City of Rocky Mount, North Carolina
Stormwater Design Manual

Chapter 1: General Design Criteria

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Disclaimer

To the best of their ability, the authors have insured that material presented in this manual is accurate and reliable. The design of engineered facilities, however, requires considerable judgment on the part of designer. It is the responsibility of the designer to insure that techniques utilized are appropriate for a given situation. The City of Rocky Mount therefore accepts no responsibility for any loss, damage, or injury as a result of the use of this manual.

Chapter 1: General Design Criteria

1.1 Introduction

The intent of this manual is to serve as a reference for City staff and practicing professionals in designing storm drainage facilities within the City of Rocky Mount (the City) and its extraterritorial jurisdiction. It is primarily a compilation of the City's accepted design procedures and practices. Design criteria listed herein are the general policy of the City of Rocky Mount and may not be applicable in every situation. Where the designer determines that conformance with this manual would create an unreasonable hardship or where an alternative design may be more appropriate, alternative designs may be accepted upon written authorization from the Director of Engineering or his designee. In order to insure good engineering design, the City staff may occasionally require more stringent standards than those presented here. This manual may also be subject to periodic change by the City staff. When changes are required, revisions will be made available via the City of Rocky Mount website.

Engineering construction specifications are contained in a separate manual entitled "City of Rocky Mount Department of Engineering Manual of Specifications", which can be obtained from the City Engineering Department.

Engineering standard details are available via the City of Rocky Mount website at http://www.ci.rocky-mount.nc.us/engineering/main.html.

The City of Rocky Mount Stormwater Design Manual consists of 3 Chapters. Chapter 1 – General Design Criteria discusses the overall minimum design criteria for storm drainage systems within the City's jurisdiction and the stormwater management criteria for new development. Chapter 2 – Structural Best Management Practices presents the acceptable structural measures that can be used to control stormwater along with minimum design and maintenance criteria. Chapter 3 – Design Calculations presents the acceptable hydrologic and hydraulic methodologies, parameters, and in some cases, equations that are required to demonstrate compliance with the minimum design criteria.

1.2 Stormwater Management Overview

Stormwater management in the City of Rocky Mount addresses issues related to the proper control of stormwater runoff in both quantity (site and roadway drainage and flood conveyance) and quality (nutrients and suspended solids). The City has adopted a variety of regulations, ordinances and policies that serve as the foundation of the City's design standards for the management of the quantity and quality of stormwater runoff. These standards are designed to protect the health and welfare of the residents of Rocky Mount, to protect the environment, and to protect those who live downstream from Rocky Mount. The following sections of this chapter present the minimum standards related to stormwater management within the City of Rocky Mount.

1.3 Plan Submittal Requirements

The City of Rocky Mount requires a separate site stormwater management plan submittal for all developments greater than ½ acre. The stormwater management plan submittal shall include the complete storm drainage system and all of the supporting calculations for review. All stormwater management plans shall include a completed City of Rocky Mount Stormwater Management Summary Sheet, unless the project is exempt or receives a written exemption from the Director of Engineering, and all of the relevant calculation forms provided in this stormwater design manual.

The stormwater management plan supporting information shall include the following:

- Location map.
- Overall site map showing the proposed site, surrounding properties, and zoning information.
- Existing condition drainage area map showing the existing stormwater outfall locations and existing land use. The drainage area map shall be the City topographic map unless more detailed topography is available. The existing drainage area map shall be large enough to show portions of the upstream and downstream drainage areas. Soil survey information shall be shown, including boundaries and hydrologic classification as identified by the NRCS
- Existing storm drainage pipes including offsite storm drainage pipes that either discharge onto the site, run parallel to the site, or will receive stormwater runoff from the site.
- Seasonal high water table, as needed for BMP design.
- Proposed condition drainage area map showing the proposed site improvements and the stormwater outfall locations. The proposed drainage area map shall be consistent with the catchment and outfall areas identified in the Stormwater Management Summary Sheet.
- All storm drainage catch basins, drop inlets, junction boxes and pipe locations and sizes along with the supporting inlet design chart, pipe design chart, and hydraulic grade line calculations as provided in Chapter 3 – Calculations.

- Profiles of all storm drainage pipe systems. The profiles shall show existing and proposed utility crossings.
- All open channel locations including vegetated swales, trapezoidal ditches, intermittent streams and perennial streams along with supporting design calculations.
- All culvert crossings locations and sizes with the supporting culvert calculation sheets.
- All structural BMPs and supporting calculations including hydrograph routings, water quality volumes, spillway and volume rating curves, outlet protection and other calculations identified in Chapter 2 – Structural BMP Design Criteria.
- Structural BMP operation and maintenance plan.
- All flood zones including those calculated as required by this manual. The floodplain and floodway boundaries shall be clearly labeled to identify the boundary source such as FEMA 100-year floodplain or calculated 100-year floodplain.
- All wooded pervious areas clearly identified with a metes and bounds description. A written conservation easement for the wooded pervious area must be executed prior to plan approval
- All proposed Finish Floor Elevations of buildings.
- All riparian or vegetated buffers shall be shown and clearly labeled.
- All existing and proposed drainage and utility easements.
- All existing wetlands, perennial and intermittent streams.
- All proposed site utilities in plan view.
- If the project is phased, a schedule for implementation of all proposed water quality BMPs that specifies when the BMP(s) will be on-line with respect to the development schedule for the drainage area serviced by the BMP.
- Certification by a North Carolina registered professional engineer, registered landscape architect, or registered land surveyor who is qualified in hydrology and hydraulics, stating that the plans comply with the standards in the City of Rocky Mount Stormwater Design Manual.
- Peak runoff calculations to each outfall leaving a site and any required BMP’s to meet peak runoff control requirements.

1.3.1 Record Drawings

Upon completion of the new construction, the developer is required to provide "record drawings", certified by a NC registered professional engineer, landscape architect, or land surveyor, prior to receiving an occupancy permit for the property.

1.4 General Design Criteria

New construction within the Rocky Mount jurisdictional area is subject to various City requirements. The following general requirements apply to both private development as well as City projects, unless the designer requests and receives approval for alternative designs from the Director of Engineering or his designee.

The City of Rocky Mount must approve all new or revised stormwater discharges from private property to a publicly maintained storm drainage system. The owner of the property or the developer may make written application to the City for stormwater discharge or may submit a stormwater management plan for City staff review under the terms of the Zoning Ordinance or General Plan Project Submittal Policy.

For all non-single family residential projects that are ½ acre or larger and single family residential projects that are 1 acre or larger, the owner or developer shall provide a stormwater management plan and supporting calculations to the City for review. The City reserves the right to require a drainage plan and calculations for projects less than a ½ acre if deemed necessary by the Director of Engineering.

A conveyance system shall be properly designed and installed when stormwater runoff is equal to or greater than 5 cubic feet per second (cfs) for the 10-year storm. A properly designed conveyance system may include a vegetated swale, open channel, catch basin with pipe, stream and floodplain, culvert or structural Best Management Practice (BMP).

When development of an area changes the flow regime from sheet flow to concentrated flow, the drainage system shall be designed to minimize impacts of the concentrated flow on adjoining properties by tying into existing drainage systems using multiple outlets, through agreements with adjacent owners, or other appropriate means.

No concentrated flow shall be discharged across walkways. Provisions are to be made through piping or other means to carry the flow under the walkway.

All stormwater drainage systems that will be maintained by the City of Rocky Mount and convey between 5 and 50 cfs shall be piped, unless those systems are Structural BMPs, intermittent or perennial streams.

No stormwater drainage shall be discharged into a sanitary sewer.

No utilities (sewers, power lines, water lines, etc.) shall be located within or under any stormwater management facility.

No utilities (sewers, power lines, water lines, etc.) shall be located over a storm drain line and along the same alignment unless approved by the Director of Engineering.

In no case shall a building be located within an impoundment area or over a storm drain line.

Where storm drainage lines cross or parallel other utility lines, appropriate clearances shall be provided according to the City of Rocky Mount Department of Engineering Manual of Specifications.

1.4.1 Easements

Drainage easements are required for all public and private drainage systems. This includes commercial developments with out-parcels, phased development and other developments with surrounding land under the same ownership as the tract being developed.

Drainage easements shall be provided:

- For all culverts, all new or existing open channels or watercourses that carry water from public rights-of-way or convey water from adjoining property across the developing property. Width of the easements shall be per the most recent City of Rocky Mount Standard Detail.
- For all stormwater BMPs, the BMP shall be located in a stormwater BMP easement with a minimum width of 15 feet beyond the top of bank or toe of slope and as necessary to provide access for maintenance.
- For all new or existing open channels or watercourses with peak flows of 15 cubic feet per second or more for the 10 year storm;
- At other locations deemed appropriate by the Director of Engineering or Engineering Department staff.

In addition:
Appropriate drainage easements must be secured prior to the submission of the construction plat if the easement(s) is entirely or partially located offsite.

Access easements, dedicated to the City of Rocky Mount, shall be provided for access and repair of velocity dissipaters, headwalls, and other structural portions of the drainage system located outside of the right-of-way (ROW) which are immediately adjacent to and directly associated with the City owned portion of the drainage system. These easements are requested to allow City staff access to repair and maintain those drainage facilities located immediately adjacent to the right-of-way which would endanger the roadway should they fail.

Adequate easements shall be provided to allow access of construction equipment, taking into consideration the limitations that may be imposed by embankment slopes or other obstacles.

Drainage easements containing only storm drainage facilities should be centered over the culvert or watercourse.

All drainage easements should be recorded based on field surveys, following construction, to insure that the drainage structure or watercourse is centered within the easement (unless specifically offset). Where this is not possible, a note shall be added to recorded plats establishing that easements are to be centered over the pipe or channel.

All drainage easements shall be designed to tie into existing easements, existing watercourses or to other appropriate locations when possible.

When other easements are not present along the property line, 5-foot easements for drainage and other general amenities are required around the entire perimeter of each lot adjacent to the lot property line.

1.4.2 Streets and Gutters

Gutters shall be designed in such a way that the spread of water during the 2-year storm does not exceed 6 feet into the travel lane. The travel lane does not include the gutter section.

When the street typical section includes a full shoulder or parking lane, no encroachment into the travel lane will be allowed.

For new streets located within or near a floodplain, the 100-year water surface elevation shall be no more than 2 feet above the low point of the road unless written approval is provided by the Director of Engineering.

No peak flow greater than 3 cfs during the 10-year storm may run down a driveway and into the street without the placement of a catch basin to intercept the flow.

Driveways that discharge 3 cfs or more during the 10-year storm into a roadside ditch shall provide outlet protection to prevent erosion of the ditch.

A minimum longitudinal slope of 0.4% shall be utilized unless approved otherwise by the Director of Engineering. When lesser slopes are encountered, the gutter shall be warped to provide the minimum slope.

New street crossings in the Tar-Pamlico riparian buffer areas shall be as close to a perpendicular angle as possible to minimize buffer disturbance.

No public or private roads are to be constructed on dams without the approval of the Director of Engineering.

1.4.3 Catch Basins, Drop Inlets and Junction Boxes

Catch basins shall be provided at street sags, up-grade of intersections, up-grade of super-elevation crossovers, and where driveways would discharge more than 3 cfs into a street for the 10-year storm.

Catch basins shall be provided in streets to intercept flow such that the spread conditions defined in 1.4.2 are not exceeded for the peak flow during the 2-year storm event.

Catch basin capacity and bypass shall be designed for the peak flow during the 2-year storm event with a 5 minute time of concentration.

Inlet capacity at sags, where relief by curb overflow is not provided, shall allow for debris blockage by providing twice the required computed opening for the 2-year storm.

No grate only type inlets are allowed in city streets.

Inlets are required on drainage systems discharging to a public street (including sheet flow) where the stormwater discharge is more than 3 cfs for the 10-year storm.

For combination grates and curb openings, ignore the curb opening on continuous grades to determine bypass and ignore the grate opening at the sag.

Limit the depth of ponding to 1 foot above the drop inlet grates located outside of the ROW during the inlet design storm.

A structure (catch basin, drop inlet or junction box) is required at all changes in grade or direction or at any pipe junction. Details shall be provided on the plans for all such structures.

Yard inlets that collect site stormwater runoff and convey the stormwater to the stormwater drainage system within the right of way shall be located outside of the right of way unless site conditions warrant otherwise.

Junction Boxes shall allow for access to the storm drainage system with a grate, manhole ring and cover, or a lid capable of being removed. No "blind boxes" are permitted.

Catch basins, drop inlets and junctions boxes shall have minimum drops between the upstream and downstream openings as provided in Table 1.1 below.

Table 1.1 - Minimum Structure Invert Drops

Change in Alignment	**Drop***
0 - 45 degrees	0.1 ft
45 – 90 degrees	0.2 ft
> 90 degrees (reverse flow conditions)	Only with detailed study and drop equal to or greater than the diameter of the pipe out
Change in Pipe Size	**Drop***
Increase in pipe size	Match the crown elevations
Decrease in pipe size	Only with a detailed study and special provisions for maintenance

* Structure invert drops that are less than the required minimum may be approved at the discretion of the Director of Engineering.

1.4.4 Storm Drainage Pipes

Storm drainage pipes convey stormwater runoff from areas such as streets, parking lots and grass areas underground to a receiving channel, stream or structural BMP. When stormwater runoff is conveyed from one side of a roadway to the other and flooding or failure can cause flow across the roadway then the conveyance system shall be designed as a culvert.

Storm drainage pipes shall be designed, at a minimum, to convey the future conditions peak flow during the 10-year storm event.

The design of storm drainage pipe systems shall consider where water flows when the storm drainage system cannot handle the stormwater runoff, such as storm events greater than the design storm. Section 1.5.2 identifies when floodplain boundaries shall be determined and shown on the plans.

For design of the storm drainage pipe it shall be assumed that the catch basin or drop inlet captures 100% of the peak flow during the 10-year storm event.

Minimum slopes for pipes 36 inches in diameter or less shall be 0.5%.

Minimum slopes for pipes greater than 36 inches in diameter shall be 0.3%

Minimum flow velocity in storm drainage pipes during the design storm event shall be 2.5 feet per second (fps).

All storm drainage pipes within the street ROW shall be reinforced concrete pipe Class III or higher.

All new pipes within easements that are to be maintained by the City shall be a minimum of 15 inches in diameter or equivalent. Privately maintained pipes within easements shall be a minimum of 12 inches in diameter or equivalent.

Maximum slope for reinforced concrete pipes is 12.0%**.** The Director of Engineering may approve greater slopes with the submittal of appropriate detailed structural designs and other supporting documentation.

Storm drainage pipes shall be designed such that the hydraulic grade line calculations demonstrate that when all tailwater conditions, pipe friction loss and all minor structure losses are considered, the hydraulic grade line remains at least 6-inches below the grate or gutter line.

Flared end sections are required at the inlet and outlet on all pipes 48 inches or less.

For multiple pipes 48-inches in diameter or less, a headwall may be used in lieu of multiple flared end sections.

Inlet and outlet headwalls are required on pipes larger than 48 inches in diameter.

If a pipe is located within the ROW, the minimum distance between the outside wall of the pipe and the ROW is 5 feet

Cover for pipes within the ROW shall be provided based on the following table:

Table 1.2 - Minimum Pipe Clearance

Pipe Size (in.)	Clearance Distance (ft) From Pipe Invert to Subgrade
15	2.4
18	2.7
24	3.3
30	3.8
36	4.4
42	4.9
48	5.4
54	6.0
60	6.5
66	7.0
72	7.6

Minimum cover for pipes outside of the ROW is 1 foot.

1.4.5 Open Channels

Open channels shall be designed to convey the future conditions peak flow during the 10-year storm event. The future condition land use shall be obtained from the City Planning Department.

New and existing open channels, with a 100-year discharge equal to or greater than 50 cfs and which are impacted by the proposed development, shall be designed to pass the 100-year storm unless the following criteria are met:

- The developer demonstrates that the 100-year discharge will not flood habitable structures or increase flood elevations on adjacent properties and the limits of the 100-year floodplain are determined and recorded.

- Existing natural channels serving do not have to be improved to carry the 10-year design flow but the limits of the 100-year floodplain must be established and recorded on the plat.

Private V-swales are acceptable for design discharges between 0 and 10 cfs. The swale shall be properly protected from erosion during the design event.

Trapezoidal swales are acceptable between 0-50 cfs. The trapezoidal swale shall be graded to a minimum of 3 feet bottom width with 3:1 side slopes and properly protected from erosion during the design event.

Open channels that convey more than 50 cfs during the 10-year design storm shall be designed as a combination channel. A combination channel shall have a separate low flow channel designed to adequately convey the 2-year storm event, without erosion, and a larger channel that can convey the 10-year design storm.

