation, the designer determines the permissible velocity of the exit channel to be 4 feet per second. The project is located in southwest portion of east Tennessee, and from geotechnical analysis of the site it has been found to be predominately comprised of a cherty-clay and sandy soil. The designer chooses to use the velocities given for easily erodible soils.

Step 2: **Classify the Degree of Retardance of the Emergency Spillway Vegetation**

Referencing Table 5A-4 of the Appendix of Chapter 5, a fair stand of fescue with an average height of 6 to 10 inches is determined to have a Degree of Retardance of D. An alternate method for determining degree of retardance would be to classify based on length of the specific type of vegetation. This table can be referenced at USDA, SCS-TP-61, 1947 (table 4)

Step 3: **Determine the Required Width of the Level Section of the Emergency Spillway**

The unit discharge per foot of spillway width must be determined based on the permissible velocity, degree of retardance, and the slope of the exit channel (given). Enter Table 8A-7 at the left side with a degree of retardance D, maximum velocity of 4 ft/sec from Step 1 and the given exit channel slope of 6%. Two rows of values are given for a maximum velocity of 4 ft/sec; however, the slope of the exit channel governs the use of the first row. Using the slope range of 1 to 10 percent, the unit discharge, q, of the exit channel is found to be 1.25 cfs/ft. The required bottom width of the spillway is computed by dividing the design discharge by the spillway unit discharge.

Bottom Width $= Q_{100}$(cfs) / q (cfs/ft)
= 43 / 1.25 = 34.4 feet (Use 35 feet)

Step 4: **Determine the Minimum Top of Embankment Elevation**

From Table 8A-7, the depth of water, H_p, upstream of the control section, above the elevation of the emergency spillway crest, with a 25-foot level section is determined to be 0.8 feet. Section 8.04.5.3 of the Chapter text provides the minimum freeboard requirements of 1 foot. The top of the embankment is then determined by:

Spillway crest elevation + H_p + freeboard = 859.25 + 0.8 + 1 = 861.05

861.05 is the minimum elevation required for the top of the embankment. The designer should evaluate the need to provide additional freeboard for wave action if necessary, and should crown or slope the top of the embankment toward the inside of the facility.

Part b)

A general discussion on performing buoyancy analysis is presented in Section 8.05.8, Multi-Stage Riser Design. The approximate unit weights of concrete, soil, and water given in Section 8.05.8 will be used for analysis. The designer should obtain the actual unit weights to be used at a specific project site.

Step 1: **Compute the Total Buoyant Force F_b exerted on the Structure**

The calculations assume that the proposed principle spillway is completely blocked, therefore the soil in the detention facility will be assumed in a saturated state. The total buoyant force acting on the structure as uplift is computed using Equation 8-33.

F_b = (total volume displaced)(unit weight of water)

$F_b = (V_{riser} + V_{ftg})(62.4 \text{ lbs/ft}^3)$

The displaced volume of the riser is computed using the exterior dimensions of the riser and the footing. Openings are neglected. The riser volume is the outside dimension of 28.45 square feet times the height of the riser. Note that the total riser height is 9.75 feet, which includes that which is 1-foot below grade. Therefore, the riser displacement is 277.4 ft^3. The footing displacement is 96 cubic feet.

$F_b = (277.4 \text{ ft}^3 + 96 \text{ ft}^3)(62.\ 4 \text{ lbs/ft}^3) = 23{,}300 \text{ lbs}$

Step 2: **Compute the Total Downward Force Exerted by the Structure**

This resistance force, F_r, is computed as the total force acting to resist uplift during a storm event. The total force can be computed using Equation 8-34.

$F_r = V_{concrete}(150 \text{ lbs/ft}^3) + V_{soil}(120 \text{ lbs/ft}^3)$ + Weight of all other riser components

$F_r = (214.8 \text{ ft}^3)(150 \text{ lbs/ft}^3) + (35.55 \text{ ft}^3)(120 \text{ lbs/ft}^3) + 280$ lbs.

$F_r = 36{,}766$ lbs

Note: All openings in riser must be accounted for (subtracted from total) when computing concrete volume. The concrete volume is the sum of the footing concrete and the riser wall concrete, minus the orifice openings and the outlet pipe opening (assumed to be 24 inches for the 18-inch outlet pipe in this example) concrete.

Step 3: **Verify Adequacy of Computed Results**

The designer should consider a minimum factor of safety of 1.25 for the results obtained in Steps 1 and 2 and should evaluate the results as follows:

When,

F_b (1.25) < F_r then the design is adequate.

(23,300 lbs)(1.25) = 29,125 lbs < 36,766 lbs = F_r The design is acceptable because F_b(1.25)< F_r. The downward force is greater than the uplift forces.

8.08.2.4 EXAMPLE PROBLEM #4: ANTI-SEEP COLLAR DESIGN

GIVEN:

Several proposed detention basins are required as part of a roadway project in Henry County, Tennessee. Basin #2 along the new route is in the final stages of development. The designer has performed detailed grading and storm routing for the proposed stormwater management facility. His design and analysis has yielded a typical cross-section cut along the principal spillway pipe as shown in Figure 8A-17.

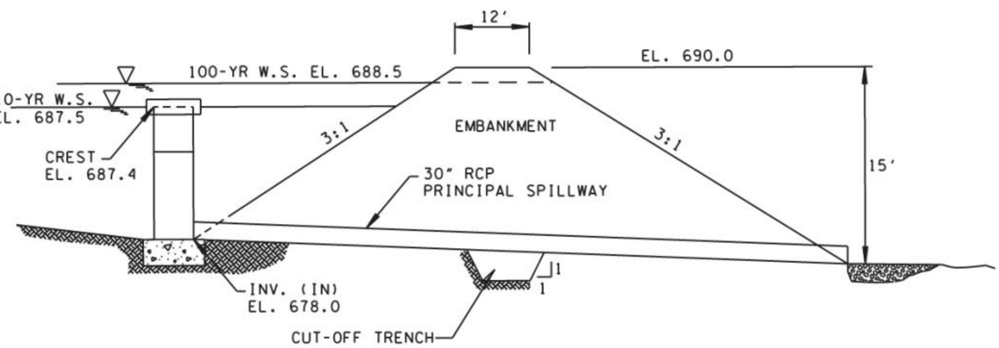


Figure 8A-17
Proposed Spillway Cross-Section
Basin #2, Henry County, TN

FIND:

Design anti-seep collars for the principal spillway barrel to increase the seepage length by 15% using both manual and graphical methods to obtain the results

SOLUTION:

The guidelines and design criteria for designing measures to minimize seepage and piping is described in Section 8.04.6.4 of the Chapter text.

Step 1: **Determine the Length of Principal Spillway Pipe within the Saturated Zone**

The length of pipe within the theoretical zone of saturation can be determined graphically on a "to-scale" drawing of the proposed cross-section as described in Section 8.04.6.4.1 or computed using Equation 8-2. The designer chooses to use the equation because a scale plot is not available for use. By Equation 8-2, the barrel length, L_S, within the theoretical saturation zone is computed as follows:

$$L_S = Y(Z+4)\times\left(1+\frac{S}{0.25-S}\right)$$

$$L_s = 9.5(3+4)\times\left(1+\frac{0.0322}{0.25-0.0322}\right) = 76.3 \text{ feet}$$

Where: Y=Computed 10-year maximum water surface elev. minus pipe inlet elevation
Y=687.5 – 678 = 9.5 feet

S=slope of the spillway pipe in feet per foot
S= (678-675) / 93’ = 0.0322 ft/ft

Step 2: **Determine the Required Seepage Length Increase**

The designer must increase the seepage length by 15 percent; therefore, the required seepage length increase can be computed by multiplying L_S by 15 percent (0.15) to determine the required total collar projection represented by the increase in seepage length.