The minimum slope of soil ditches shall be 0.5%.

For open channels, gradual changes in alignment, not to exceed a minimum radius of 4 times the top width of the channel, is recommended. Where no other options are available, sharper changes in alignment may be allowed under the following conditions:

Table 1.3 – Open Channel Alignment

Open Channel Bend	Requirements
20 - 45 degrees	Bank stabilization must be provided according to tractive force analysis
>45 degrees	Same as for above but in addition, freeboard superelevation shall be calculated to demonstrate adequate channel depth on the outside bank.

Side slopes for vegetated open channels in residential areas should be no greater than 3 to 1 for stability, safety, and ease of maintenance. Where the channel width must be limited, side slopes may be increased if suitable vegetative or structural stabilization techniques (see following table) and safety measures are utilized. Aesthetics and ease of maintenance should also be considered in the design.

Table 1.4 – Maximum Side Slopes

Stabilization Type	Maximum Side Slopes
Vegetative*	2:1
Stone	1.5:1
Grid Pavers	1.5:1
Paving**	1:1
Retaining Walls	Vertical
Bioengineering	Varies
Other Methods	Approved by Director of Engineering

**Note: Special consideration must be given to the use of vegetative linings in channels. In some cases, structural stabilization is required along the lower portions of the channel bank where continuous or frequent water contact weakens the soil structure and may impede the growth of vegetation (recommend protection to a point 2' above the bottom of the channel or the high water mark for the 2-year storm, which ever is greater). The designer is directed to the State of North Carolina "Erosion and Sediment Control Planning and Design Manual" for the selection of appropriate vegetation based on soil types and flow velocities.*

***Note: Asphalt channel linings are not allowed in the City right-of-way.*

In the interest of preserving existing vegetation (which helps to stabilize stream banks and provides shade thereby reducing temperature extremes) and in order to preserve the aesthetics of natural channels, not all streams have to be altered to protect them from erosion. However, existing channels which are an integral part of the development and which may endanger new or existing structures or other improvements (such as parking lots and tennis courts) as the result of future stream bank erosion, should be evaluated for the need for additional erosion protection. In addition, those existing channels which will be subject to peak flow increases of 100% or more as the result of complete build-out of the contributing watershed and those existing channels with sharp bends should also be evaluated for the need for additional erosion protection.

Vegetation and other bioengineering measures are the preferred method of stabilizing channels. However, calculations must demonstrate that design storm velocities do not exceed those acceptable for the measure.

1.4.6 Culverts

Culverts are structures, such as pipes or boxes, which convey surface water and/or stormwater runoff from one side of a roadway to the other and when the culvert can not handle all of the stormwater runoff, flooding will occur across the roadway.

Culverts shall be designed to meet the following requirements unless the Director of Engineering requires more stringent conditions or waives the minimum requirements identified below.

- **Residential Local –** Designed to convey the 25-year design storm with a 1 foot freeboard and a HW/D<1.2
- **Residential Collector –** Designed to convey the 25-year design storm with a 1 foot freeboard and a HW/D<1.2
- **Commercial Local –** Designed to convey the 50-year design storm with a 0.5 foot freeboard and a HW/D<1.2
- **Commercial Collector –** Designed to convey the 50-year design storm with a 0.5 foot freeboard and a HW/D<1.2
- **Industrial Local –** Designed to convey the 50-year design storm with a 0.5 foot freeboard and a HW/D<1.2
- **Industrial Collector –** Designed to convey the 50-year design storm with a 0.5 foot freeboard and a HW/D<1.2
- **Minor Arterial –** Designed to convey the 50-year design storm with a 0.5 foot freeboard and a HW/D<1.2
- **Major Arterial –** Designed to convey the 50-year design storm with a 0.5 foot freeboard and a HW/D<1.2

Minimum size of culvert shall be 18-inches in diameter.

Headwater elevations for all roads crossing watercourses for the peak flows during the design storm and 100-year storms, including weir calculations, shall be provided for those situations where overtopping is allowed.

Culverts on intermittent and perennial streams shall be designed to maintain the integrity of the stream channel. If the 10-year peak discharge is greater than 50 cfs then the culvert shall be designed to handle the 2-year storm event within a lower section set at the stream invert and the design storm in a larger section set at a higher elevation such that the larger section is utilized when water elevations exceed the water elevations during the 2-year storm event.

Flared end sections are required at the inlet and outlet on all culverts 48 inches or less.

Inlet and outlet headwalls are required on culverts larger than 48 inches in diameter.

1.4.7 Energy Dissipation Design

Energy dissipators shall be employed whenever the velocity of the flow leaving a storm drainage pipe or culvert exceeds the erosive velocity of the receiving channel.

Energy dissipators shall be designed per the North Carolina Erosion and Sediment Control Planning and Design Manual or Hydraulic Engineering Circular No. 14 “Hydraulic Design of Energy Dissipators for Culverts and Channels”.

1.5 Stormwater Quantity Control

Stormwater quantity control, for purposes of this manual, refers to the management of the impact new development has on downstream drainage systems and properties and the management of potential flooding within the development site. In order to manage the downstream impact, new development must control the peak flow rates that leave the site. This control is referred to as stormwater detention. In order to manage the potential flooding within the development, the potential floodplain boundaries must be identified. This is referred to as floodplain management.

1.5.1 Stormwater Detention

New development shall not result in an increase in the peak stormwater runoff leaving the site from the pre-development conditions for the following storm events unless the development is demonstrated to be exempt:

- **1-year, 24 hour storm event** – To reduce downstream channel degradation
- **10-year, 24-hour storm event** – To protect downstream drainage system capacity.
- **25-year, 24-hour storm event** – To protect downstream properties.

A development is exempt from the above control requirements if:

- The overall impervious surface is less than 15 percent of the total site and the pervious portions of the site are used to the maximum extent practical to convey and control the stormwater runoff, or;
- The increase in peak flow between the pre-development and post-development conditions does not exceed 10 percent, or;
- The Director of Engineering makes a determination that peak stormwater control at this particular location will increase flooding,

accelerate erosion or negatively impact existing storm drainage problems in the area. In such cases, an alternate method of stormwater quantity control may be required.

The designer shall demonstrate quantity control requirements are satisfied by routing hydrographs using the acceptable methods presented in Chapter 3 – Calculations.

If an impoundment is used to control the peak flow, it is the responsibility of the designer to verify the whether the impoundment is regulated by the Division of Land Resources under the jurisdiction of the Dam Safety Act NCAC T15A 02K.0100. An impoundment that is 15 feet high or more and has 10-acre-feet of storage or more must comply with the North Carolina Dam Safety Act which has specific spillway and embankment design standards and requires a separate submission to the Division of Land Resources for review and approval. Impoundments below the established threshold can be classified as high hazard by the Division of Land Resources and also be required to satisfy the rules of the Dam Safety Act.

1.5.2 Floodplain Management

Chapter 9 of the City's Land Development Code identifies the allowable activities and procedures for any activities within the flood zones.

In general, drainage systems are not designed to convey all potential storm events or quantity of stormwater runoff nor is it possible to guarantee that the system will never experience a debris blockage or other unpredictable event. For this reason, the designer shall always take into consideration where the water would flow in the event the system capacity is exceeded or the system is blocked or failed.

For storm drainage pipe systems and open channels that convey a peak flow of more than 50-cfs during the 100-year storm event, the 100-year floodplain boundary shall be calculated and shown on the development plans. When the drainage system includes a watercourse, the Subdistrict C flood zone shall be shall be shown as defined in Chapter 9 of the Land Development Code unless calculations demonstrate otherwise.

For storm drainage pipe systems and open channels that convey a peak flow of less than 50-cfs during the 100-year storm event, the design shall consider where water will flow if the system overflows. The Director of Engineering may require calculation of the 100-year floodplain boundary if there is concern that system overflows may cause structural flooding or unsafe roadway or driveway flooding.

All drainage systems and site development shall consider the effects of the FEMA Floodplain as shown on the most recent Flood Insurance Rate maps and floodplains defined by the City of Rocky Mount. The floodway and flood plain boundaries must be shown for any mapped stream.

The designer shall determine floodplain boundaries using acceptable step backwater calculations as presented in Chapter 3 – Calculations unless FEMA or the City has developed an existing floodplain boundary.

1.6 Stormwater Quality Control

These policies implement citywide measurable performance goals for control of nutrients, total nitrogen and total phosphorous in stormwater runoff as required by the Tar-Pamlico Rules, NPDES regulations and others. The control of sediment is also required for construction site runoff as part of the City's Erosion Control Program, and specific restrictions and performance-based criteria for controlling total suspended solids in stormwater runoff exist in the water supply watershed protection area. This section summarizes the City's nutrient and total suspended solids control requirements to satisfy the Tar-Pamlico Nutrient Sensitive Waters and Water Supply Watershed Protection Rules. Guidelines and design requirements for the City's Sediment and Erosion Control Program can be found in Chapter 8, section 802 of the City's Land Development Code.

The City of Rocky Mount encourages satisfying these stormwater quality requirements through onsite land planning measures to reduce the disturbed and impervious areas to the maximum extent practical, as well as the design and installation of as few structural BMPs as practical.

The City also strongly encourages the use of onsite structural BMPs to provide both stormwater quantity and quality control.

Garbage dumpsters and apartment or condominium car washing areas must be located such that runoff from these areas sheet flows across a densely vegetated area. These facilities cannot be located in close proximity to streams or other watercourses.

1.6.1 Tar-Pamlico Nutrient Sensitive Waters

Chapter 8, Section 802 of the City's Land Development Code describes the requirements for the Tar-Pamlico Nutrient Sensitive Waters in detail. This section provides a brief summary of those requirements. Nutrient offset payment can be made in lieu of onsite land planning and/or structural control measures. Any nutrient offset payments must be in accordance with North Carolina Administrative Code 15A NCAC 02B .0240 and subsequent amendments.

1.6.2 Nutrient Control – One Acre Disturbance (Single-Family Residential)

New single-family residential development that disturbs greater than **1** (one) acre of land to establish, expand, or replace a single family residential development or recreational facility shall demonstrate the following nutrient loading requirements are met:

- **Total Nitrogen (TN) Export is reduced to 4.0 pounds per acre per year (lbs/ac/yr).**
  - If the nitrogen reduction can not be obtained through onsite land planning measures and/or structural BMPs then an equivalent mass load reduction shall be obtained through the treatment of existing offsite areas either in the site structural BMP, separate offsite structural BMP or regional structural BMP as long as the Total Nitrogen Export from the site does not exceed **6.0 lbs/ac/yr** and the offsite facility is located within the same classified surface water as defined in Chapter 8 of the City's Land Development Code.

- **Total Phosphorous (TP) Export is reduced to 0.4 lbs/ac/yr.**
  - If the phosphorous reduction can not be obtained through onsite land planning measures and/or structural BMPs then an equivalent mass load reduction shall be obtained through the treatment of existing offsite areas either in the site structural BMP(s), separate offsite structural BMP(s) or regional structural BMP located within the same classified surface water as defined in Chapter 8 of the City's Land Development Code.

Nutrient offset payment for both nitrogen and phosphorous can be made in lieu of onsite land planning and/or structural control measures. Any nutrient offset payments must be in accordance with North Carolina Administrative Code 15A NCAC 02B .0240 and subsequent amendments.

1.6.3 Nutrient Control – One Half Acre Disturbance (All Other Projects)

New development that disturbs greater than **1/2** (one-half) acre of land to establish, expand, or replace a multi-family residential development, commercial, industrial, institutional or any other non-residential development shall demonstrate the following nutrient loading requirements are met:

- **Total Nitrogen Export is reduced to 4.0 Lbs/acre/year.**
  - If the nitrogen reduction can not be obtained through land planning measures and/or structural BMPs then an equivalent mass load reduction shall be obtained through the treatment of existing offsite areas either in the site structural BMP, separate

offsite structural BMP or regional structural BMP as long as the Total Nitrogen Export from the site does not exceed **10.0 lbs/ac/yr** and the offsite facility is located within the same classified surface water as defined in Chapter 8 of the City's Land Development Code.

- **Total Phosphorous (TP) Export is reduced to 0.4 lbs/ac/yr.**
  - If the phosphorous reduction can not be obtained through onsite land planning measures and/or structural BMPs then an equivalent mass load reduction shall be obtained through the treatment of existing offsite areas either in the site structural BMP(s), separate offsite structural BMP(s) or regional structural BMP located within the same classified surface water as defined in Chapter 8 of the City's Land Development Code.

Nutrient offset payment can be made in lieu of onsite land planning and/or structural control measures. Any nutrient offset payments must be in accordance with North Carolina Administrative Code 15A NCAC 02B .0240 and subsequent amendments.

The City strongly encourages onsite structural BMPs be designed for existing offsite development in order to achieve offsite nutrient load reductions.

When offsite load reduction is provided in a separate structural BMP, the structural BMP shall be reviewed and approved by the City of Rocky Mount and all of the necessary offsite easements and maintenance agreements shall be in place prior to receiving approval to begin land development activities.

Offsite facilities shall meet the conditions outlined in Chapter 8 of the City's Land Development Code.

Nutrient Loading Calculations are presented in Chapter 3 – Calculations. These calculations have been incorporated into a spreadsheet that must be completed with each development.

1.6.4 Riparian Buffers

The Tar-Pamlico Nutrient Sensitive Waters Rule established a 50 foot wide riparian buffer on all sides of intermittent and perennial streams, ponds and lakes shown on the most recent version of either the Natural Resources Conservation Service Soil Survey or a 1:24,000 scale (7.5 minute quadrangle) topographic map prepared by the U.S. Geological Survey (USGS) as appropriate. The buffer is measured from the top of bank or normal pool of an impoundment. The City will not approve new development plans that include land area within the riparian buffer unless the development receives approval from DWQ.

1.6.5 Water Supply Watershed Protection

Chapter 8, Section 803 of the City's Land Development Code describes the City's requirements for development within the designated Watershed Protection Areas. This section briefly summarizes those requirements.

New development that disturbs more than 1 acre of land located within the WS-IV-CA or WS-IV-PA as shown on the City 's watershed map, which is available on the City's website, shall meet the following conditions in addition to satisfying the stormwater quantity and quality control identified in earlier sections:

- WS-IV-CA (Critical Area)
    - Maintain the following low density and built upon limits
        - Single family residential developments shall not exceed 2 dwelling units per acre.
        - All other residential and non residential shall not exceed 24 percent built upon area.
        - Maintain a minimum of a 30 foot wide vegetative buffer along perennial streams.
    - Limit the percent imperviousness to 50 percent and construct structural BMPs that control the runoff from the first 1-inch of rainfall runoff such that 85% TSS is achieved, and maintain a 100 foot wide vegetative buffer along perennial waters. The buffer is measured from the top of bank or normal pool of impoundment.

- WS-IV-PA (Protected Area)
    - Maintain the following low density and built upon limits
        - Single family residential developments <u>with</u> a curb and gutter street system shall not exceed 2 dwelling units per acre.
        - Single-family residential developments <u>without</u> a curb and gutter street system shall not exceed 3 dwelling units per acre.
        - All other residential and non residential <u>with</u> a curb and gutter street system shall not exceed 24 percent built upon area.
        - All other residential and non residential <u>without</u> a curb and gutter street system shall not exceed 36 percent built upon area.
        - Maintain a minimum of a 30 feet wide vegetative buffer on perennial streams.
    - Limit the percent imperviousness to 70 percent and construct structural BMPs that control the runoff from the first 1-inch of rainfall runoff such that 85% TSS is achieved, and maintain a 100 foot wide vegetative buffer along perennial waters. The buffer is measured from the top of bank or normal pool of impoundment.

City of Rocky Mount, North Carolina
Stormwater Design Manual

Chapter 2: Structural Best Management Practices

December 2006

Rocky Mount Engineering Department
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Rocky Mount, NC 27802

Chapter 2: Structural Best Management Practices

2.1 Introduction

Structural best management practices (BMPs) are constructed stormwater management facilities designed to treat stormwater runoff and/or mitigate the effects of increased stormwater runoff peak rate, volume, and velocity due to urbanization.

The structural BMPs discussed in this section have been approved by both the City of Rocky Mount and the State and can be utilized for attenuating peak flows and reducing pollutants in stormwater runoff from new developments. Structural BMPs shall be designed by North Carolina registered professionals with experience and knowledge of the hydrologic and hydraulic methodologies presented in this Stormwater Management Design Manual. Registered professionals are defined as professional engineers; landscape architects to the extent that G.S 89A allows; and land surveyors the extent that the design represents incidental drainage within a subdivision as provided in G.S. 89C-3(7).

This chapter presents general guidelines for selecting the appropriate structural BMP, general maintenance requirements, water quality volume calculations, minimum required design standards and in some cases design calculations, recommended design standards to enhance the BMP and minimum operation and maintenance for the acceptable structural BMPs.

2.2 Selecting the Appropriate BMPs

Structural BMPs are required to be designed and constructed on new developments to control stormwater runoff in both quantity and quality. The control requirements for the City of Rocky Mount are summarized in Chapter 1 of the Stormwater Management Design Manual. The City of Rocky Mount prefers that structural BMPs be located such that the maximum amount of site-developed area drains to the BMP to minimize the number of structural BMPs that require maintenance and inspection. The City also prefers that structural BMPs that naturally receive offsite drainage areas be designed to handle the existing offsite area in lieu of diverting around the structural BMP.

The following structural BMPs and associated pollutant removal efficiencies have been approved by both the State and the City of Rocky Mount. These pollutant removal efficiencies are assuming the minimum design standards are followed in the design and construction of the structural BMP. The City encourages incorporating the recommended design standards to enhance the overall BMP, however, additional pollutant removal will not be acknowledged.

Table 2.1 - Acceptable Structural BMPs

BMP Type	TN Removal	TP Removal	TSS Removal	Peak Flow Control
Wet Detention Ponds	25%	40%	85%	High
Stormwater Wetlands	40%	35%	85%	High
Wet Pond/Wetland Series	55%	61%	98%	High
Restored Riparian Buffers	30%	30%	40%	Low
Grass Swales	20%	20%	35%	Low
Water Quality Swale	30%	30%	35%	Low
Vegetated Filter Strip with Level Spreader	20%	35%	40%	Low
Bioretention (rain gardens)	40%	35%	85%	Low
Sand Filters	35%	45%	85%	Low
Permeable Pavement	Varies	Varies	Varies	Varies
Proprietary BMPs	Varies	Varies	Varies	Low
Other BMPs	Varies	Varies	Varies	Varies

If more than one BMP is installed in series, then the removal rate shall be determined through serial rather than additive calculations. *For example*, if a wet detention pond discharges through a riparian buffer, then the removal rate shall be estimated to be 47.5%:

1. The pond removes 25 percent of the nitrogen and discharges 75 percent to the buffer.
2. The buffer then removes 30 percent of the nitrogen that discharged from the pond, which is 22.5 percent.
3. The sum of 25 and 22.5 is 47.5. *(NOT 25 plus 30, or 55 percent)*

Alternative or innovative BMPs, such as manufactured BMPs, will require prior approval from the Director of Engineering. The designer shall provide complete documentation on the BMP, including specifications, performance, and maintenance requirements to be submitted. The Director of Engineering will determine if the measure warrants consideration and what, if any, monitoring controls and performance bonding will be required.

2.3 BMP Operation and Maintenance Requirements

The City of Rocky Mount requires the owning entity to operate and maintain the structural BMP so the intended function such as pollutant removal and/or peak flow control does not diminish over time. Chapter 8 Section 804 of the City's Land Development Code presents the general requirements for the posting of a performance bond, establishment of a maintenance agreement, recordation of easements and general operation, maintenance and repair requirements. This chapter establishes the minimum required maintenance of each of the structural BMPs. These minimum maintenance requirements shall be included in the Operation and Maintenance Plan.

The City of Rocky Mount will perform annual inspections of the structural BMPs to verify the operation and maintenance is being performed as identified in the operation and maintenance plan. Chapter 8 of the City's Land Development Code identifies the measures the City will undertake to ensure structural BMPs are being properly maintained.

2.4 Water Quality Volume and Peak Flow

The structural BMPs presented in this Chapter, except for the typical grass swale, all control and treat the water quality volume (WQv). The WQv, sometimes referred to as the first flush runoff, is the runoff from the first inch of precipitation, which is generally the portion of the runoff with the highest concentrations of most conventional nonpoint source runoff contaminants. The structural BMPs treat the WQv in various ways to achieve pollutant load reduction. Some BMPs hold the WQv volume for a period of time to allow pollutants to settle, some hold the WQv in a permanent surface to allow the water to interact with vegetation, some allow the WQv to filter through a media or sheet across filter strip. In all cases, the WQv is the basis for the structural BMP water quality component and in some cases will drive the overall size of the structural BMP.

The WQv is directly related to the amount of impervious cover created at a site. In numerical terms, it is equivalent to an inch of rainfall multiplied by the volumetric runoff coefficient (Rv) and site area.

The following equation is used to determine the storage volume, WQv (in acre-feet of storage):

$$WQv = \frac{(1.0 - inch)(Rv)(A)}{12}$$

Where:

$WQv =$ water quality volume (in acre-feet)

$Rv =$ 0.05 + 0.009(I) where I is percent impervious cover

$A =$ area in acres draining to the structural BMP

Some structural BMPs, such as bioretention areas, sand filters and level spreaders function best when large storms are bypassed. These large storms tend to damage these types of structural BMPs and it is impractical to properly size these facilities to handle them. When the WQv is diverted to a structural BMP, a flow splitter is required to handle the WQv peak flow. The WQv peak flow (Qwq) is based on the NRCS Unit Hydrograph Method presented in Chapter 3. The RCN is calculated based on the runoff of the WQv event by using the equation below:

$$RCN = \frac{1000}{10 + (5)(P) + (10)(Q) - (10)\left(Q^2 + 1.25(Q)(P)\right)^{1/2}}$$

Where:

$RCN =$ Runoff Curve Number
$P =$ The 1 – inch rainfall event
$Q =$ WQv runoff (inches) = $(1.0)(.05 + .009(I))$
Where:

$I =$ the drainage area % imperviousness

The calculated RCN is then used with the graphical method presented in Chapter 3 to calculate a peak discharge. For this calculation, if the Ia/P calculation is less than 0.1, then 0.1 should be used. It is important to note that this is an approximation of the peak flow and it is assumed that if the diversion can handle the WQv peak flow then all the WQv will be diverted to the structural BMP.

When the drainage area is highly impervious (greater than 95%) then the Qwq can be calculated using:

$$Qwq = 1.48(A)$$

Where:

$Qwq =$ water quality peak flow in cfs
$A =$ drainage area in acres

2.5 Wet Detention Ponds

Wet detention ponds, as shown in Figure 2.1, are ponds that are sized and configured to provide significant removal of pollutants from incoming stormwater runoff and can provide peak flow control. They maintain a permanent pool of water that is designed for a target total suspended solids (TSS) removal rate according to the size and imperviousness of the contributing watershed. Above this permanent pool of water, they are also designed to hold the WQv and release this over a period of two to five days. These two basic requirements result in a pond where a majority of the suspended sediment and pollutants attached to the sediment are allowed to settle out of the water. The wet detention pond outlet device can be designed to provide peak control of larger storm events through a combination of orifices, weirs and pipe/barrel sizing. Chapter 3 presents the basic equations for typical outlet devices.

Minimum Design Standards

- Minimum drainage area of 10 acres unless otherwise approved by the Director of Engineering.
- Normal pool volume based on Table 2.2. This table was developed based on Driscoll's model (US EPA, 1986) based on the long-term average storm retention time.
- Normal pool minimum depth of 3 feet.
- Maximum normal pool depth of 8 feet.
- The normal pool shall have a combination aquatic vegetation and safety bench that extends at least 10 feet into the normal pool at a maximum slope of 6%.
- Minimum length to width ratio of normal pool 1.5:1 (preferably expanding outward toward the outlet).
- A sediment forebay shall be located at the upstream side of the normal pool and should consist of a separate cell, formed by an acceptable barrier protected from erosion.
- The volume of the sediment forebay shall be 0.1-inch times the impervious acreage draining to the forebay. The surface area of the forebay shall be considered part of the pond surface area.
- A permanent benchmark shall be installed in the vicinity of the sediment forebay to allow for ready determination of the sediment depth.
- The WQv shall be stored above the normal pool and slowly released over at least 48 hours.
- The top of the embankment shall be a minimum 6-inches above the 100-year, 24-hour elevation with 1-foot recommended, and a minimum of 1.0 foot above the 25-year, 24-hour elevation.
- Earthen embankment side slopes should be no greater than 3:1 and shall have a well-established grass cover.
- A separate emergency spillway shall be provided to convey events greater than the 25-year storm. In special circumstances the Director of Engineering may allow all storm events to be conveyed through the principal spillway.

- The principal spillway shall be designed to control the design events. If the principal spillway is a riser/barrel type, then the design shall be such that the barrel controls during the storm event prior to the use of the emergency spillway.
- The principal spillway shall have trash racks, hoods, or other debris control devices as necessary to prevent clogging.
- Anti-vortex measures shall be incorporated into the trash racks to prevent a vortex from forming during large storm events.
- The principal spillway shall be checked for buoyancy and appropriate anti-floatation measures incorporated to maintain a minimum factor of safety of 1.5.
- Riprap protection or other stilling basin type structure per HEC-14 shall be provided for the principal spillway outlet and all inlet structures into the pond.
- When the principal spillway is a riser/barrel structure, anti-seep collars or filter diaphragms shall be provided to reduce failure due to piping.
- Pipes through an embankment shall be watertight.
- The normal pool shall be able to be drawn down to the elevation of the outlet invert within 24 hours through the opening of some type of emergency drain (i.e. sluice gate, drawdown pipe).
- Design the emergency spillway to pass the 100-year storm event.
- Provide for vehicle maintenance access, a minimum of 15 feet wide to the embankment and sediment forebay along with provisions for equipment to maneuver.
- Vehicle maintenance access paths shall be at slopes no greater than 10 percent.
- If the normal pool area is used as temporary sediment basin during construction, all sediment shall be removed and properly disposed of prior to final inspection.

Table 2.2 - Normal Pool SA/DA Ratios in Percent

DA Impervious	PERMANENT POOL AVERAGE DEPTHS IN FEET					
PERCENT	3.0	4.0	5.0	6.0	7.0	8.0
10%	0.59	0.49	0.43	0.35	0.31	0.29
20%	0.97	0.79	0.70	0.59	0.51	0.46
30%	1.34	1.08	0.97	0.83	0.70	0.64
40%	1.73	1.43	1.25	1.05	0.90	0.82
50%	2.06	1.73	1.50	1.30	1.09	1.00
60%	2.40	2.03	1.71	1.51	1.29	1.18
70%	2.88	2.40	2.07	1.79	1.54	1.35
80%	3.36	2.78	2.38	2.10	1.86	1.60
90%	3.74	3.10	2.66	2.34	2.11	1.83

Recommended Standards

- Inlet and outlet located to maximize flow length. Use baffles if short-circuiting cannot be prevented with inlet-outlet placement. Long flow paths and irregular shapes are recommended.
- Design the pond for multi-objective use, such as amenities or flood control.
- Landscaping management of buffer as meadow.
- Provide a length to width ratio of 3:1 to 4:1 (preferably wedge shaped).
- Use reinforced concrete instead of corrugated metal for pipes.
- For minor pond inlets, level spreaders through a vegetated area should be installed to reduce the sediment loading.
- Consider artificial mixing for small sheltered ponds through the installation of fountains or mixers.
- Impervious soil boundary to prevent drawdown may be needed.
- Shallow marsh area around fringe 25 to 50 percent of area (including aquatic vegetation) should be established.
- When the normal pool depth is greater than 4 feet, the safety bench should be increased to 15 feet.
- Include an aquatic bench with the safety bench that extends inward from the normal shoreline and has a maximum depth of eighteen inches below the normal pool water surface elevation.
- Minimum 25 foot wide buffer around pool.
- Provide on-site disposal areas for two sediment removal cycles. These disposal areas should be protected from runoff.
- Provide an oil and grease skimmer on the principal spillway in areas with a 50% or more impervious roadway and parking.
- Harden the bottom of the forebay (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.

Unique Calculations

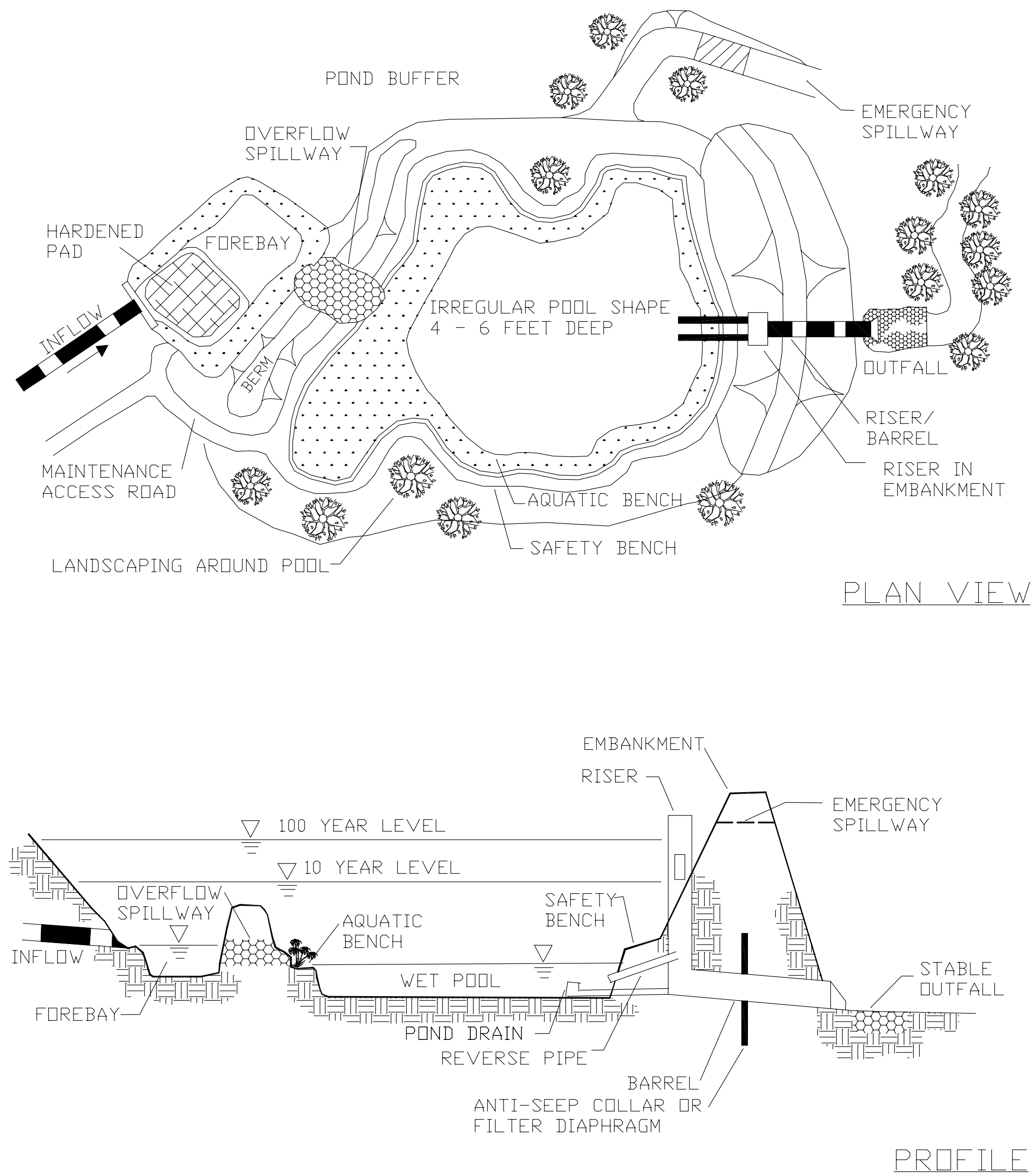
- No unique calculations required.

Operation and Maintenance Recommendations

- Maintenance should always include minimizing erosion problems and pollutant export to the pond from the contributing watershed. Care should be taken to secure all appropriate legal agreements for the easement.
- The sediment forebay should be cleaned out when the sediment volume is 50% of the original volume.
- Sediment within the remaining normal pool should be removed when 20% of the original normal pool volume is filled.
- No woody vegetation should be allowed on the embankment without special designs.
- Vegetation over 18 inches high should be cut unless it is part of planned landscaping.
- Debris should be removed from blocked inlet and outlet structures and from areas of potential clogging.