Seepage length increase = (0.15) L_S = 11.44 feet

Step 3: **Determine the Size of the Anti-Seep Collars**

Before the size of the collars can be determined, the designer should assume a value for the required number of collars. For this site the designer first chooses 3 collars for the trial run of computations. With the number of collars estimated, the size of the anti-seep collars can be determined by solving the equation $(L_S+2nV)/L_S \geq 1.15$ for V as follows:

$$\left(\frac{L_s+2nV}{L_s}\right)\geq 1.15$$

$$\left(\frac{76.3+2(3)V}{76.3}\right)\geq 1.15$$

Solving the equation for V yields $6V = 11.44$ or $V = 1.91$ feet, use 2.0 feet

The computed collar projection of 1.91 feet does not meet minimum size requirements. As required, the minimum collar projection, V, from the spillway barrel shall be 2 feet in all directions. From this analysis, the designer would specify 3 collars along the barrel of the principal spillway. Each of the concrete collars would be 6.5 feet square. The collars would be spaced within the zone of saturation according to the spacing requirements of Section 8.04.6.4.1.

The designer should examine the results obtained and use professional judgment as to the practicality of the results. Construction of anti-seep collars tends to be labor intensive, and the use of multiple collars increases the potential for zones of poor compaction around the collars. The designer may consider reducing the number of collars by increasing the size of the collars. From Section 8.04.6.4.1, the minimum number of collars at this facility is 2 because of the embankment height.

Step 4: Re-Evaluate the Design Using 2 Anti-Seep Collars

Evaluating the spillway barrel for 2 collars using the following Equation, and solving for V:

$$\left(\frac{L_s + 2nV}{L_s}\right) \geq 1.15$$

$$\left(\frac{76.3 + 2(2)V}{76.3}\right) \geq 1.15$$

Solving the equation for V yields $4V = 11.44$ or $V = 2.86\,\text{feet}$

The computed collar projection, V = 2.86', will be rounded to 3 feet.

Using these results, the designer will specify two equally spaced 8'-6" (2.5-foot pipe diameter + 3 foot projection each side) square concrete anti-seep collars along the spillway barrel to provide a 15% increase in seepage length.

Step 5: Design the Collars Using Graphical Methods

With the length of conduit in the saturation zone, L_S, previously determined in Step 1, and using Figure 8A-18, enter the graph at the bottom of Figure 8A-18 at the point along the y-axis corresponding to L_S. Draw a horizontal line to the point where it intersects with the line that matches the desired number of collars. Initially, 3 collars were examined in the above calculations. From the intersection point with the 3-collar line, draw a line vertically to the upper chart, passing through the collar projection value along the top of the lower chart.

The vertical line should be extended into the upper chart until it intersects with the line corresponding to the pipe (barrel) diameter of the principal spillway. Interpolate the intersection point for pipe sizes not directly shown on the chart. From that point, draw a horizontal line to the right end of the upper chart to read the required size of the collars.

By graphical analysis, the collar projection was determined to be approximately 1.91 feet, and the final size of each collar found to be about 6.3 feet square. This is the value computed in Step 3, neglecting rounding. If this is too large, the number of collars should be increased, with the size of each collar being decreased. For this example, the number of collars is decreased with the size of each being increased.

The graphical solution steps above are repeated for the 2 collar design and the results of both runs are shown graphically in Figure 8A-18.

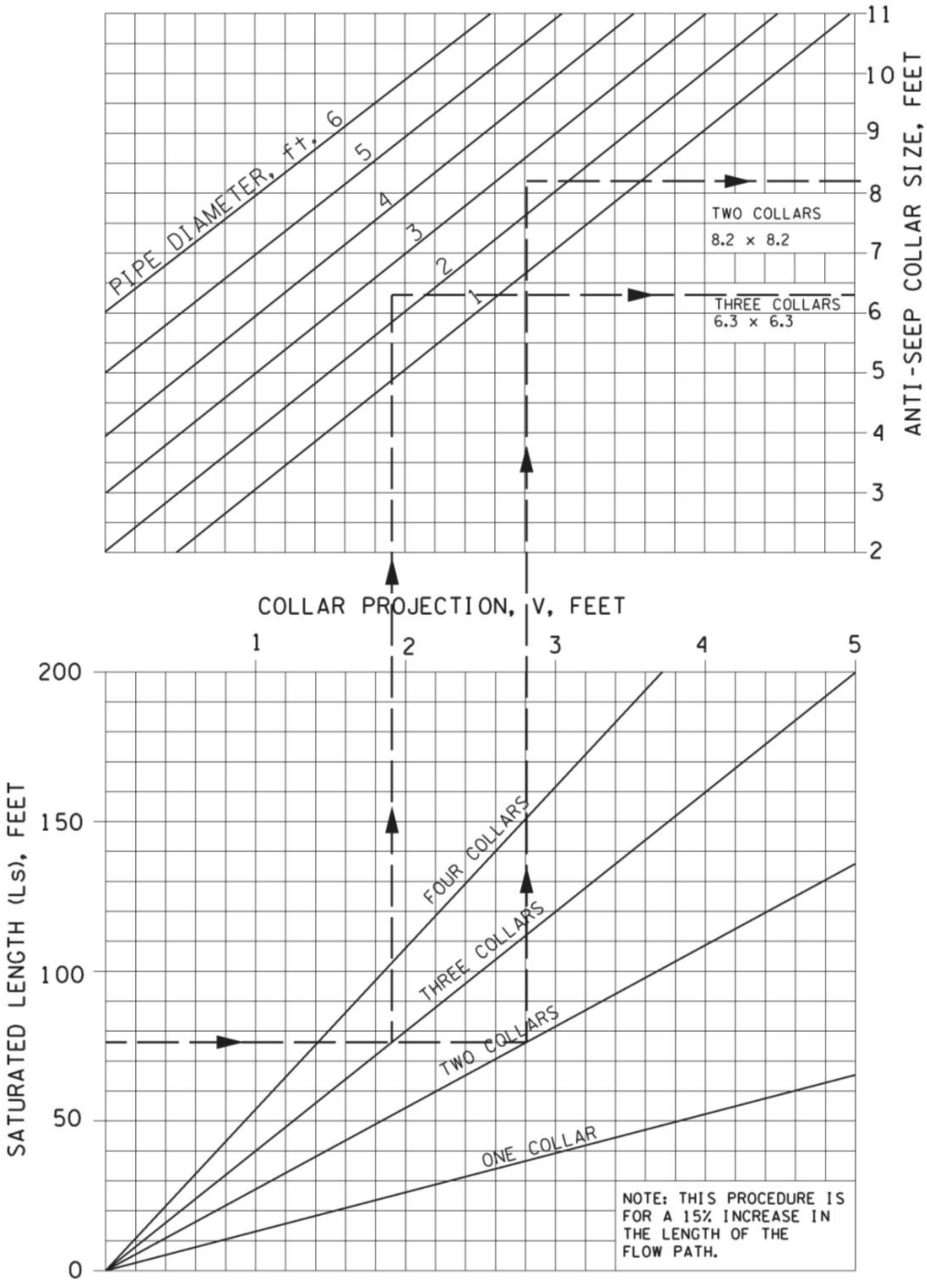


Figure 8A-18
Graphical Anti-Seep Collar Design for
Example Problem # 4

8.08.2.5 EXAMPLE PROBLEM #5: DETENTION BASIN DESIGN USING TR-20

GIVEN:

An existing 4-lane road passes through 58.7-acre watershed in central Tennessee. A new interstate highway is proposed to have an interchange with the existing road. Together, the new interstate and interchange will cover 40.2 acres within the watershed. Existing land uses within the watershed are rural, and flows at the outlet are contained in a swale which does not appear to have a defined channel. Inspection of the quadrangle map for the site shows that the area does not include a “blue line” stream.

The swale in which the basin is to be constructed may be represented as having a trapezoidal cross section with a 4-foot bottom width and 20:1 side slopes. The slope of the swale is 0.7%. The surface cover consists of rocks, grass and weeds and a Manning’s n-value of 0.055 has been determined for the site.