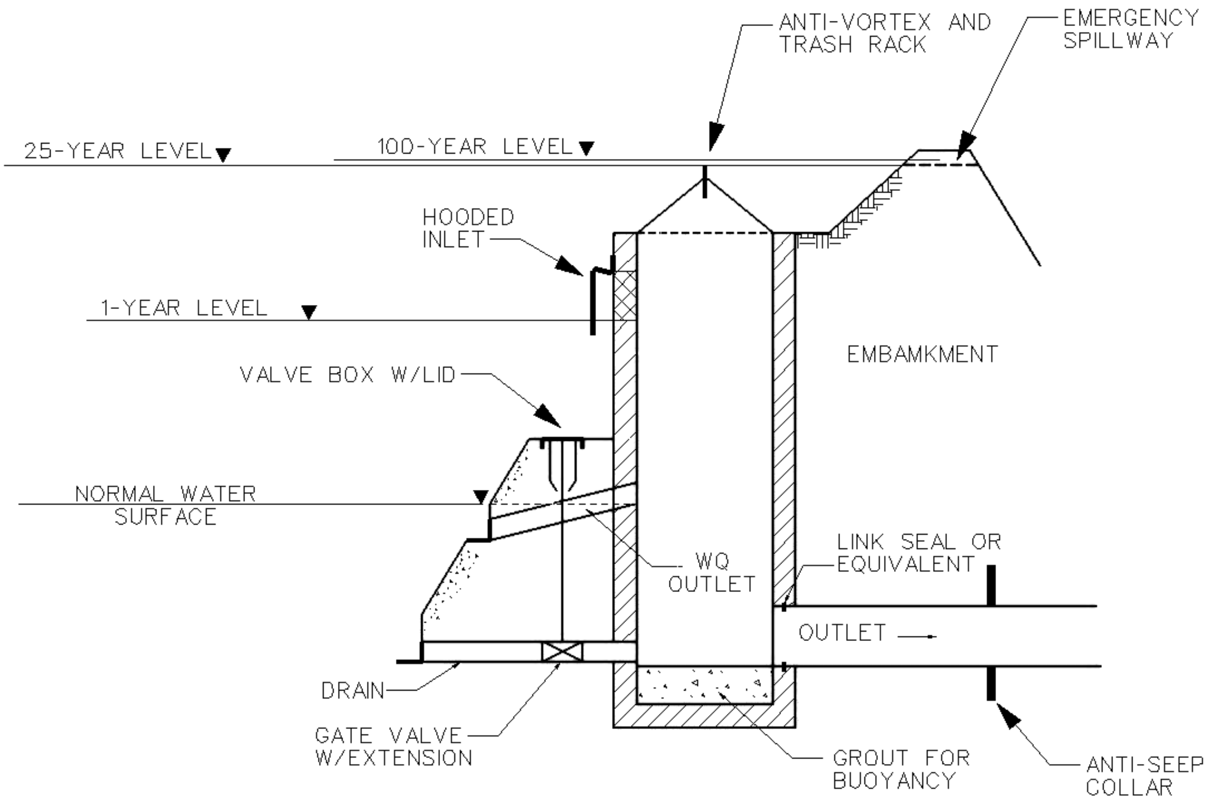
- The outlet control should be kept structurally sound, free from erosion, and functioning as designed.
- Periodic removal of dead vegetation should be accomplished.
- Inspection requirements should be outlined in the operation and maintenance manual. The manual shall identify special maintenance needs and include routine inspections including but not limited to:
  - Debris removed after every major storm
  - Routine mowing schedule
  - Sediment buildup and the need for removal
  - Erosion along the bank and the need for reseeding or stabilization
  - If reseeding is necessary, a reseeding schedule
  - Erosion at the inlet and outlet and methods of stabilization
  - Seepage through the dam
  - Emergency spillway inspection
  - Operation of any valves or mechanical components
  - Consider chemical treatment by alum if algal blooms are a problem.

Figure 2.1 - Wet Detention Pond



Source: Controlling Urban Runoff

Figure 2.2 - Spillway Configuration



2.6 Stormwater Wetlands

Stormwater wetlands, as shown in Figure 2.3, can be defined as pocket wetlands which rely on groundwater to maintain adequate water supply and shallow marsh wetlands, as shown in Figure 2.4, which have sufficient drainage area to maintain an adequate water supply through stormwater runoff. For the purposes of this manual, the pocket wetland and shallow marsh wetland are both considered stormwater wetlands and provide the same pollutant removal efficiencies. Stormwater wetlands consist of micropools, deep water and areas with shallow flooding to support various types of wetland vegetation. The micropool, deep water and shallow flooding are sized for the surface area requirement of a 3 foot deep wet pond with varying zones that promote stormwater wetland vegetation. Above this permanent pool of water, the stormwater wetland is also designed to hold the WQv and release this over a period of two to five days. These two basic requirements result in a stormwater wetland where a majority of the suspended sediment and pollutants attached to the sediment are allowed to settle out of the water and different aerobic and anaerobic zones are created that enhance nutrient removal. The stormwater wetland outlet device can be designed to provide peak control of larger storm events through a combination of orifices, weirs and pipe/barrel sizing. Chapter 3 presents the basic equations for typical outlet devices.

Minimum Design Standards

- For drainage areas up to 10 acres the micropool and shallow flooding area must be excavated into the normal groundwater or a spring shall be present to maintain the permanent water surface elevation.
- The Director of Engineering may require the micropool and shallow flooding area be excavated into the normal groundwater for drainage areas larger than 10 acres based on local knowledge unless it can be demonstrated through water balance calculations that adequate water will be provided by stormwater runoff.
- The permanent pool surface area shall be sized using Table 2.2 with an average depth of 3.0 feet.
- .A minimum of 70 percent of the permanent pool surface area shall be designed as a marsh. The marsh area shall have an almost equal distribution of low and high marsh areas. The low marsh areas shall be between 6 and 12-inches deep and the high marsh area shall be between 0 and 6-inches deep.
- A soil depth of at least 4 inches should be used for shallow wetland basins
- A micropool that is between 3 and 6 feet deep shall be located near the outlet structure. The micropool surface area shall be at least 15 percent of the permanent pool surface area.
- The remaining surface area shall be distributed between a deepwater area, with an average depth of 3 feet and the sediment forebay(s).

- The length to width ratio should be at least 2 to 1
- The WQv shall be stored above the permanent pool surface area and slowly released over a period between 2-5 days.
- A sediment forebay shall be located at the upstream side of the wetland and at separate inflow points that contribute more than 10% of WQv and should consist of a separate cell, formed by an acceptable barrier protected from erosion.
- The volume of the sediment forebay shall be 0.1-inch times the impervious acreage draining to the forebay. The surface area of the forebay below the permanent pool shall be considered part of the stormwater wetland permanent pool surface area.
- A permanent benchmark shall be installed in the vicinity of the sediment forebay to allow for ready determination of the sediment depth.
- The top of the embankment shall be a minimum 6-inches above the 100-year 24-hour elevation with 1.0 foot recommended and a minimum of 1.0 foot above the 25-year 24-hour elevation.
- Earthen embankment side slopes should be no greater than 3:1 and shall have a well-established grass cover.
- A separate emergency spillway shall be provided to convey events greater than the 25-year storm. In special circumstances the Director of Engineering may allow all storm events to be conveyed through the principal spillway.
- The principal spillway shall be designed to control the design events. If the principal spillway is a riser/barrel type, then the design shall be such that the barrel controls during the storm event prior to the use of the emergency spillway.
- The principal spillway shall have trash racks, hoods, or other debris control devices as necessary to prevent clogging.
- Anti-vortex measures shall be incorporated into the trash racks to prevent a vortex from forming during large storm events.
- The principal spillway shall be checked for buoyancy and appropriate anti-floatation measures incorporated to maintain a minimum factor of safety of 1.5.
- Riprap protection or other stilling basin type structure per HEC-14 shall be provided for the principal spillway outlet and all inlet structures into the pond.
- When the principal spillway is a riser/barrel structure, anti-seep collars or filter diaphragms shall be provided to reduce failure due to piping.
- Pipes through an embankment shall be watertight.
- The permanent pool shall be able to be drawn down to the elevation of the outlet invert within 24 hours through opening some type of emergency drain (i.e. sluice gate, drawdown pipe, etc).
- Design the emergency spillway to pass the 100-year storm event.
- The deepwater area of the wetland should include the outlet structure so outflow from the basin is not interfered with by sediment buildup.
- Stabilize surrounding slopes with vegetation to trap sediments and other pollutants, preventing them from entering the wetland.

- A maintenance plan should be provided and adequate provision made for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
- The wetland should be maintained to prevent loss of water available for emergent vegetation due to sedimentation and/or accumulation of plant material.
- Provide for vehicle maintenance access, a minimum of 15 feet wide to the embankment and sediment forebay along with provisions for equipment to maneuver.
- Vehicle maintenance access paths shall be at slopes no greater than 10 percent.
- If a minimum coverage of 50% is not achieved in the planted wetland zones after the second growing season, a reinforcement planting will be required.

Recommended Specifications

- The designer should maximize use of existing- and post-grading pondscaping design to create both horizontal and vertical diversity and habitat.
- It is recommended that the frequently flooded zone surrounding the wetland be located within approximately 10 to 20 feet from the edge of the permanent pool.
- Soil types conducive to wetland vegetation should be used during construction.
- The wetland should be designed to allow slow percolation of the runoff through the substrate (add a layer of clay for porous substrates).
- As much vegetation as possible and as much distance as possible should separate the basin inlet from the outlet.
- The water should gradually get shallower about 10 feet from the edge of the pond.
- The planted areas should be made as square as possible within the overall design of the wetland, rather than long and narrow.
- The only site preparation that is necessary for the actual planting (besides flooding the basin) is to ensure that the substrate is soft enough to permit relatively easy insertion of the plants.
- The most common and reliable technique for establishing an emergent wetland community in a stormwater wetland is to transplant nursery stock obtained from local aquatic plant nurseries. The transplanting window extends from early April to mid-June. Planting after these dates is not recommended, as the wetland plants need a full growing season to build the root reserves needed to get through the winter. If at all possible, the plants should be ordered at least three months in advance to ensure the availability of the desired species.
- The optimal depth requirements for several common species of emergent wetland plants are often six inches of water or less. To add diversity to the wetland, 5 to 7 species of emergent wetland plants should be used. Of these, at least three species should be selected from "aggressive colonizers" plants such as bulrush,

pickerelweed, arrow arum, three square and rice cutgrass (MDE, 1986).

- Plants should be installed in clumps with individual plants located an average of 18 inches on center within each clump. Individual plants should be spaced 12 inches to 24 inches on center.
- Wetland mulch, if used, should be spread over the high marsh area and adjacent wet zones (-6 to +6 inches of depth) to depths of 3 to 6 inches.

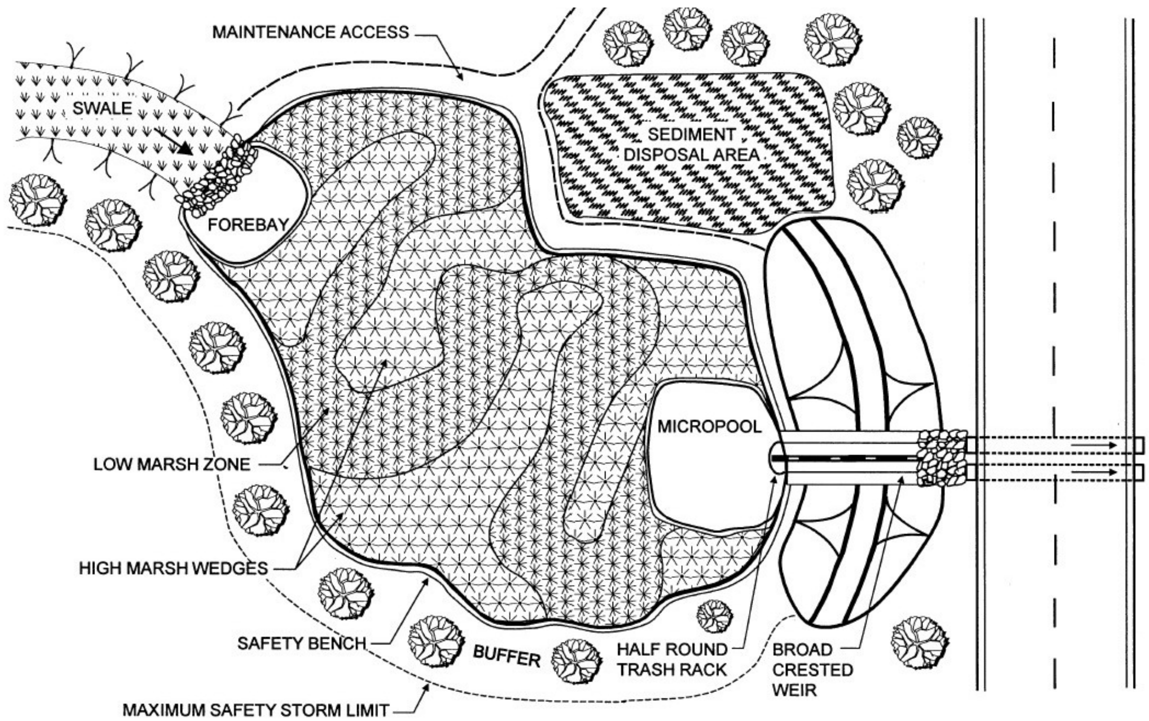
Unique Calculations

- No unique calculations required.

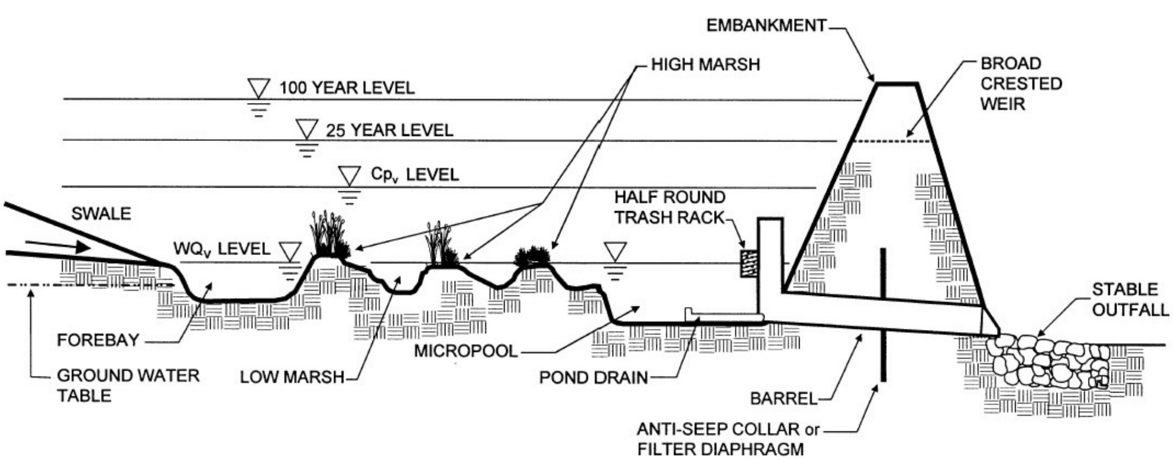
Operation and Maintenance Recommendations

- A stormwater maintenance manual is required for each facility. The maintenance manual should require the owner of the wetland to periodically clean the structure. The manual should provide for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
- The wetland should be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/or accumulation of plant material.
- Sediment forebays should be cleaned every 2 to 5 years, except for pocket wetlands without forebays which are cleaned after a six-inch accumulation of sediment.
- The ponded water area may be maintained by raising the elevation of the water level in the permanent pond, by raising the height of the orifice in the outlet structure, or by removing accumulated solids by excavation.
- Water levels may need to be supplemented or drained periodically until vegetation is fully established.
- It may be desirable to remove contaminated sediment deposits or to harvest above ground biomass and remove it from the site to permanently remove pollutants from the wetland.
- Performance enhancement can be obtained by increasing the size of the marsh area, by incorporating multiple pools into marsh area, or by incorporating a network of shallow channels in the marshy area.
- Remove woody vegetation/trees in excess of 2-inches in diameter.

Figure 2.3 - Pocket Wetland

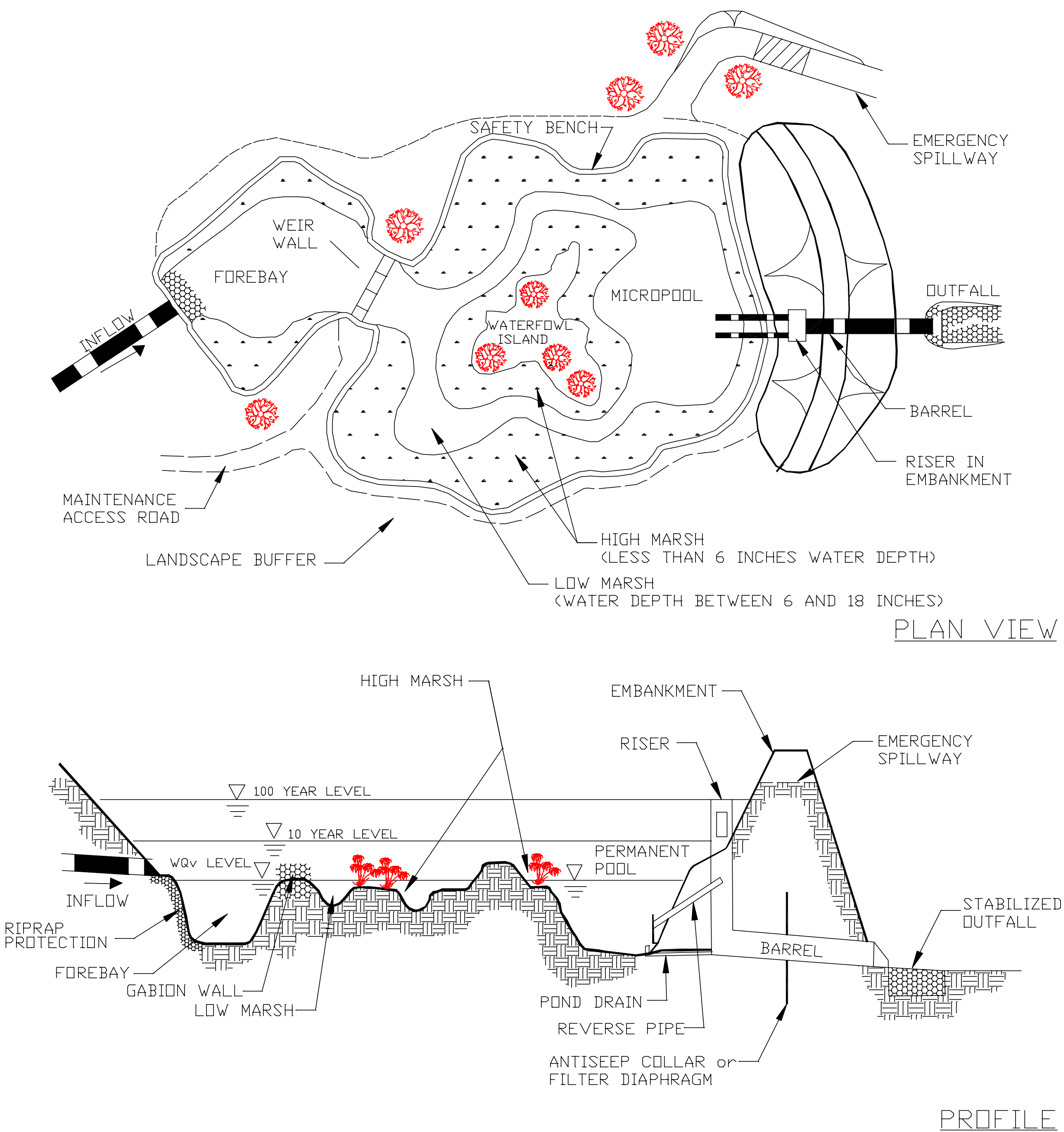


PLAN VIEW



PROFILE

Figure 2.4 - Shallow Marsh Wetland



2.7 Wet Pond/Wetland Series

The City of Rocky Mount prefers that nutrient reduction be obtained using onsite controls in series. However the City does not want multiple embankments and unnecessary sediment forebays, therefore the features of the wet detention pond and the stormwater wetlands discussed above have been combined into a single device as shown in Figure 2.5. The basis for this device is to size the wet pond using the same criteria identified in the wet detention basin and size the stormwater wetland micropool and shallow flooding using the same criteria as identified in the stormwater wetland but design a single sediment forebay, single embankment and single outlet control structure to minimize maintenance and potential areas of failure. The pond/wetland series outlet device can be designed to provide peak control of larger storm events through a combination of orifices, weirs and pipe/barrel sizing. Chapter 3 presents the basic equations for typical outlet devices.

Minimum Design Standards

- Minimum drainage area of 10 acres unless otherwise approved by the Director of Engineering.
- Normal pool volume of the pond based on Table 2.2. This table was developed based on Driscoll's model (US EPA, 1986) model based on the long-term average storm retention time.
- Normal pool minimum depth of 3 feet.
- Maximum normal pool depth of 8 feet.
- The normal pool shall have a combination aquatic vegetation and safety bench that extends at least 10 feet into the normal pool at a maximum slope of 6%.
- Minimum length to width ratio of normal pool of 1.5:1 (preferably expanding outward toward the outlet).
- A sediment forebay shall be located at the upstream side of the normal pool and should consist of a separate cell, formed by an acceptable barrier protected from erosion.
- The volume of the sediment forebay shall be 0.1-inch times the impervious acreage draining to the forebay. The surface area of the forebay shall be considered part of the pond surface area.
- A permanent benchmark shall be installed in the vicinity of the sediment forebay to allow for ready determination of the sediment depth.
- The pond shall flow into a stormwater wetland with a permanent pool surface area sized using Table 2.2 with an average depth of 3.0 feet.
- .A minimum of 70 percent of the wetland permanent pool surface area shall be designed as a marsh. The marsh area shall have an almost equal distribution of low and high marsh areas. The low marsh areas shall be between 6 and 12-inches deep and the high marsh area shall be between 0 and 6-inches deep.
- A soil depth of at least 4 inches should be used for shallow wetland basins

- A micropool that is between 3 and 6 feet deep shall be located near the outlet structure. The micropool surface area shall be at least 15 percent of the permanent pool surface area.
- The remaining permanent pool surface area shall be distributed in a deepwater area with an average depth of 3 feet.
- 2 times the WQv shall be stored above the normal pool of the wet pond and stormwater wetland and slowly released over at least 48 hours.
- The top of the embankment shall be a minimum 6-inches above the 100-year 24-hour elevation with 1.0 foot recommended and a minimum of 1.0 foot above the 25-year 24-hour elevation.
- Earthen embankment side slopes should be no greater than 3:1 and shall have a well established grass cover.
- A separate emergency spillway shall be provided to convey events greater than the 25-year storm. In special circumstances the Director of Engineering may allow all storm events to be conveyed through the principal spillway.
- The principal spillway shall be designed to control the design events. If the principal spillway is a riser/barrel type, then the design shall be such that the barrel controls during the storm event prior to the use of the emergency spillway.
- The principal spillway shall have trash racks, hoods, or other debris control devices as necessary to prevent clogging.
- Anti-vortex measures shall be incorporated into the trash racks to prevent a vortex from forming during large storm events.
- The principal spillway shall be checked for buoyancy and appropriate anti-floatation measures incorporated to maintain a minimum factor of safety of 1.5.
- Riprap protection or other stilling basin type structure per HEC-14 shall be provided for the principal spillway outlet and all inlet structures into the pond.
- When the principal spillway is a riser/barrel structure, anti-seep collars or filter diaphragms shall be provided to reduce failure due to piping.
- Pipes through an embankment shall be watertight.
- The normal pool shall be able to be drawn down to the elevation of the outlet invert within 24 hours through the opening some type of emergency drain (i.e. sluice gate, drawdown pipe).
- Design the emergency spillway to pass the 100-year storm event.
- Provide for vehicle maintenance access, a minimum of 15 feet wide, to the embankment and sediment forebay along with provisions for equipment to maneuver.
- Vehicle maintenance access paths shall be at slopes no greater than 10 percent.
- If the normal pool area is used as temporary sediment basin during construction, all sediment shall be removed and properly disposed of prior to final inspection.

Recommended Standards

- The same recommended standards for the wet detention pond and stormwater wetland apply to the pond/wetland series.

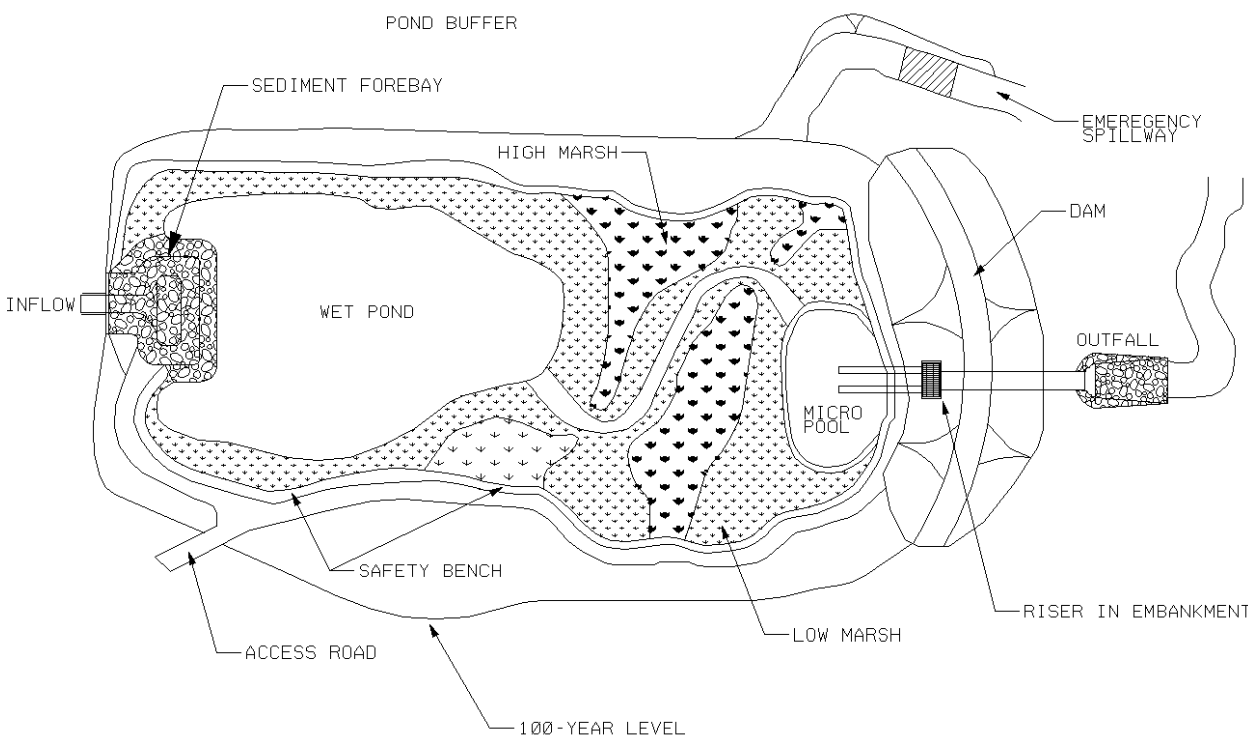
Unique Calculations

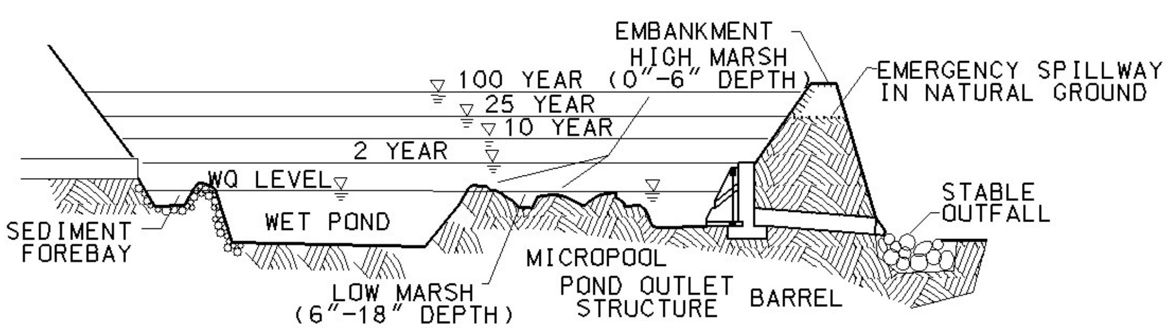
- No unique calculations required.

Operation and Maintenance Recommendations

The operation and maintenance recommendations for the wet detention pond and stormwater wetland also apply to the pond/wetland series.

Figure 2.5 - Wet Pond/Wetland Series





2.8 Restored Riparian Buffers

The Tar-Pamlico Rules require maintenance of existing riparian buffers and establishes how to restore a riparian buffer. This section describes how to design the riparian buffer for nutrient reduction. Details for the riparian buffers can be found in the Tar-Pamlico Riparian Buffer Rule 15A NCAC 2B. 259.

Minimum Design Standards

- The use of buffers should be limited to drainage areas of 10 acres or less with the optimal size being less than 5 acres.
- Riparian buffers must meet the Tar-Pamlico Buffer rules.
- Runoff entering the buffer must be sheet flow.
- If the runoff does not enter as sheet flow then a level spreader that meets the most recent DWQ guidelines.

Recommended Standards

- The most damage to the level spreaders and riparian buffers occurs during the infrequent large storm events. The City of Rocky Mount prefers the WQv be diverted to the level spreader, and the larger storms conveyed to the intermittent and perennial stream in an adequately designed drainage system to prevent erosion during the larger storm events.
- Runoff water containing high sediment loads should be treated in a sediment trapping device before being released to a flow spreader.
- Buffers should not be cleaned of leaf litter or "managed," particularly Zone 1 of a regulated riparian buffer.

Unique Calculations

- The flow diversion calculations that demonstrate the WQv peak flow is controlled through the diversion is required.

Operation and Maintenance Recommendations

- The buffer should be inspected for signs of erosion or concentrated flow after significant rainfall events and needed repairs made promptly to maintain sheet flow through the riparian buffer.

2.9 Grassed Swales

Grassed swales are shallow trapezoidal or parabolic earthen channels covered with a dense growth of a hardy grass such as Reed Canary or Tall Fescue. Grassed swales are sometimes classed as a type of biofilter because the vegetation on the swale takes up some pollutants and helps filter sediment and other solid particles out of the runoff. These channels convey stormwater and provide some stormwater management for small storms by retarding peak flow rates, lowering velocities of runoff and by infiltrating runoff water into the soil. Swales are used primarily in single-family residential developments, at the outlets of road culverts, and as highway medians.

Minimum Design Standards

- Longitudinal slope should be in the range of 2 to 4%. If the slope along the flow path exceeds 4%, then checkdams must be installed to reduce the effective slope to below 4%.
- Side slopes should be no greater than 3: 1 horizontal to vertical.
- Maximum runoff velocity should be 2 fps for the peak runoff of the 2-year storm.
- The design must also non-erosively pass the peak runoff rate from the 10-year storm.
- The length of swale shall be at least 100 feet per acre of drainage area.
- A vegetation plan shall be prepared in accordance with the recommendations found in the Erosion and Sediment Control Planning and Design Manual.
- Swales should be stabilized within 14 days of swale construction.

Recommended Standards

- Swales should be constructed on permeable, noncompacted soils.
- Swales should be sited in areas where the seasonal high water table is at least one foot below the bottom of the swale.
- Swales should not carry dry-weather flows or constant flows of water; and
- Swales should have short contact times or short grass.

Unique Calculations

- No unique calculations required.

Operation and Maintenance Recommendations

- Remove excess sediment at least once annually, especially from the upstream edge to maintain original contours and grading.
- At least once annually, repair any erosion and regrade the swale to ensure that the runoff flows evenly in a thin sheet through the swale.
- At least once annually, inspect vegetation and revegetate the swale to maintain a dense growth of vegetation.
- Grassed swales shall be routinely mowed such that the maximum height of vegetation is 6-inches.

2.10 Water Quality Swales

Water quality swales are similar to grass swales except a check dam is installed to capture the WQv and drain it through engineered soil to an underdrain. Larger storm events flow over the check dam.

Minimum Design Standards

- Design for the $WQ_{v.}$
- Only dry water quality swales will be permitted in the City and are not suited for intermittent streams or conditions with high ground water.
- The swale shall be designed to adequately convey the design storm of the storm drainage system.
- Velocities during the design storm should be limited to less than 2.5 ft/s.
- The average depth of the engineered soil depth above the underdrain shall be at least 1.0 foot.
- Bottom slopes of the swale should be graded as close to zero as drainage will permit.
- Swale slope should not exceed 4 percent (2 percent is preferred).
- Swale cross-section should have side slopes of 3:1 (h:v) or flatter.
- Engineered soil shall have a high permeability (fc > 0.5 inches per hour) and be of the same quality defined for the bioretention facility.
- Dense cover of a water tolerant, erosion resistant grass should be established.

Recommended Standards

- As a BMP, water quality swales are limited to residential or institutional areas where the percentage of impervious area is relatively small.
- Seasonally high water table should be more than 3 feet below the bottom of the swale.
- Check dams can be installed in swales to promote additional infiltration. The recommended method is to sink a railroad tie halfway into the swale. Riprap stone should be placed on the downstream side to prevent erosion.
- Maximum ponding time behind a check dam is to be less than 48 hours. Minimum ponding time of 30 minutes is recommended to meet water quality goals.

Unique Calculations

- The water quality volume storage calculation is required.

Operation and Maintenance Recommendations

- A stormwater maintenance manual is required for each facility. The manual should require the owner of the water quality swale to periodically clean the underdrain.
- Water quality swale should be maintained to keep grass cover dense and vigorous.
- Maintenance should include periodic mowing, occasional spot reseeding, and weed control. Swale grasses should never be mowed close to the ground. Grass heights in the 4 to 6 inch range are recommended.
- Fertilization of water quality swale should be done when needed to maintain the health of the grass, with care not to over-apply the fertilizer.