Curve numbers for the pre-project and post-project conditions have been computed based on the following table:

	Existing **Areas in Acres, by Soil Group and Land Use**		***Proposed*** **Areas in Acres, by Soil Group and Land Use**	
Land Use	**Soil Group B**	**Soil Group C**	**Soil Group B**	**Soil Group C**
Wooded	1.2	11.9		6.7
Cultivated	13.7	10.2	2.1	
Pasture	7.7	12.2	7.0	2.7
Paved	0.8	1.0	4.5	7.6
Grass			9.8	18.3
	Total Area:	58.7	Total Area:	58.7
	Curve Number:	71.8	Curve Number:	74.7

Table 8A-13
Curve Number Computations Summary for
Example Problem #5

The proposed interchange will channelize a portion of the existing swale and will somewhat shorten the flow length. Thus, the time of concentration computations are summarized in the following table:

	Existing Conditions	Proposed Conditions
Flow Length (feet)	2513	2484
Fall (feet)	39.5	39.5
Computed T_c (hours)	0.69	0.52

Table 8A-14
Time of Concentration Computations Summary for
Example Problem #5

Because of the nature of the swale at the drainage area outfall, water quality measures will not be required at this site.

FIND:

Design a detention basin at the drainage area outfall which will serve to meet the detention criteria provided Section 8.04.2. At a minimum, the basin should ensure that the 2-year and 10-year peak outflow rates from the site will not be increased. Additionally, it should allow flows from the 100-year flood to pass through the basin without overtopping the embankment. The basin is to be analyzed using the computer program TR-20.

SOLUTION:

TR-20 is a powerful computer program which can be used to analyze a variety of hydrologic design issues, including detention design. Although this sample problem is not intended to provide comprehensive instructions on how to use the program, some limited discussion will be provided on important features of the program related to detention design. TR-20 is based on “old-style” FORTRAN programming; thus, data in the input file must be entered into specific columns for the program to interpret the input data properly. Detailed information on the use of the program may be found in the TR-20 User’s Manual which is available on the NRCS website.

Step 1: The essential data required for developing a TR-20 computer model are:

- rainfall depths
- rainfall distribution
- drainage area
- curve number
- time of concentration

Since the drainage area, curve numbers and times of concentration have been provided in the “given” statement, all that remains to be determined are the rainfall depths and distribution.

As discussed in Section 8.04.2, detention design is to be based upon controlling the 2-year and 10-year 24-hour storm events and passing the 100-year, 24-hour rainfall event safely through the basin.

Based on rainfall depths provided in the Appendix of Chapter 4, the 24-hour storm depths are found to be:

- 2-year: 3.7 inches
- 10-year: 5.1 inches
- 100-year: 6.9 inches

Since rainfall intensity usually varies during the duration of a given storm, a rainfall distribution is used to describe what portion of the total rainfall depth falls during each portion of the total storm duration. Rainfall distributions are usually expressed as a graph of the percentage of the total storm depth versus the percentage of total storm duration. Figure 4-4 illustrates a "balanced" distribution which may be used for a 24-hour storm. Section 4.04.5.3.2 of this Manual recommends that the NRCS Type II distribution be used in analyses with TR-20. The data for this rainfall distribution has been "built in" to the program.

Step 2: The pre-project data determined above may now be assembled into a TR-20 data file to create an "existing conditions" model. Figure 8A-19 shows the completed TR-20 input file for this problem.

```
JOB TR-20                                 SUMMARY
TITLE 001 TDOT Drainage Manual Chpater 8 Detention Sample Problem #5 using TR-20
TITLE 001 Existing Conditions, 2, 10 and 100-year storms, NRCS Type II Dist.
 6 RUNOFF 1 001       1 0.0917      71.8        0.69        1 1 0 1 1 1
   ENDATA
 7 INCREM 6             .10
 7 COMPUT 7 001   001   0.0         3.7         1.0         2 2  01  02 2YR 24HR
   ENDCMP 1
 7 COMPUT 7 001   001   0.0         5.1         1.0         2 2  01  10 10YR 24HR
   ENDCMP 1
 7 COMPUT 7 001   001   0.0         6.9         1.0         2 2  01  99 100YR 24HR
   ENDCMP 1
   ENDJOB 2
```

Figure 8A-19
TR-20 Existing Conditions Input File

A typical TR-20 model usually consists of 4 parts. The first part includes the job and title cards which are used to describe the analysis and set program options. The second part is the tabular data, which is used to provide the program with rainfall distribution and hydrologic routing data. Since this model uses a "built-in" distribution, the input data file does not contain tabular data.

The third section is the "Standard Control" section, which describes the physical watershed which is being modeled. The Standard Control for this problem consists of a single RUNOFF statement, which is used to process rainfall on a given watershed into runoff. As can be seen, the data provided on the RUNOFF statement include the drainage area (0.0917 square miles), the curve number (71.8), and the time of concentration (0.69 hours). Since the existing conditions model includes only a single watershed, only one standard control statement is needed, and this section is concluded with an "ENDATA" statement.

The fourth section is the "Executive Control" section, which is used to specify the computation time interval and to describe the storm events to be modeled. In this model, each storm is represented by a single COMPUT statement. Data provided on the COMPUT statements includes the storm start time (0.0 hours for all three), the total rainfall depth and the storm duration. In this model, the storm duration has been set to 1.0 for all three storms. The NRCS Type II rainfall distribution is specifically a 24-hour distribution. Thus, the 1.0 entered in the COMPUT statements serves as a place-keeper since the program will automatically use a duration of 24 hours. The next number in the COMPUT statements is a “2” which signals the program to use the NRCS type II distribution, followed by the antecedent moisture condition (also a “2”), the alternate number (01), the storm number (which for this model has been set equal to the storm frequency), and a short description of the storm. This section is ended by the use of an ENDJOB statement, which also ends the data file.

The output generated by a single run of TR-20 can be substantial. The output normally includes an “echo” of the input data, followed by detailed printouts of the hydrographs (flow or elevation values versus time) computed for each statement in the standard control for each storm event. The last section of the output file contains a variety of summary tables. Figure 8A-20 shows a portion of the detailed output for the 2-year, 24-hour storm event. Computed peak values for the hydrograph are presented above the detailed hydrograph data.

```
TR20 ---------------------------------------------------------------------- SCS -
TDOT Drainage Manual Chapter 8 Detention Sample Problem using  VERSION
06/16/**  Existing Conditions, 2, 10 and 100-year storms, NRCS Type II D2.04TEST
08:32:16                    PASS   1   JOB NO.   1                      PAGE   2



EXECUTIVE CONTROL  INCREM    MAIN TIME INCREMENT =   .100 HOURS


EXECUTIVE CONTROL  COMPUT    FROM XSECTION   1  TO XSECTION   1         2YR 24HR
STARTING TIME =   .00       RAIN DEPTH =  3.70       RAIN DURATION =  1.00
ANT. RUNOFF COND. = 2       MAIN TIME INCREMENT =   .100 HOURS
ALTERNATE NO. = 1           STORM NO. = 2            RAIN TABLE NO. = 2


OPERATION RUNOFF    XSECTION   1

PEAK TIME(HRS)             PEAK DISCHARGE(CFS)          PEAK ELEVATION(FEET)
12.33                          46.3                       (RUNOFF)

HYDROGRAPH POINTS FOR   ALTERNATE = 1,   STORM = 2
HRS       MAIN TIME INCREMENT =  .100 hr,      DRAINAGE AREA =    .09 SQ.MI.
11.30 CFS       .37      .56      .83    1.25    2.09    4.18    9.45   18.90
12.10 CFS     31.06   41.38   46.05   45.01   40.04   33.44   27.42   22.77
12.90 CFS     19.14   16.32   14.12   12.40   11.05    9.98    9.11    8.40
13.70 CFS      7.81    7.31    6.86    6.45    6.09    5.78    5.52    5.30
14.50 CFS      5.12    4.96    4.82    4.71    4.60    4.50    4.41    4.32
(continues)...
```

Figure 8A-20
Excerpt from Detailed TR-20 Output

Based on the detailed outputs, the following existing conditions runoff rates were computed for this watershed:

Frequency (years)	Flow Rate (cfs)
2	46.3
10	88.4
100	150.1

Table 8A-15
TR-20 Results for Existing Conditions

As shown in Figure 8A-21, the summary tables provide a more compact and more general printout of the model results.

```
                             SUMMARY TABLE 1
                             ---------------
   SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL IN ORDER PERFORMED.
   A CHARACTER FOLLOWING THE PEAK DISCHARGE TIME AND RATE (CFS) INDICATES:
F-FLAT TOP HYDROGRAPH   T-TRUNCATED HYDROGRAPH   R-RISING TRUNCATED HYDROGRAPH

XSECTION/  STANDARD                                    PEAK DISCHARGE
STRUCTURE   CONTROL    DRAINAGE   RUNOFF   ------------------------------------
   ID      OPERATION     AREA     AMOUNT   ELEVATION   TIME     RATE    RATE
                        (SQ MI)    (IN)       (FT)     (HR)     (CFS)   (CSM)

RAINFALL OF   3.70 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.
RAINTABLE NUMBER  2,    ARC 2
MAIN TIME INCREMENT   .100 HOURS

   ALTERNATE    1   STORM    2
   ---------------------------
XSECTION   1  RUNOFF      .09     1.24        ---     12.33       46    511.1

RAINFALL OF   5.10 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

   ALTERNATE    1   STORM   10
   ---------------------------
XSECTION   1  RUNOFF      .09     2.26        ---     12.32       88    977.8

RAINFALL OF   6.90 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

   ALTERNATE    1   STORM   99
   ---------------------------
XSECTION   1  RUNOFF      .09     3.73        ---     12.30      150   1666.7
```

Figure 8A-21
Excerpt from TR-20 Summary Output

Once the existing conditions discharges have been determined, the design of the proposed detention basin can begin.

Step 3: One of the types of data needed for a detention analysis is a rating curve of the storage volume in a basin versus elevation, or stage-storage curve. Often this data is determined by measuring the surface area of water stored in the proposed basin for successive elevations and computing volume based on the average surface area between two given elevations. Data used in this sample problem are shown in the table on the following page.

Step 4: Another type of data needed for a detention model is an analysis of the performance of the proposed outlet works, or stage-discharge curve. In this problem, the proposed outlet will consist of a 48-inch reinforced concrete pipe with a low "riser" placed at the inlet.

The 48-inch outlet pipe is proposed to be placed on the grade of the existing swale. It will be approximately 50 feet long, have an inlet elevation of 548.20 feet and an outlet elevation of 547.85 feet, for a slope of 0.7%. The performance of this pipe is analyzed using the computer program HY-8. The HY-8 output for this pipe is included at the end of this problem as Figure 8A-27.

The proposed riser will consist of a reinforced concrete box 3.6 feet high, with internal dimensions of 6 feet by 5 feet. A "V"-notch weir will be constructed in the upstream face of the riser. The vertex of the "V" will be at an elevation of 548.20 feet, which is equal to the invert of the 48-inch outlet pipe. The "V" notch will have an internal angle of 70° and will have a width of 5.0 feet at a height of 3.6 feet. At depths in the detention basin below 3.6 feet, outflows will tend be controlled by the "V"-notch weir. However, since the vertex of the weir is at the same elevation as the pipe inlet, flows through the weir will be at least partially submerged. At depths greater than 3.6 feet, the two sides of the riser will begin to act as weirs.

Again, because of backwater from the 48-inch pipe, the weir flow across the sides of the riser will be at least partially submerged. As the flow rate increases, the headwater elevation due to flows through the pipe will become sufficiently great that it will tend to dominate the basin outflow. However, even at depths significantly above the top of the riser, its presence will create additional head losses as flows enter the pipe. Thus, flows through the proposed outlet will be somewhat less efficient than flows through the pipe alone.

Elevation (feet)	Surface Area (ac.)	Storage (ac-ft)
548.20	0	0.00
548.70	0.018	0.0046
549.20	0.078	0.0288
549.70	0.163	0.0891
550.20	0.288	0.2019
550.70	0.438	0.3835
551.20	0.630	0.6504
551.70	0.844	1.0189
552.20	1.102	1.5056
552.70	1.382	2.1267
553.20	1.706	2.8987
553.70	2.051	3.8379
554.20	2.430	4.9581
554.70	2.786	6.2621

Table 8A-16
Stage-Storage Rating Table for the Proposed Basin

The submerged weir flows across the riser were analyzed using methods recommended in the *Handbook of Hydraulics* (King, Brater, et. al.), page 5.18. Flow depths on the downstream side of the riser were evaluated using the HY-8 model shown in Figure 8A-27. The resulting stage-discharge rating table is presented in the Table 8A-17, as well as in Figure 8A-22.

Step 5: Using linear interpolation, the stage-storage and stage-discharge rating curves determined in Steps 3 and 4 are assembled into a structure rating table for TR-20. This rating data is entered into a TR-20 data file in the tabular data section as shown in Figure 8A-23. For each elevation in the table, a corresponding outflow rate in cfs and a storage volume in ac-ft are provided.

Basin Elevation (feet)	Outflow (cfs)
548.20	0.00
548.70	0.31
549.20	1.74
549.70	4.74
550.20	9.45
550.70	15.63
551.20	23.44
551.80	35.52
552.20	59.52
552.70	79.17
553.20	94.36
553.48	116.40
553.75	135.80
553.96	155.20
554.14	174.60
554.31	194.00

Table 8A-17
Stage-Discharge Rating Table for the Proposed Basin

It is possible for a TR-20 model to contain multiple storage basins, thus a structure number is assigned to each structure so that it can be identified in the standard and executive control statements of the data file. For this problem, the proposed detention basin is assigned structure number 01, and this number is shown in the header for the table of structure data in the input file. Since the standard control represents the physical layout of a watershed, a RESVOR statement has been added to the standard control statements in the proposed conditions model. The structure number 01 has been entered into this statement to connect the basin at this location with the tabular data entered above.

In addition, the executive control statements in the proposed conditions model are slightly modified as compared to the statements in the existing conditions model. Each COMPUT statement contains a starting cross section number and an ending cross section number. These numbers correspond to the cross section numbers entered for each statement in the standard control. In the existing conditions model, there was only one statement in the standard control, and it was at cross section 001. Thus, the COMPUT statements in the executive control start at cross section 001 and end at cross section 001. Since an additional standard control statement has been added to the proposed conditions model, the COMPUT statements for that model have been modified to start at cross section 001 and to end at structure 01.