Figure 2.6 - Schematic of a Water Quality Swale - Cross Section

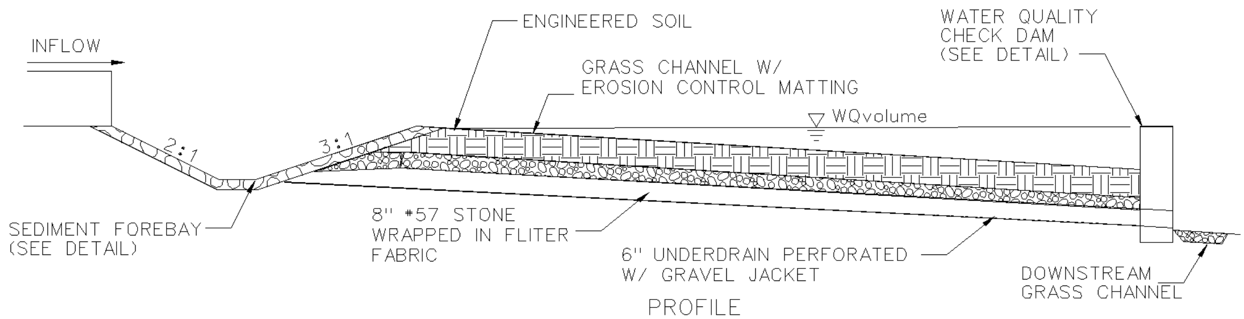
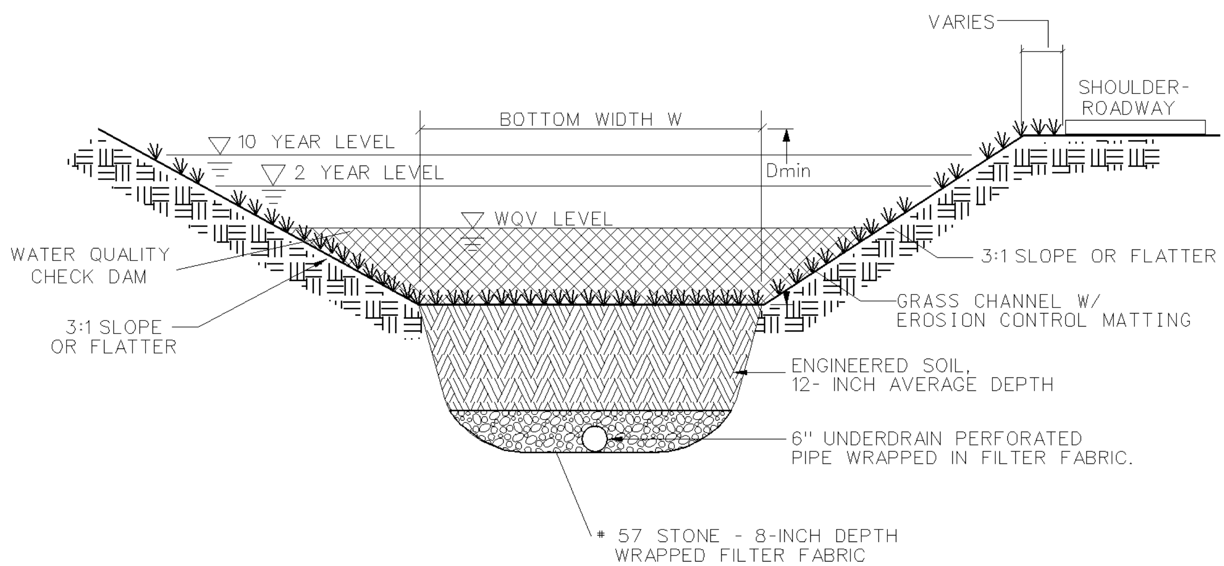


Figure 2.7 - Schematic of a Water Quality Swale – Profile



2.11 Vegetated Filter Strips with Level Spreader

Filter strips are uniformly graded and densely vegetated sections of land engineered and designed to treat runoff and remove pollutants through vegetative filtering and infiltration. Filter strips are best suited to treating runoff from roads and highways, roof downspouts, very small parking lots, and pervious surfaces. They are also ideal components of the "outer zone" of a stream buffer, or as pretreatment for another structural stormwater control. Filter strips can serve as a buffer between incompatible land uses, be landscaped to be aesthetically pleasing, and provide groundwater recharge in areas with pervious soils. Filter strips are often used as a stormwater site design credit.

Filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. There can also be a significant reduction in runoff volume for smaller flows that infiltrate pervious soils while contained within the filter strip. To be effective, however, sheet flow must be maintained across the entire filter strip at a depth of around 1-inch. Once runoff flow concentrates, it effectively short-circuits the filter strip and reduces any water quality benefits.

Special provisions must be made to ensure design flows spread evenly across the filter strip. Therefore, a level spreader must be included in the filter strip design. Filter strips are susceptible to damages from large intense storm events. Once damaged, the filter strip will no longer provide the pollutant removal and could damage the level spreader structure. Therefore, the City of Rocky Mount prefers that the WQv be diverted to the vegetated filter strip to reduce the potential damage from large storm events.

Minimum Design Standards

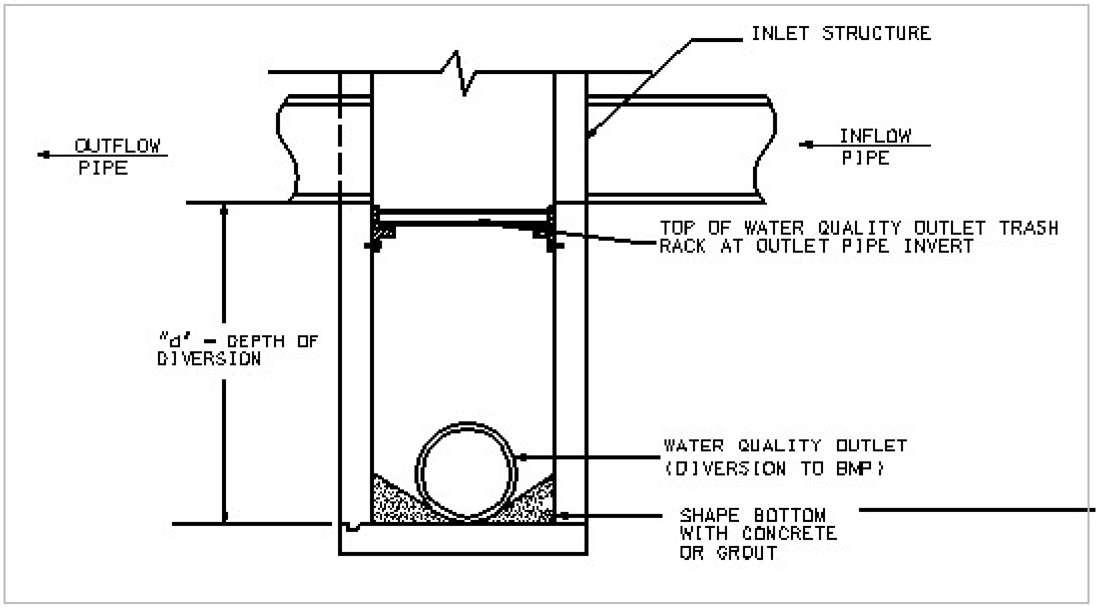
- Filter strips shall be constructed outside the riparian stream buffer area.
- Filter strips shall be designed for slopes between 2% and 6%. Greater slopes than this would encourage the formation of concentrated flow. Flatter slopes would encourage standing water.
- Filter strips should not be used on soils that cannot sustain a dense grass cover with high retardance. Designers should choose a grass that can withstand relatively high velocity flows at the entrances, and both wet and dry periods.
- The width of the filter strip shall maintain the WQv peak flow at maximum depth of 1-inch.
- The length of the filter strip shall be designed such that the 2-year 24-hour storm flow has a minimum travel time of 5 minutes.
- In no case shall the vegetated filter strip be less than 15 feet long to provide filtration and contact time for water quality treatment.

- The WQv peak flow shall be diverted to the level spreader unless calculations demonstrate the flow depths for the larger events do not cause erosive velocities.
- When the WQv is diverted, the flow from the larger storms shall be conveyed in an adequate drainage system to the receiving stream or drainage system.
- The length of the level spreader shall be equal to the width of the vegetated filter strip.
- The level spreader lip shall be constructed at 0% grade and be of a rigid material that will not deform over time.
- The level spreader shall have a depth of at least 6-inches to allow for the water to pond up prior to flowing into the vegetated filter strip.
- The bottom of the level spreader shall have weep holes and a gravel drain to prevent standing water.
- Pedestrian traffic across the filter strip should be limited through channeling onto sidewalks.

Recommended Standards

- The vegetated filter strip should be sodded. Establishing a dense uniform grass from seed is difficult and often requires multiple growing seasons.
- If the drainage area is expected to be heavy in sediments, a sediment forebay should be installed prior to the level spreader.
- The preferred diversion structure is a drop inlet with a modified bottom as shown in Figure 2.8.

Figure 2.8 – Diversion Structure



Unique Calculations

- The filter strip width is based on the maximum discharge loading per foot of the filter strip. This is determined using the following form of the Manning's equation:

$$q = \frac{0.00236}{n} Y^{5/3} S^{1/2}$$

Where:

$q =$ discharge per foot of width of filter strip (cfs/ft)
$Y =$ allowable depth of flow (inches)
$S =$ slope of filter strip (percent)
$n =$ Manning's "n" roughness coefficient
(use 0.15 for medium grass, 0.25 for dense grass, and 0.35 or very dense Bermuda-type grass)

Therefore the minimum width of the filter strip is:

$$Wf\ \mathrm{min} = \frac{Q}{q}$$

Where:

$Wf\ \mathrm{min} =$ minimum filter strip width perpendicular to flow (feet)

$Q =$ WQv peak flow (cfs)

To determine the minimum length of the filter strip, use the overland travel time equation from the NRCS time of concentration methodology presented in Chapter 3 and solve for length. The equation is provided below:

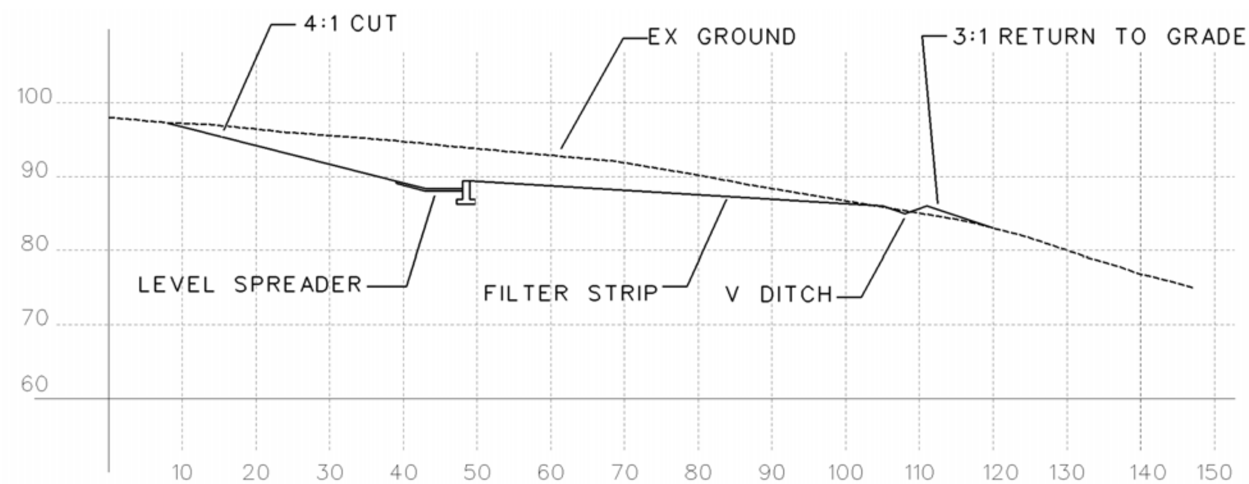
$$Lf = \frac{(Tt)^{1.25}(P_{2-24})^{0.625}(S)^{0.5}}{3.34(n)}$$

Where:

$Lf =$ length of filter strip parallel to flow path (ft)
$Tt =$ minimum travel time through filter strip (minutes)
$P_{2-24} =$ 2-year, 24-hour rainfall depth (inches)
$S =$ slope of filter strip (percent)
$n =$ Manning's "n" roughness coefficient (use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)

Minimum Operation and Maintenance Requirements
- Filter strips require similar maintenance to other vegetative practices. Maintenance is very important for filter strips, particularly in terms of ensuring that flow does not short circuit the practice.
- Mow grass regularly (frequently) to maintain a 2 to 4 inch height.
- Inspect level spreader and remove built up sediment.
- Inspect the diversion structure and remove collected debris and properly dispose.
- Inspect vegetation for rills and gullies and correct. Seed or sod bare areas.
- Inspect to ensure that grass has established. If not, replace with an alternative species.

Figure 2.9 - Filter Strip with Level Spreader



2.12 Bioretention (Rain Gardens)

Bioretention areas, or rain gardens, are structural stormwater controls that capture and temporarily store the WQ_v using soils and vegetation in landscaped areas to remove pollutants from stormwater runoff as shown in Figure 2.10. Bioretention areas are engineered facilities in which runoff is conveyed to the "treatment area," consisting of a sediment forebay ponding area, organic or mulch layer, planting soil, and vegetation. The filtered runoff is collected in an underdrain system and returned to the site drainage system. Although bioretention areas have some affect on peak flows, it is not recommended these areas be used for site peak flow reduction control because this will likely require ponding depths that may destroy the vegetation.

Bioretention areas are also susceptible to damage from large intense storm events. These events can overwhelm a facility dislocating or removing the mulch layer and damaging the vegetation. Once damaged, the bioretention area does not provide the pollutant removal and typically becomes a nuisance given that they are generally located in highly visible areas. Therefore, the City of Rocky prefers that the WQv be diverted to the bioretention area to reduce the potential damage from large storm events.

Minimum Design Standards

- Design for the WQ_v.
- The WQv shall drain through the bioretention within 2 days with the filter (engineered soil) coefficient of permeability (k) of 0.5 ft/day.
- The engineered soil shall be a minimum depth of 2.5 feet and consist of sandy loam, loamy sand or loam texture with a clay content rating from 10 to 25 percent.
- The engineered soil must have an infiltration rate of at least 0.5 inches per day and a pH between 5.5 and 6.5.
- The engineered soil must have a 1.5 to 3 percent organic content and a maximum 500-ppm concentration of soluble salts.
- The engineered soil must have a Phosphorus Index between 20 and 40.
- The maximum ponding depth above the mulch layer is 6-inches.
- The contributing drainage area must be 5 acres or less, though 0.5 to 2 acres is preferred.
- The WQv shall be diverted to the bioretention area unless calculations demonstrate the larger storm events do not increase the ponding depth to more than 9-inches above the mulch layer and the larger flow events can be adequately dispersed through the area.
- Sediment forebays shall be designed to handle particles greater than 40 microns in size.
- Sediment forebays shall be separated from the engineered soil with an impervious layer of material to prevent short circuiting to the underdrain system.

- The sediment forebay shall be used to sheet the flow across the mulch area.
- The mulch layer should consist of approximately 3 inches of commercially available triple hardwood mulch.
- The underdrain collection system should be equipped with a 6-inch perforated schedule 40 PVC or greater strength pipe in an 8-inch gravel layer wrapped in filter fabric. The pipe should have 3/8-inch perforations, spaced on 6-inch centers with a minimum of 4 holes per row.
- Underdrain pipes should be spaced at a maximum of 10 feet on center, and a minimum grade of 0.5% must be maintained.
- The depth to the water table from the bottom of the bioretention facility to the high water table should be a minimum of 2 feet.
- Runoff captured by the facility must be sheet flow to prevent erosion of the organic or mulch layer.
- Bioretention areas are designed for intermittent flow and to drain and aerate between rainfall events. Sites with continuous flow from groundwater, sump pumps or other areas should be avoided.
- An overflow structure and a non-erosive overflow channel must be provided to safely pass the flow from the bioretention area that exceeds the storage capacity to a stabilized downstream area. The high flow structure within the bioretention area can consist of a grated drop inlet with the top set 6 inches above the mulch layer.
- If the bioretention area is confined by an earthen berm, the top of the berm should be set with 6-inches of freeboard above the anticipated maximum water surface elevation.

Unique Calculations

- The sediment forebay surface area shall be designed using the Camp-Hazen equation, which accounts for the effects of turbulent flow. This equation is provided below:

$$As = \left(\frac{Qo}{W}\right)(E')$$

Where:

$As =$	sedimentation basin surface area (ft^2)
$Qo =$	discharge rate from basin = (WQv/24 hr)
$W =$	particle settling velocity (ft/sec) 0.0033 ft/sec (particle size=40 microns)
$E' =$	sediment trapping efficiency constant; for a sediment trapping efficiency (*E*) of 90%, *E'* = 2.30

The sediment trapping efficiency constant (*E'*) may be calculated from the sediment trapping efficiency (*E*) using the following equation:

$$E' = -\ln\left[1 - \left(E/100\right)\right]$$

The equation reduces to:

$Asf = (0.0081)(WQv)$ for 40 micron particle size

Where:

$Asf =$ sedimentation basin surface area full (square feet)

The required bioretention area is based on the Darcy equation. This equation calculates the required surface area using the following equation:

$$Af = \frac{(WQv)(df)}{((k)(hf + df)(tf))}$$

Where:

$Af =$ Surface area of filter bed (ft^2)
$WQv =$ water quality volume (ft^3)
$df =$ filter bed depth (ft)
$k =$ coefficient of permeability of filter media (ft/day)
$hf =$ average height of water above filter bed (ft)
$tf =$ design filter bed drain time (days)

Recommended Standards

- Bioretention areas can be incorporated into the landscaping plan as depressed parking lot islands.
- A dense and vigorous groundcover should be established over the contributing pervious drainage area before runoff can be diverted into the facility.
- Use native plants, selected based upon hardiness and hydric tolerance.

Operation and Maintenance Requirements

- The bioretention area should be periodically cleaned and dead, dying or diseased plant material replaced.
- Inlets should be inspected for signs of erosion after every significant rainfall.
- Sediment forebay should be cleaned when 50% full of sediment.
- The sediment forebay lip should be inspected to ensure sheet flow.
- Vegetation should be kept healthy.
- The mulch will need to be replaced and/or replenished on an annual basis. If the mulch layer becomes substantially clogged with sediment, remove the mulch and replace.

Figure 2.10 - Bioretention Area Plan View

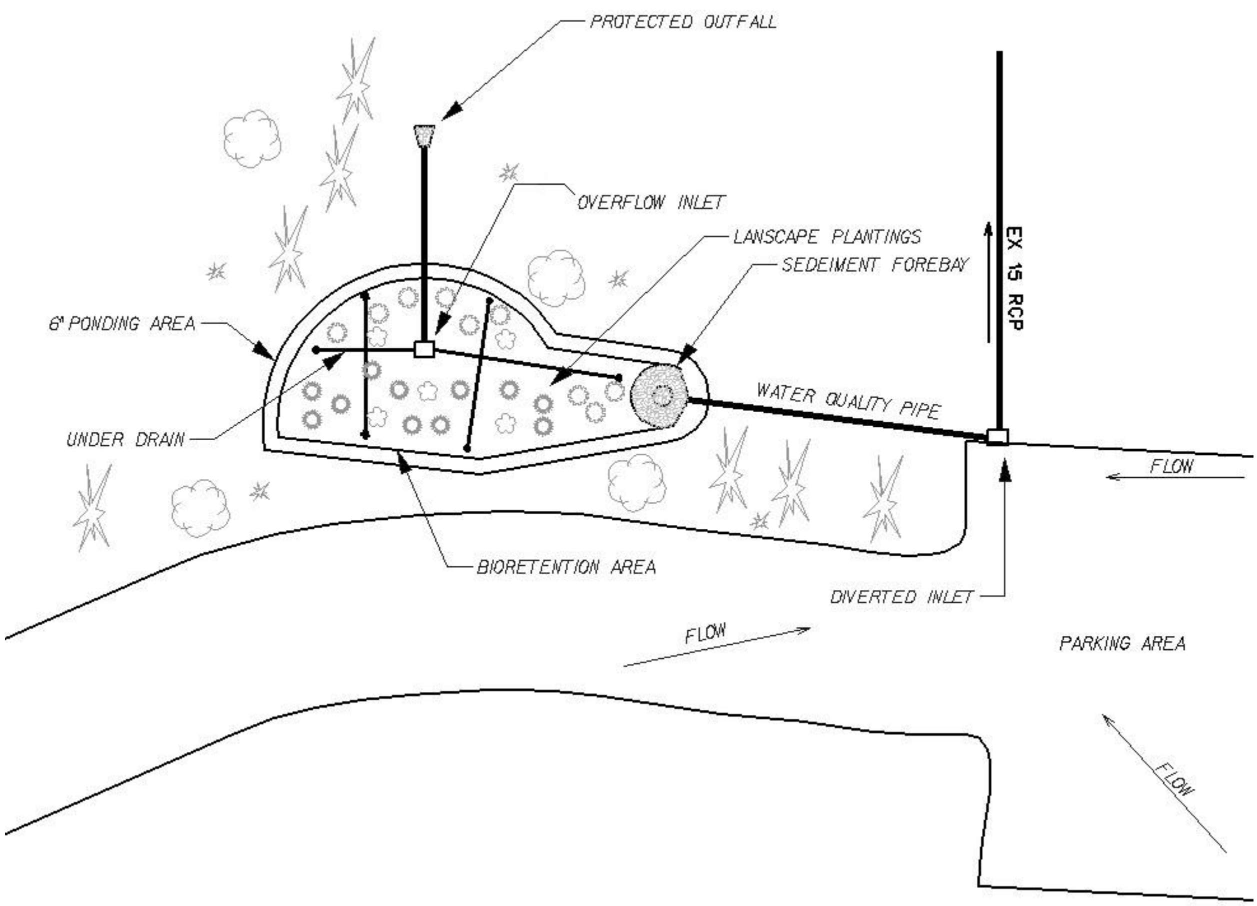
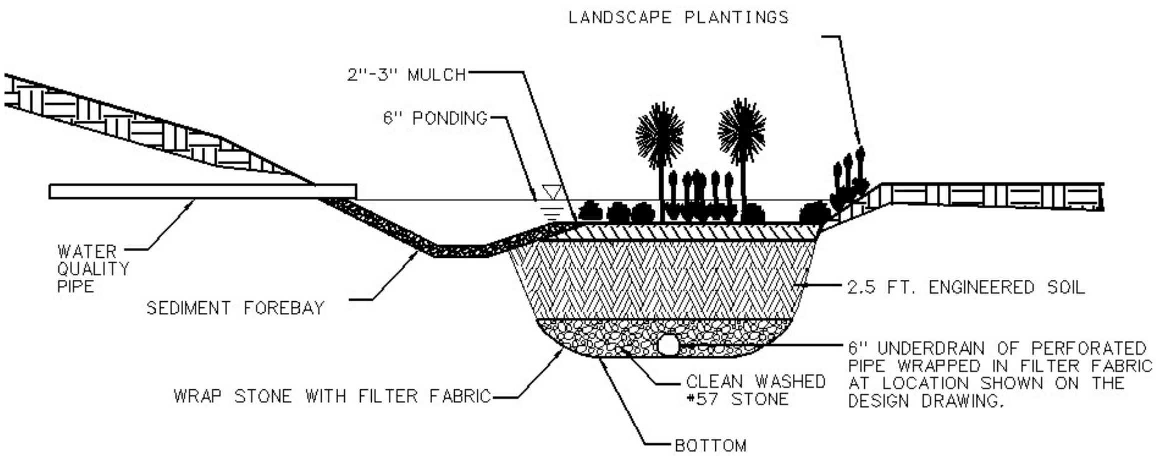


Figure 2.11 - Bioretention Area Section View



2.13 Sand Filters

Sand Filters are structural stormwater controls that capture and temporarily store the WQ_v either in open areas (as shown if Figure 2.12) or in underground vaults (as shown in Figure 2.13) and allow the water to filter through a sand media. These structural BMPs are similar to Bioretention areas except they can be installed under ground in vaults. Most sand filters have two chambers. The first chamber is a sedimentation chamber that removes floatables and other heavy sediments. The second chamber is the filtration chamber. This chamber removes additional pollutants by filtering the runoff through a sand bed. The filtered runoff is collected in an underdrain system and returned to the site drainage system. Although sand filters areas have some affect on peak flows, it is not recommended these areas be used for site peak flow reduction control.

Sand Filters areas are also susceptible to damage from large intense storm events. These events can overwhelm a facility dislocating and shifting the sand media and resususpending trapped sediments. Therefore, the City of Rocky Mount prefers that the WQv be diverted to the sand filters to reduce the potential damage from large storm events.

Minimum Design Standards

- Maximum contributing drainage area to an individual sand filter should be less than 5 acres.
- Design volume based on WQ_v.
- Designed to completely empty in 36 hours with a sand bed depth of at least 18-inches.
- Sand permeability shall be 3.5 ft/day.
- A diversion structure shall be located such that the sediment chamber and filter only receive the WQv and large storms are adequately bypassed.
- A sediment chamber shall be designed to capture the 40 micron particle size. The volume of the sediment forebay should be subtracted from the WQv when calculating the required surface area of the media.
- Inlet structure to the sand media should be designed to spread the flow uniformly across the surface.
- The underdrain collection system should be equipped with a 4 inch perforated schedule 40 PVC or greater strength pipe and wrapped in filter fabric. The pipe should have 3/8-inch perforations, spaced on 6-inch centers with a minimum of 4 holes per row.
- Minimum grade of underdrain piping should be 0.5 %.
- Access for cleaning all underdrain piping should be provided.
- Surface filters may have a grass cover to aid in pollution adsorption.
- Sand filters shall not be placed into service until the ground cover of the contributing drainage has been established.

Unique Calculations

- The sediment forebay surface area shall be designed using the Camp-Hazen equation presented in Chapter 3.
- The filter surface area shall be designed using the Darcy equation presented in Chapter 3.

Recommended Standards

- Incorporating an outlet device that will also trap oil and grease will enhance the performance of the sand filter and will decrease the maintenance frequency required to maintain effective performance.
- If a surface sand filter is used, incorporate a vegetative cover to increase nutrient removal potential.

Operation and Maintenance Recommendations

- Scrape off sediment layer buildup during dry periods with steel rakes or other devices.
- Replace some or all of the sand when permeability of the filter media is reduced to unacceptable levels, which should be specified in the design of the facility. A minimum infiltration rate of 0.5 inches per hour should be used for all infiltration designs.

Figure 2.12 - Surface Sand Filter
Source: Center for Watershed Protection

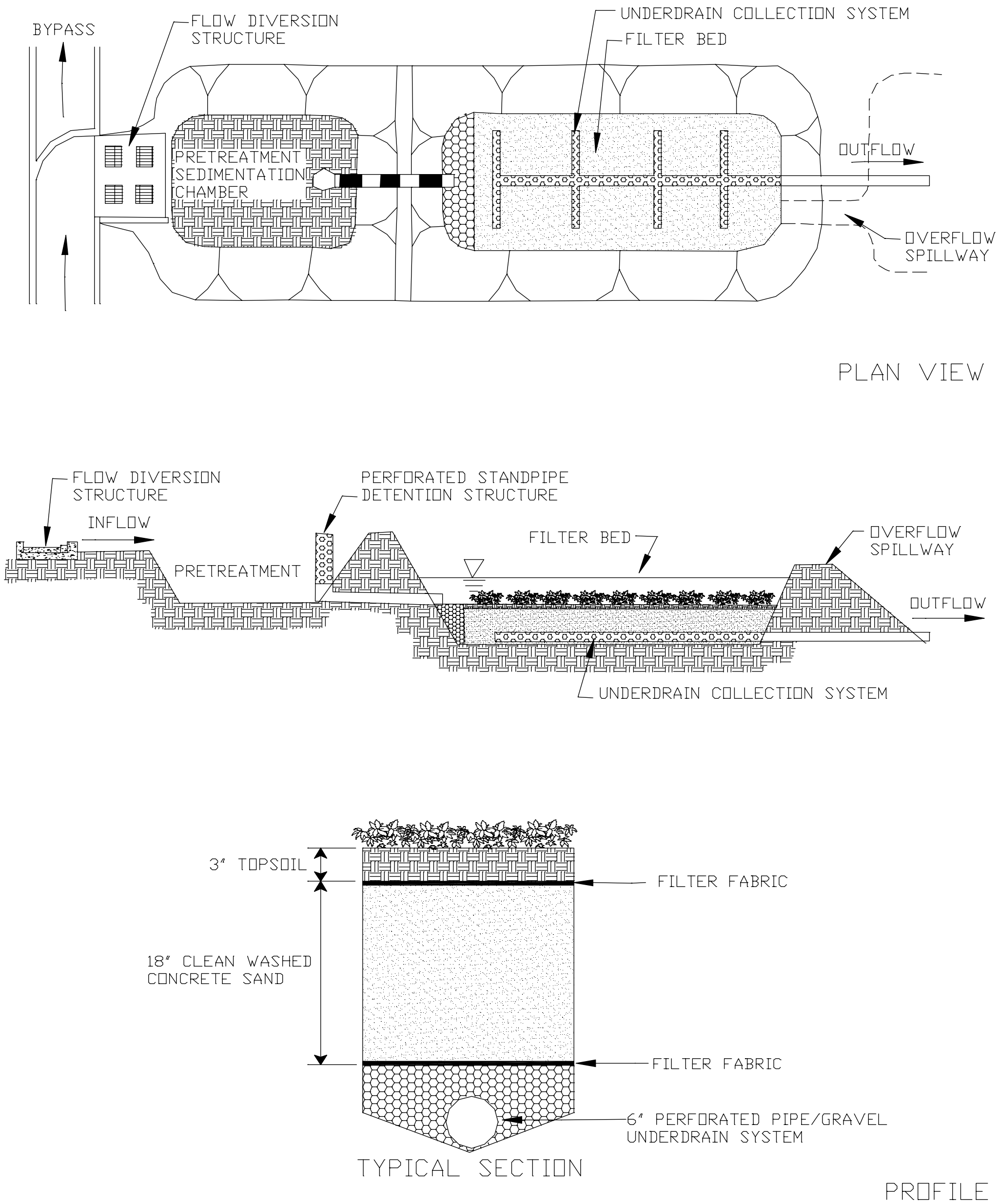
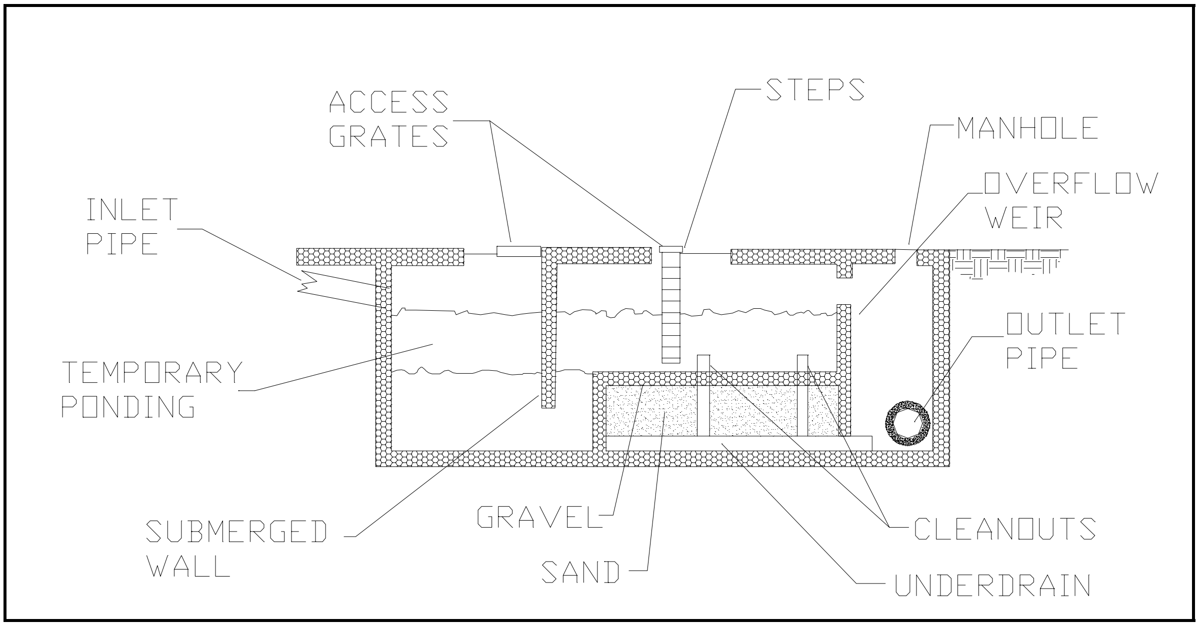


Figure 2.13 Underground Sand Filter



2.14 Permeable Pavement

Traditional paved surfaces, such as asphalt and concrete, do not allow water to infiltrate and convert almost all rainfall into runoff. If designed and implemented correctly, permeable pavement systems, as shown in Figure 2.14, allow at least a portion of stormwater to infiltrate, thus reducing peak runoff volumes and flows. Permeable paving materials include, but are not necessarily limited to, porous concrete, permeable interlocking concrete pavers, concrete grid pavers, and porous asphalt. Compacted gravel will not be considered as permeable pavement.