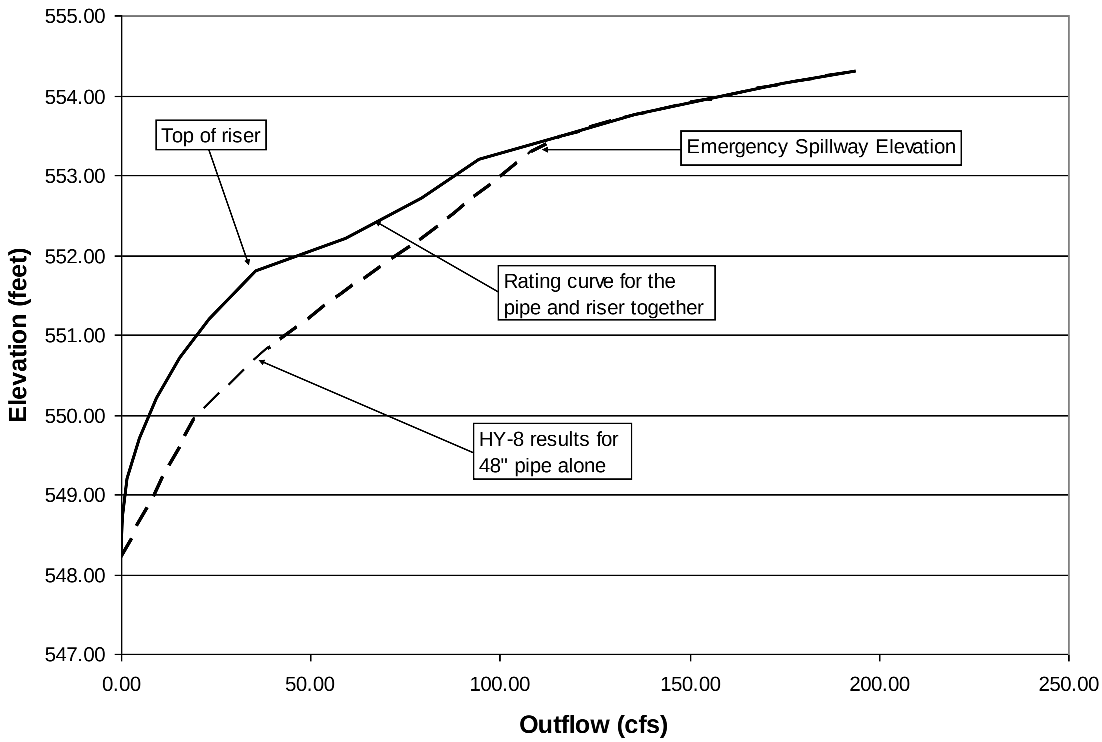


Figure 8A-22
Proposed Detention Basin Composite Outlet Rating Curve

As shown in Figure 8A-23, a “RESVOR” statement has been placed into the proposed conditions data file to represent the proposed detention basin. The only data required on this statement is the water surface elevation at the start of the rainfall event. Since this is to be a dry-bottom facility, the starting elevation is at the bottom of the basin.

```
JOB TR-20                                         SUMMARY
TITLE 001 TDOT Drainage Manual Chapter 8 Detention Sample Problem #5 using TR-20
TITLE 001 Proposed Conditions, 2, 10 and 100-year storms, NRCS Type II Dist.
 3 STRUCT      01
 8                      548.20      0.00        0.000
 8                      548.70      0.31        0.005
 8                      549.20      1.74        0.029
 8                      549.70      4.74        0.089
 8                      550.20      9.45        0.202
 8                      551.20      23.44       0.650
 8                      551.80      35.52       1.116
 8                      552.20      59.52       1.506
 8                      552.70      79.17       2.127
 8                      553.48      116.40      3.425
 8                      553.75      135.80      3.950
 8                      553.96      155.20      4.420
 8                      554.31      194.00      5.245
 9 ENDTBL
 6 RUNOFF 1 001       1 0.0917      74.7        0.52        1 1 0 1 1 1
 6 RESVOR 2    01 1   2 548.2                               1   1 1     Prop Basin
   ENDATA
 7 INCREM 6             .10
 7 COMPUT 7 001      01 0.0         3.7         1.0         2 2  01  02 2YR 24HR
   ENDCMP 1
 7 COMPUT 7 001      01 0.0         5.1         1.0         2 2  01  10 10YR 24HR
   ENDCMP 1
 7 COMPUT 7 001      01 0.0         6.9         1.0         2 2  01  99 100YR 24HR
   ENDCMP 1
   ENDJOB 2
```

Figure 8A-23
TR-20 Proposed Conditions Input File

Each statement in the standard control generates a hydrograph, which is assigned a number. Hydrograph numbers are used to control which hydrographs are used in subsequent standard control operations. In this case, the hydrograph generated by the RUNOFF statement is assigned number 1. In the following RESVOR statement, hydrograph number 1 is the “input” hydrograph, while outflows from the reservoir are assigned to hydrograph number 2.

The results of the proposed conditions model are summarized in Table 8A-18 below. The basin effectively controls the post-project flows so that basin discharges for the 2-year and 10-year events are slightly less than the existing runoff rates. It is interesting to note that the peak basin elevation for the 2-year flood is just somewhat higher than the elevation of the top of the riser, which is 551.80 feet. Thus, the riser provides the primary flow control for this event. In contrast, the 10-year peak water surface elevation is sufficiently high that backwater from the pipe is the primary control. It is apparent that while the 48-inch RCP would provide sufficient control for the 10-year flood, it would allow too much outflow in the 2-year event and would therefore not meet the detention criteria presented in this chapter. Thus, the riser is needed to control smaller rainfall events. Further, it can be seen that the basin succeeds in controlling the 100-year event as well. However, as discussed in Section 8.04.2, this is not a requirement for an acceptable basin design.

Frequency	Pre-Project Flow Rate (cfs)	Post-Project Inflow to Basin (cfs)	Peak Outflow from Basin (cfs)	Peak Water Surface Elevation (feet)
2	46.3	66.5	45.3	551.96
10	88.4	119.4	81.2	522.77
100	150.1	193.5	134.1	553.66

Table 8A-18
TR-20 Results for Proposed Conditions

An excerpt of the summary output for the proposed conditions run is provided in Figure 8A-24.

Step 6: As discussed in Section 8.04.7.4, the emergency spillway should be a minimum of 0.5 feet above the 10-year peak water surface elevation. In order to meet this criteria while passing as much of the 100-year flood as possible, the emergency spillway elevation was set at 553.30 feet. The emergency spillway is proposed to be trapezoidal in shape with a 20-foot bottom width and 3:1 side slopes. This information has been entered into the HY-8 data file included in Figure 8A-27, and the effect of the emergency spillway on the basin outlet rating can be seen in Figure 8A-22.

```
TR20 ---------------------------------------------------------------------- SCS -
          TDOT Drainage Manual Chapter 8 Detention Sample Problem #5 usi VERSION
06/24/**  Proposed Conditions, 2, 10 and 100-year storms, NRCS Type II D2.04TEST
11:38:51                     SUMMARY, JOB NO.   1                      PAGE   9


                             SUMMARY TABLE 1
                             ---------------
    SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL IN ORDER PERFORMED.
    A CHARACTER FOLLOWING THE PEAK DISCHARGE TIME AND RATE (CFS) INDICATES:
 F-FLAT TOP HYDROGRAPH   T-TRUNCATED HYDROGRAPH   R-RISING TRUNCATED HYDROGRAPH

 XSECTION/  STANDARD                                   PEAK DISCHARGE
 STRUCTURE   CONTROL    DRAINAGE   RUNOFF   -----------------------------------
    ID      OPERATION     AREA     AMOUNT   ELEVATION   TIME     RATE    RATE
                        (SQ MI)     (IN)       (FT)      (HR)     (CFS)   (CSM)

 RAINFALL OF   3.70 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.
 RAINTABLE NUMBER  2,    ARC 2
 MAIN TIME INCREMENT   .100 HOURS

    ALTERNATE    1   STORM    2
    ---------------------------
 XSECTION   1  RUNOFF      .09     1.42        ---     12.22       66    733.3
 STRUCTURE  1  RESVOR      .09     1.42     551.96     12.45       45    500.0

 RAINFALL OF   5.10 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

    ALTERNATE    1   STORM   10
    ---------------------------
 XSECTION   1  RUNOFF      .09     2.50        ---     12.20      119   1322.2
 STRUCTURE  1  RESVOR      .09     2.50     552.76     12.43       82    911.1

 RAINFALL OF   6.90 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

    ALTERNATE    1   STORM   99
    ---------------------------
 XSECTION   1  RUNOFF      .09     4.03        ---     12.20      193   2144.4
 STRUCTURE  1  RESVOR      .09     4.03     553.66     12.44      129   1433.3
```

Figure 8A-24
TR-20 Proposed Conditions Summary Output

The TR-20 results for the 100-year event indicate that the basin would reach an elevation of 553.66 feet. This corresponds to a depth of 0.36 feet on the emergency spillway, which is less than the recommended maximum flow depth of 1 foot specified in Section 8.04.7.1. Thus, the proposed spillway cross section should be adequate.

The emergency spillway should be provided with a lining which will provide adequate erosion protection in the 100-year flood. Based on a pool elevation of 553.66 feet in the basin, it is computed that the flow over the spillway will be 11.7 cfs (see discussion in Step 7) which will flow on a 3:1 slope. It is proposed to line the spillway with sod, which corresponds to vegetal retardance class “C.” Using the methods provided in Chapter 5 for analyzing flow in vegetated channels, it is determined that the flow depth will be 0.21 feet for an effective n-value of 0.11 and a flow velocity of 2.69 ft/sec. This result is compared to the allowable velocity values provided in Table 8A-6 for vegetated spillways, and it is noted that the allowable velocity for sod forming grasses is 4 ft/sec. for slopes up to 10%. Since the proposed spillway will be at a slope of 33%, the actual allowable should be somewhat lower; however, for spillways which operate only in floods greater that the 10-year event, the allowable velocity may be increased 25%. Thus, it is judged that a sod lining will be adequate.

Step 7: To finalize the proposed detention basin design, the adequacy of the emergency spillway should be tested for the event that debris were to block the principle outlet. To be conservative, it is assumed that essentially no flow is possible through the 48-inch culvert under the embankment. This will allow the elevation of the crest of the embankment to be determined.

The analysis of flows over the embankment is conducted using Equation 8-22 for broad-crested weirs. From Table 8A-1 in the Appendix, a value of 2.6 is selected for the weir coefficient, "C." Since the proposed spillway is trapezoidal, the length of the weir will increase as the depth of flow over the weir increases. In addition, the depth of flow on the sloping sides of the trapezoid will be less than the depth on the base of the spillway. Thus, for increasing levels of flow over the spillway, an effective weir length and average head, H, are used to compute flow over the emergency spillway as shown in the table below.

Using linear interpolation, the results of the weir flow computations for the emergency spillway are entered into the TR-20 structure data table shown in Figure 8A-25. Until the crest elevation of the emergency spillway (553.30 feet) is reached, very little flow is allowed from the detention basin. Note that TR-20 requires increasing discharge values for increasing elevation; small amounts of increasing flow are allowed until the emergency spillway elevation is reached.

```
JOB TR-20                                         SUMMARY
TITLE 001 TDOT Drainage Manual Chapter 8 Detention Sample Problem #5 using TR-20
TITLE 001 Outlet plugged, 2, 10 and 100-year storms, NRCS Type II Dist.
 3 STRUCT      01
 8                      548.20      0.00        0.000
 8                      549.20      0.01        0.029
 8                      550.20      0.02        0.202
 8                      551.20      0.03        0.650
 8                      551.80      0.04        1.116
 8                      552.20      0.05        1.506
 8                      552.70      0.06        2.127
 8                      553.30      0.07        3.087
 8                      553.45      3.06        3.368
 8                      553.68      12.54       3.800
 8                      554.00      32.16       4.488
 8                      554.75      103.50      6.403
 8                      555.50      202.00      8.730
 9 ENDTBL
 6 RUNOFF 1 001       1 0.0917      74.7        0.52        1 1 0 1 1 1
 6 RESVOR 2    01 1   2 548.2                               1   1 1     Prop Basin
   ENDATA
 7 INCREM 6             .10
 7 COMPUT 7 001      01 0.0         3.7         1.0         2 2  01  02 2YR 24HR
   ENDCMP 1
 7 COMPUT 7 001      01 0.0         5.1         1.0         2 2  01  10 10YR 24HR
   ENDCMP 1
 7 COMPUT 7 001      01 0.0         6.9         1.0         2 2  01  99 100YR 24HR
   ENDCMP 1
   ENDJOB 2
```

Figure 8A-25
TR-20 Input File for Testing Emergency Spillway Capacity
Operating in a "Stand-Alone" Condition

An excerpt from the summary output for this TR-20 run is shown in Figure 8A-26. The peak flow rate over the emergency spillway with the outlet plugged was computed to be 116.4 cfs at a peak water surface elevation of 554.85 feet.

As described in Section 8.04.5.3, a minimum freeboard of 1 foot should be maintained above the peak water surface in the basin for the 100-year flood hydrograph. In Step 6, this elevation was computed to be 553.66 feet, and normally this would require an embankment elevation of 554.66 feet. However, it is judged that the embankment should offer at least 0.5 feet of freeboard above the elevation of the basin when the outlet is plugged. Thus, the final embankment elevation will be set at 555.40 feet.

```
TR20 -------------------------------------------------------------------- SCS -
          TDOT Drainage Manual Chapter 8 Detention Sample Problem #5     VERSION
06/28/**  Outlet plugged, 2, 10 and 100-year storms, NRCS Type II Dist. 2.04TEST
16:20:36                    SUMMARY, JOB NO.   1                         PAGE  12


                            SUMMARY TABLE 1
                            ---------------
    SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL IN ORDER PERFORMED.
    A CHARACTER FOLLOWING THE PEAK DISCHARGE TIME AND RATE (CFS) INDICATES:
 F-FLAT TOP HYDROGRAPH   T-TRUNCATED HYDROGRAPH   R-RISING TRUNCATED HYDROGRAPH

 XSECTION/  STANDARD                                   PEAK DISCHARGE
 STRUCTURE   CONTROL    DRAINAGE   RUNOFF   ------------------------------------
    ID      OPERATION     AREA     AMOUNT   ELEVATION   TIME     RATE    RATE
                        (SQ MI)    (IN)       (FT)      (HR)     (CFS)   (CSM)

 RAINFALL OF   3.70 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.
 RAINTABLE NUMBER  2,    ARC 2
 MAIN TIME INCREMENT   .100 HOURS

    ALTERNATE    1   STORM    2
    ---------------------------
 XSECTION   1  RUNOFF      .09     1.42        ---     12.22       66   733.3
 STRUCTURE  1  RESVOR      .09      .82     553.61     13.33       10   111.1

 RAINFALL OF   5.10 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

    ALTERNATE    1   STORM   10
    ---------------------------
 XSECTION   1  RUNOFF      .09     2.50        ---     12.20      119  1322.2
 STRUCTURE  1  RESVOR      .09     1.90     554.19     12.62       50   555.6

 RAINFALL OF   6.90 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

    ALTERNATE    1   STORM   99
    ---------------------------
 XSECTION   1  RUNOFF      .09     4.03        ---     12.20      193  2144.4
 STRUCTURE  1  RESVOR      .09     3.42     554.85     12.48      116  1288.9
```

Figure 8A-26
Excerpt of the Summary Output File for Testing Emergency Spillway Capacity
Operating in a “Stand-Alone” Condition

```
1

CURRENT DATE: 06-24-2004                                FILE DATE: 06-24-2004
CURRENT TIME: 11:22:52                                  FILE NAME: OUTLET

ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ   FHWA CULVERT ANALYSIS    ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ     HY-8, VERSION 6.1      ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
ÚÄÄÄÂÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÂÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¿
³ C ³        SITE DATA             ³          CULVERT SHAPE, MATERIAL, INLET      ³
³ U ÃÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ´
³ L ³  INLET   OUTLET  CULVERT ³ BARRELS                                          ³
³ V ³  ELEV.    ELEV.  LENGTH  ³ SHAPE        SPAN   RISE  MANNING   INLET      ³
³NO.³  (ft)     (ft)    (ft)   ³ MATERIAL     (ft)   (ft)     n      TYPE       ³
³ 1 ³ 548.20   547.85   50.00 ³ 1 RCP        4.00   4.00   .013   CONVENTIONAL³
³ 2 ³                          ³                                                  ³
³ 3 ³                          ³                                                  ³
³ 4 ³                          ³                                                  ³
³ 5 ³                          ³                                                  ³
³ 6 ³                          ³                                                  ³
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SUMMARY OF CULVERT FLOWS (cfs)         FILE: OUTLET           DATE: 06-24-2004

 ELEV (ft)   TOTAL      1       2       3       4       5       6  ROADWAY ITR
   548.20      0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.00  1
   549.93     19.4    19.4     0.0     0.0     0.0     0.0     0.0     0.00  1
   550.83     38.8    38.8     0.0     0.0     0.0     0.0     0.0     0.00  1
   551.51     58.2    58.2     0.0     0.0     0.0     0.0     0.0     0.00  1
   552.15     77.6    77.6     0.0     0.0     0.0     0.0     0.0     0.00  1
   552.51     88.0    88.0     0.0     0.0     0.0     0.0     0.0     0.00  1
   553.48    116.4   112.4     0.0     0.0     0.0     0.0     0.0     3.96  3
   553.75    135.8   118.3     0.0     0.0     0.0     0.0     0.0    17.05  4
   553.96    155.2   122.7     0.0     0.0     0.0     0.0     0.0    31.24  3
   554.14    174.6   126.5     0.0     0.0     0.0     0.0     0.0    46.98  3
   554.31    194.0   129.8     0.0     0.0     0.0     0.0     0.0    63.25  3
   553.30    108.3   108.3     0.0     0.0     0.0     0.0     0.0 OVERTOPPING
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  SUMMARY OF ITERATIVE SOLUTION ERRORS   FILE: OUTLET         DATE: 06-24-2004

       HEAD            HEAD           TOTAL          FLOW           % FLOW
       ELEV (ft)     ERROR (ft)     FLOW (cfs)     ERROR (cfs)       ERROR
       548.20          0.000           0.00          0.00             0.00
       549.93          0.000          19.40          0.00             0.00
       550.83          0.000          38.80          0.00             0.00
       551.51          0.000          58.20          0.00             0.00
       552.15          0.000          77.60          0.00             0.00
       552.51          0.000          88.00          0.00             0.00
       553.48         -0.001         116.40          0.05             0.04
       553.75         -0.004         135.80          0.48             0.35
       553.96         -0.010         155.20          1.23             0.79
       554.14         -0.008         174.60          1.12             0.64
       554.31         -0.006         194.00          0.99             0.51
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  <1> TOLERANCE (ft) = 0.010                   <2> TOLERANCE (%) = 1.000
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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Figure 8A-27a
HY-8 Output for 48-inch RCP for
Example Problem #5

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CURRENT DATE: 06-24-2004                                        FILE DATE: 06-24-2004
CURRENT TIME: 11:22:52                                          FILE NAME: OUTLET
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      PERFORMANCE CURVE FOR CULVERT 1 - 1(  4.00 (ft) BY   4.00 (ft)) RCP
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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DIS- CHARGE FLOW (cfs)	HEAD- WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	548.20	0.00	0.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
19.40	549.93	1.73	1.73	1-S2n	1.07	1.29	1.11	0.77	6.82	1.29
38.80	550.83	2.63	2.63	1-S2n	1.56	1.85	1.62	1.03	8.16	1.53
58.20	551.51	3.31	3.31	1-S2n	1.96	2.29	2.03	1.21	9.09	1.70
77.60	552.15	3.95	3.95	1-S2n	2.34	2.66	2.41	1.36	9.81	1.83
88.00	552.51	4.31	4.31	5-S2n	2.54	2.84	2.62	1.43	10.11	1.88
112.39	553.48	5.28	5.28	5-S2n	3.07	3.20	3.00	1.60	11.14	2.02
118.27	553.74	5.54	5.54	5-S2n	3.22	3.26	3.16	1.70	11.11	2.10
122.73	553.96	5.76	5.58	2-M2c	3.39	3.31	3.31	1.79	11.07	2.17
126.49	554.14	5.94	5.70	2-M2c	3.53	3.35	3.35	1.88	11.29	2.24
129.76	554.31	6.11	5.81	2-M2c	4.00	3.38	3.38	1.96	11.49	2.30

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        El. inlet face invert     548.20 ft   El. outlet invert   547.85 ft
        El. inlet throat invert     0.00 ft   El. inlet crest       0.00 ft
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***** SITE DATA ***** CULVERT INVERT **************
      INLET STATION                          0.00 ft
      INLET ELEVATION                      548.20 ft
      OUTLET STATION                        50.00 ft
      OUTLET ELEVATION                     547.85 ft
      NUMBER OF BARRELS                      1
      SLOPE (V/H)                            0.0070
      CULVERT LENGTH ALONG SLOPE            50.00 ft

***** CULVERT DATA SUMMARY ************************
      BARREL SHAPE          CIRCULAR
      BARREL DIAMETER         4.00 ft
      BARREL MATERIAL       CONCRETE
      BARREL MANNING'S n    0.013
      INLET TYPE            CONVENTIONAL
      INLET EDGE AND WALL   GROOVED END PROJECTION
      INLET DEPRESSION      NONE

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Figure 8A-27b
HY-8 Output for 48-inch RCP for
Example Problem #5 *(continued)*

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CURRENT DATE: 06-24-2004                                 FILE DATE: 06-24-2004
CURRENT TIME: 11:22:52                                   FILE NAME: OUTLET

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ          TAILWATER          ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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******* REGULAR CHANNEL CROSS SECTION ****************
        BOTTOM WIDTH                                4.00 ft
        SIDE SLOPE  H/V (X:1)                       %20.0
        CHANNEL SLOPE V/H (ft/ft)                   0.007
        MANNING'S n (.01-0.1)                       0.055
        CHANNEL INVERT ELEVATION                  547.85 ft
        CULVERT NO.1 OUTLET INVERT ELEVATION      547.85 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

        FLOW     W.S.E.  FROUDE   DEPTH    VEL.   SHEAR
        (cfs)     (ft)   NUMBER    (ft)   (f/s)   (psf)
         0.00    547.85   0.000    0.00    0.00    0.00
        19.40    548.62   0.258    0.77    1.29    0.34
        38.80    548.88   0.267    1.03    1.53    0.45
        58.20    549.06   0.272    1.21    1.70    0.53
        77.60    549.21   0.276    1.36    1.83    0.60
        88.00    549.28   0.278    1.43    1.88    0.63
       116.40    549.45   0.282    1.60    2.02    0.70
       135.80    549.55   0.284    1.70    2.10    0.74
       155.20    549.64   0.286    1.79    2.17    0.78
       174.60    549.73   0.288    1.88    2.24    0.82
       194.00    549.81   0.289    1.96    2.30    0.86

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ  ROADWAY OVERTOPPING DATA  ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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      ROADWAY SURFACE                             GRAVEL
      EMBANKMENT TOP WIDTH                         12.00 ft
***** USER DEFINED ROADWAY PROFILE
      CROSS-SECTION      X          Y
      COORD. NO.         ft          ft
           1            0.00      555.30
           2          249.00      555.30
           3          255.00      553.30
           4          275.00      553.30
           5          281.00      555.30
           6          296.00      555.30
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Figure 8A-27c
HY-8 Output for 48-inch RCP for
Example Problem #5 *(continued)*

8.08.3 GLOSSARY

The following list of terms is representative of those used in stormwater management and design of stormwater storage facilities. All of the terms may not necessarily be used in the chapter text; but rather are commonly used by engineers, scientists, and planners.

ABUTMENT – The part of a valley side (wall) against which an embankment is constructed.

ANTI-SEEP COLLAR – An impermeable diaphragm usually of sheet metal or concrete constructed at intervals within the zone of saturation along the outside of a conduit through an embankment to increase the seepage length along the outer surface of the conduit, thereby reducing the potential for piping and seepage.

ANTI-VORTEX DEVICE – A device designed and placed on the top of a riser or the entrance of a pipe to prevent the formation of a vortex in the water at the entrance.

AQUATIC BENCH – A graded bench which is located around the inside perimeter of a permanent pool facility for enhanced safety and is normally vegetated with aquatic plants.

AXIS OF EMBANKMENT – The horizontal centerline of an embankment in the longitudinal direction.

BARREL – The closed conduit part of the principal spillway used to convey water under and through an embankment.

BASE FLOW – Discharge of water which is independent of surface runoff, usually dependent on groundwater levels.

BEST MANAGEMENT PRACTICE (BMP) – A structural or non-structural device designed to temporarily store or treat stormwater runoff to mitigate flooding and/or improve stormwater quality.

BLANKET DRAIN – A drain that generally extends in a horizontal direction under a relatively large area of the downstream portion of an embankment, and is used to prevent saturation of the downstream toe by intercepting seepage through the embankment and its foundation.

CONDUIT – Any open or closed channel intended for the conveyance of water through or under an embankment.

CONTROL SECTION – The component of a spillway that regulates the discharge from a stormwater basin. A control section prevents or limits discharge below fixed water levels and can regulate the quantity of discharge when the stage is above such fixed level.

CORE TRENCH – A constructed trench filled with relatively impervious material intended to reduce seepage of water through porous strata.

CREST – The top of the stormwater embankment, spillway, or weir structure, frequently restricted to the overflow portion.

CUTOFF TRENCH – A trench (keyway) constructed along the centerline (axis) of an embankment which is filled with relatively impervious material intended to reduce seepage of water through porous strata.

DESIGN STORM – A selected storm event described in terms of the probability of occurring once within a given number of years for which stormwater storage facilities or other flood control improvements are designed. For example, a 10-year, 24-hour duration storm event has a 10 percent probability of occurring in any given year. The precipitation associated with that storm would be measures over a 24 hour period.

DETENTION – The temporary storage of stormwater runoff in a BMP, prior to its gradual release, with the goal of controlling peak discharge rates.

DETENTION BASIN – A permanent stormwater management facility used for the temporary storage of runoff that is designed *not* to create a permanent pool of water.

DRAWDOWN – The lowering of water surface levels in a storage management facility resulting from the release of water from the facility.

DROP INLET – An over-fall structure in which water enters by overtopping a horizontal surface, drops through a vertical shaft, and then discharges to receiving waters through a connected conduit. In stormwater management, it is typically referred to as a riser.

EMERGENCY SPILLWAY – A spillway constructed in natural ground designed to discharge flows in excess of the principal spillway design discharge around an impoundment structure.

EVAPOTRANSPIRATION – The loss of water to the atmosphere through the combined process of evaporation and transpiration, the process by which plants release water they have absorbed.

EXTENDED DETENTION – A stormwater management facility designed to provide for the gradual release of a volume of water in order to increase settling of pollutants and protect downstream channels from frequent storm events. Normally dry between rainfall events.

FOREBAY – A storage space located at or near a BMP inlet which serves to capture incoming coarse sediment before it reaches and accumulates in the primary storage area.

FOUNDATION DRAIN – A pipe or series of pipes which improves embankment stability by collecting groundwater from the foundation of the embankment and discharges it at an acceptable location.

FREEBOARD – The vertical distance between the maximum water surface elevation anticipated during design and the top of a retaining structure. Freeboard is provided to prevent structure overtopping resulting from unforeseen conditions.

HEAD – The height of water above any plane of reference. The energy, either kinetic or potential, possessed by each unit weight of a liquid expressed as the vertical height through which a unit weight would have to fall to release the average energy possessed.

HYDRAULIC GRADIENT – The slope of the hydraulic grade line. Includes static and potential head.

HYDROGRAPH – A graphic representation showing variation in discharge or depth (stage) of a stream of water over a specified period of time. May be defined in terms of discharge from a stormwater basin or runoff from a watershed.

IMPERVIOUS AREA – The permanent surfaces in a drainage basin which cannot infiltrate rainfall. Areas typically consist of rooftops, sidewalks, pavements, slabs, driveways, etc…

NAPPE – The lower surface, (underside) of a free falling stream of water, typically over a weir.

NONPOINT POLLUTION – Contaminants, whereby the source cannot be definitively pinpointed, but rather washed from the surface via stormwater.

ONE HUNDRED YEAR STORM – The extreme flood event which occurs on average once every 100 years or statistically has a 1 percent chance on average of occurring in a given year.

PIPING – The progressive development of internal erosion by seepage. Removal of soil material through subsurface flow channels.

POROSITY – Ratio of open space volume to total solids volume.

PRINCIPAL SPILLWAY – The primary structure(s) which carries base flows and/or storm flows through the embankment of a stormwater management facility. A device constructed of permanent material to regulate discharge from a stormwater basin.

RETENTION – The amount of stormwater runoff from a drainage basin that does not escape a developed site as runoff. Described as the difference between total precipitation and total runoff. Retention basins are normally wet (permanent pool), even between rainfall events

RISER – A vertical pipe, box or other structure which extends from the bottom of a stormwater basin and contains the control devices used to achieve the desired discharge rates for a specified design.

SAFETY BENCH – A relatively flat area above the permanent pool and surrounding stormwater basin designed to provide a separation to adjacent side slopes.

SEEPAGE – The process by which water percolates through a permeable material. Water escaping or emerging from the ground. Seepage rate will depend on the permeability of the material in and under the structure, the depth of water behind the embankment, and the length of path the water must travel under or through an embankment.

SEEPAGE LENGTH – The length along the principal spillway conduit and around any anti-seep collars that is within the zone of saturation through an embankment.

SPILLWAY – An open or closed channel used to transport normal and/or excess stormwater discharges through or over a stormwater facility.

STORM ROUTING – The determination of the attenuating effect of storage on a storm passing though a stormwater management facility.

TEN YEAR STORM – The 24-hour storm event which occurs on average once every 10 years or has a 10 percent chance of occurring in a given one year period.

TRASH RACK – A screen, grill, grate, or other structural device installed at the intake of a pipe, spillway, or other hydraulic structure to prevent floating or submerged debris from entering the structure, thereby reducing the potential for clogging.

TRICKLING DITCH – Typically a concrete or paved channel from the inlet to the outlet of a stormwater management facility designed to carry low flow runoff or base flow directly to the outlet device of the basin without detention. Aids in preventing standing water in basin.

TWO YEAR STORM - The 24-hour storm event which occurs on average once every 2 years or has a 50 percent chance of occurring in a given one year period.

ULTRA-URBAN – Densely developed urban areas in which a minimal amount of pervious area exists.

UNDERDRAIN – Typically a small diameter perforated pipe which allows the bottom of an embankment and an extended detention basin to drain over an extended period of time.

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8.08.5 ABBREVIATIONS

ASCE – American Society of Civil Engineers
BMP – Best Management Practice
CMP – Corrugated Metal Pipe
DCR – Department of Conservation and Recreation (Virginia)
EM – Engineering Manual
FEMA – Federal Emergency Management Agency
GaSWCC – Georgia Soil and Water Conservation Commission
HDS-5 – Hydraulic Design Series Number 5
HEC-22 – Hydraulic Engineering Circular Number 22
HEC-HMS – Hydrologic Engineering Center Hydrologic Modeling System
HDPE – High Density Polyethylene Pipe
IDF – Intensity Duration Frequency
NEH-5 – National Engineering Handbook Number 5
NRCS – Natural Resource Conservation Service (USDA)
NVPDC – Northern Virginia Planning and Development Commission
OSHA – Occupational Safety and Health Administration
RCP – Reinforced Concrete Pipe
SCS – Soil Conservation Service
SWMM – Storm Water Management Manual
TDOT – Tennessee Department of Transportation
TIN – Triangulated Irregular Network
TP – Technical Paper
TR – Technical Release
USACE – United States Army Corps of Engineers
USDA – United States Department of Agriculture
USDOT – United States Department of Transportation
USGS – United States Geological Survey