Design and installation of permeable pavement systems must be performed by appropriate professionals. The primary factors that should direct permeable pavement design include the following:

1. Providing adequate infiltration and temporary storage
2. Preventing sediment, oils, and greases from reaching the permeable pavement surface where they have the potential to clog
3. Using construction techniques that minimize the compaction of subsurface soils

FIGURE 2.14
Various permeable pavement systems
Courtesy of NC State University – Biological and Agricultural Engineering Department

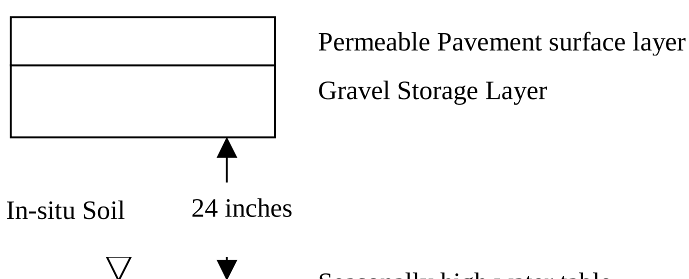
Minimum Design Standards

- A washed aggregate base must be used, and washed 57-size stone is generally acceptable. Fine particles from standard “crusher run” will clog the pores at the bottom of the pavement and will not be allowed.

- Traffic volume must be less than 100 vehicles per day.

- As shown in Figure 2.14 below, seasonally high water table must be at least 2 ft from the base of the permeable pavement or gravel storage layer. Water tables approaching the permeable pavement system will not allow water to exfiltrate.

FIGURE 2.15
Schematic of water table design constraint.



- The completed permeable pavement must be installed at a grade less than 0.5%. Steeper slopes will reduce the storage capacity of the permeable pavement.

- Permeable pavement systems are not allowed in areas, such as buffers, where impervious surfaces are not permitted.

- The construction sequence must be inspected to insure that the surface installation is planned to be completed after adjacent areas are stabilized with vegetation. Run-on to the permeable pavement from exposed areas can cause the system to perform ineffectively.

- The *in-situ* soils beneath the permeable pavement must have sufficient infiltration capacity for the permeable pavement to drain. To satisfy this requirement, the following conditions must be met:

    - The footprint of the permeable pavement installation must have a vertical saturated hydraulic conductivity of at least 2 in/hr for the top 3 ft of soil as determined by a soil analysis.

    - The top 3 ft of soil must also have no finer texture than Loamy Very Fine Sand as defined by the United States Department of Agriculture – Natural Resources Conservation Service (USDA-NRCS) and as determined by a soil analysis.

    - Only 2 ac-ft of soil per acre disturbed can be moved for the footprint of the permeable pavement. Mass grading can significantly alter the site's applicability for permeable pavement. If mass grading occurs and conditions (a) and (b) are still met, then an exception for this requirement can be given. However, a soil analysis will be required after the grading is completed to verify the soil properties.

Recommended Specifications

- Permeable pavement should not be placed where upland land disturbance is occurring or will potentially occur. Land disturbance upland of the lot could result in frequent pavement clogging.

- Avoid overhanging trees above the permeable pavement installation.

- During preparation of the subgrade, special care must be made to avoid compaction of soils. Compaction of the soils can reduce the infiltration capacity of the soil.

- Permeable pavement should not be designed to receive concentrated flow from roofs or other surfaces. Incidental run-on from stabilized areas is permissible, but the permeable pavement should primarily be designed to infiltrate the rain that falls on the pavement surface itself. No credit will be given for volume or peak reduction for run-on from impervious surfaces.

Unique Calculations

Permeable pavement will not receive direct credit for any pollutant removal (percent reduction, e.g.). However, for the purpose of meeting pollutant control requirements, credit received for reducing percent imperviousness will have the effect of reducing pollutant loads. The extent of pollutant reduction will depend on the site configuration.

For permeable pavement systems, credit will be given such that a portion of the permeable pavement will not be counted as impervious for built-upon-area calculations. Depending on the type of system used and its construction (see Table 2.4), a portion of the permeable pavement will be counted as "managed grass." The remainder will be counted as impervious.

TABLE 2.4: CREDIT RECEIVED FOR VARIOUS PERMEABLE PAVEMENT SYSTEMS

Permeable Pavement System	Credit as Percent Managed Grass
Permeable concrete without gravel base	40 %
Permeable concrete with at least 6" of gravel base	60 %
Flexible pavements with at least 4" of gravel base	40 %
Flexible pavements with at least 7" of gravel base	60 %

Operation and Maintenance Recommendations

Maintenance requirements are critical for the success of permeable pavement. The following installation and maintenance requirements are designed to ensure that the permeable pavement system will work effectively. A maintenance agreement is required for each permeable pavement installation to receive credit. The maintenance agreement should include specific requirements and responsibilities of the property owner and provide for enforcement.

- Inspect monthly for first 3 months after installation.
- Inspect annually after first three months.
- Inspections should ensure that:
  - Permeable pavement surface is free of sediment
  - Contributing and adjacent areas are stabilized and mowed with clippings removed
  - There is no deterioration of pavement system
  - The permeable pavement system is dewatering between storm events
- Perform repairs of permeable pavements with similar permeable materials
- Vacuum sweep permeable pavement surface annually

City of Rocky Mount, North Carolina
Stormwater Design Manual

Chapter 3: Stormwater Design Calculations

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Chapter 3: Stormwater Design Calculations

3.1 Introduction

The design of properly sized storm drainage systems requires some knowledge of the hydrologic behavior of the watershed in question and hydraulic principles of fluids. For adequate design of gutters, catch basins, inlets, storm drainage pipes, open channels and culverts it is appropriate to estimate the peak discharge of the drainage area for the required design frequency. The peak discharge is then used to calculate the capacity of the storm drainage system based on the system's hydraulic characteristics.

When a watershed is large and complicated, it may be necessary to generate, route and add hydrographs to determine the peak discharge. When it is necessary to control the peak flow, hydrographs must be generated and routed to demonstrate the peak flow reduction. When routing hydrographs, it is easiest and most reliable to use an established computer model as long as the model utilizes the acceptable methodologies presented in this manual.

This chapter presents the accepted hydrologic and hydraulic calculations and methodologies to generate peak discharges and hydrographs for use in the design of stormwater drainage systems within the City of Rocky Mount. This section is not intended to be a design reference manual and the designers are expected to be familiar with the identified source publications.

3.2 Computer Software

The City of Rocky Mount Engineering Department utilizes the Hydraflow software packages to perform internal designs and independent design verifications on peak discharges, hydrograph generation, impoundment routings, storm drainage pipe design, spread calculations and culvert design. The City of Rocky Mount acknowledges that there are numerous software packages that adequately perform the calculations described in this chapter and some calculations can be adequately performed without the use of software packages. The software packages listed in Table 3.1 are accepted by the City of Rocky Mount as long as the methods and parameters described in this Chapter are followed.

Table 3.1 – Computer Software

Software	Typical Use
Hydraflow Hydrographs	Peak Flows, Hydrograph Routings
Hydraflow Storm Sewers	Storm Drainage Pipes, HGL's, Gutter spread, Inlet Design
Hydraflow HEC-RAS	Culverts, Floodplains, Open Channels
HEC-HMS	Peak Flows, Hydrograph Routings
HEC-RAS	Culverts, Irregular Channels
Pond Pack	Peak Flows, Hydrograph Routings
Storm CADD	Storm Drainage Pipes, HGL's, Gutter Spread, Inlet Design
TR-20	Peak Flows, Hydrograph Routings
HY-8	Culvert
HEC-12	Gutter Spread, Inlet Design

In order to maintain consistency with design calculations, the City has provided design forms and checklists in Appendix B that must be completed by the designer. The designer must also provide the City with a copy of the relative computer outlet, inlet and output design files.

In special scenarios, other software packages or methods described in this chapter are more appropriate. If the designer prefers to use a computer model other than that described here, the designer must receive written authorization from the Director of Engineering. When requesting the use of another model, the designer must provide a written description for the need to use the computer software or other methodology along with supporting technical information to evaluate the request.

3.3 Peak Discharge

The City of Rocky Mount allows peak discharges to be calculated using either the Rational Method or the NRCS (SCS) Method. These two methods are presented below.

3.3.1 Rational Equation

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed). The rational formula is expressed as follows:

Rational Equation Formula

$$Q = (C)(I)(A)$$

Where:

$Q =$ peak flow from the drainage area (cfs)
$C =$ coefficient of runoff (dimensionless)
$I =$ rainfall intensity for a given time to peak (in/hr)
$A =$ drainage area (acres)

The rational equation is based on the assumption that rainfall is uniformly distributed over the entire drainage area at a steady rate, causing flow to reach a maximum at the outlet to the watershed at the time to peak (Tp). The rational method also assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area, then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

The rational equation shall only be used for drainage areas less than 100 acres.

3.3.1.1 Runoff Coefficient

The runoff coefficient (C) is the variable of the rational method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 3.1 gives the recommended runoff coefficients for the rational method.

Table 3.1 Recommended Runoff Coefficient Values

(Sources: North Carolina Erosion and Sediment Control Planning and Design Manual and The City of Rocky Mount's Minimum Storm Drainage Design Requirements)

Description of Area	Runoff Coefficient, C
Woodlands	0.20 - .025
Parks, cemeteries	0.25
Playgrounds	0.30
Lawns and Cropland:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Graded or no plant cover	
Sandy soil, flat, 0-5%	0.30
Sandy soil, average, 5 - 10%	0.40
Clay soil, flat, 0-5%	0.50
Clay soil, average, 5-10%	0.60
Residential:	
R-15, very low density	0.50
R-10. low density	0.50
R-8, manufactured	0.55
R-6, single family	0.55
R-6MFA, medium density multi-family	0.60
MFA, multi-family	0.70-0.75
MHP, mobile home park	0.75
Business:	
O & I and all B Zones	0.85
All Industrial Zones	0.85-0.95
Commercial/Shopping Centers	0.85-0.95
Streets	
Gravel areas	0.50
Drives, walks, roofs	0.95
Asphalt and Concrete	0.95-1.00

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 3.1 by using percentages of different land uses, as illustrated in the Composite C Equation below. In addition, more detailed composites can be made with coefficients for different surface types such as roofs, asphalt, and concrete streets, drives and walks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

Composite C Equation:

$$CompositeC = \frac{(C1)(A1) + (C1)(A2) + ...(Cx)(Ax)}{A1 + A2 + ...Ax}$$

3.3.1.2 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in inches/hour for a duration for a selected return period. The duration is equal to the time of concentration (Tc) for the drainage area. Acceptable time of concentration methods is presented in section 3.1.3. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from the intensity-duration-frequency (IDF) data for the City of Rocky Mount given in Table 3.2.

Table 3.2 Intensity – Duration - Frequency Table
City of Rocky Mount, NC

Tc (duration)	Frequency (Yrs)						
	1	2	5	10	25	50	100
5 mins	4.48	5.76	6.58	7.50	8.19	8.96	9.72
10	3.73	4.76	5.54	6.13	7.01	7.71	8.40
15	3.20	4.04	4.74	5.25	6.03	6.64	7.24
20	2.80	3.47	4.12	4.64	5.42	5.93	6.47
30	2.49	2.70	3.28	3.71	4.32	4.80	5.28
40	1.87	2.28	2.77	3.15	3.70	4.08	4.48
50	1.60	1.94	2.38	2.71	3.19	3.53	3.88
60	1.40	1.70	2.12	2.41	2.84	3.17	3.50
90	1.02	1.22	1.52	1.74	2.06	2.29	2.53
2 hr	0.80	0.95	1.20	1.37	1.62	1.81	2.00
3	0.56	0.71	0.89	1.02	1.21	1.35	1.50
6	0.30	0.44	0.56	0.65	0.77	0.86	0.96
12	0.15	0.26	0.33	0.39	0.46	0.52	0.57
24	0.08	0.15	0.19	0.22	0.27	0.30	0.33

3.3.2 NRCS (SCS) Method

The NRCS (SCS) method utilizes the Runoff Curve Number (RCN), Type II distribution storm event, and the SCS Dimensionless Unit Hydrograph to calculate a peak discharge. This method has been incorporated into many computer software packages and is the preferred method in the City of Rocky Mount when comparing pre-developed and post-developed peak discharges for the various frequency events. The graphical method to calculate peak discharges is summarized in this section and is described in detail in the SCS Urban Hydrology for Small Watersheds, Technical Release No. 55, Second Edition.

The graphical method is limited to the family of Ia/P curves presented later. This method should not be used when results are outside the family of curves provided. In this case, a computer model should be used as long as the computer model parameters fall within the time step limitations to adequately model the entire storm event.

The NRCS method requires the calculation of the amount of water during a given rainfall event that will not soak into the ground or fill up small voids in the surface. This amount of runoff is based on the RCN. The amount of runoff is converted to a peak discharge based on the relationship between the family of curves developed by the SCS using the Type II storm distribution and the SCS Dimensionless Unit Hydrograph.

3.3.2.1 Runoff Volume

The amount of rainfall that turns into runoff is based on the land cover type, soil type, and antecedent moisture content (AMC). NRCS developed RCN's based on these three parameters. Table 3.3 summarizes the RCN's for the AMC II conditions, which is considered normal. The soil types are divided into four major hydrologic soil groups denoted by the letters A through D. A soils are those which have high infiltration capacity and subsequently low runoff rates. D soils are those with very low infiltration capacity and very high runoff rates. A list of soils common in Nash and Edgecombe Counties can be obtained from the respective counties local NRCS field office.

Table 3.3 Runoff Curve Numbers[1]

Cover Description	Curve Numbers for Hydrologic Soil Groups			
Cover type and hydrologic condition	**A**	**B**	**C**	**D**
Cultivated land:				
without conservation treatment	72	81	88	91
with conservation treatment	62	71	78	81
Pasture or range land				
poor condition	68	79	86	89
good condition	39	61	74	80
Meadow:				
good condition	30	58	71	78
Wood or forest land:				
thin stand, poor cover	45	66	77	83
good cover	25	55	70	77
Open Space (lawns, parks, golf courses, cemeteries, etc.) [2]				
Poor condition (grass cover <50%)	68	79	86	89
Fair condition (grass cover 50% - 75%)	49	69	79	84
Good condition (grass cover >75%)	39	61	74	80
Impervious areas:				
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)	98	98	98	98
Streets and Roads:				
Paved; curbs and storm drains (excluding right-of-way)	98	98	98	98
Paved; open ditches (including right-of-way)	83	89	92	93
Gravel (including right-of-way)	76	85	89	91
Dirt (including right-of-way)	72	82	87	89
Developing urban areas and newly graded areas (pervious area only, no vegetation)	77	86	91	94
Urban districts by zoning:				
O& I and all B-Zones	96	97	98	98
Industrial Zones	98	98	98	98
Commercial/Shopping Centers				
Residential districts by zoning:				
R-15, very low density	61	75	83	87
R-10. low density	61	75	83	87
R-8, manufactured	71	80	87	92
R-6, single family	71	80	87	92
R-6MFA, medium density multi-family	80	85	90	95
MFA, multi-family	86	90	93	96
MHP, mobile home park	92	94	96	97

1 Average runoff condition, and $I_a = 0.2S$

2 CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type. Assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the NRCS method has an adjustment to reduce the effect.

3.3.2.2 Runoff Volume Equation

The volume of flood runoff can be calculated by the following equation.

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S}$$

Where:

$Q =$ accumulated direct runoff (in.)

$P =$ accumulated rainfall (potential maximum runoff) (in.) found in Table 3.4

$Ia =$ initial abstraction including surface storage, interception, and infiltration prior to runoff (in.) = 0.2*S

$S =$ potential maximum soil retention (in.)

$$\left(\frac{1000}{RCN}\right) - 10$$

Table 3.4 Accumulated 24-hour Rainfall Frequency City of Rocky Mount, NC

Duration	Frequency (Yrs)/Rainfall (inches)						
	1	2	5	10	25	50	100
24-hour	3.20	3.60	4.56	5.28	6.48	7.20	8.0

3.3.2.3 NRCS Peak Discharge

The peak discharge equation used by the Natural Resources Conservation Services has the form:

$$Qp = (Qu)(A)(Q)(Fp)$$

Where:

$Qp =$ peak discharge (cfs)

$Qu =$ unit peak discharge found from Figure 3.1 (csm/in)

$A =$ drainage area (sq mi)

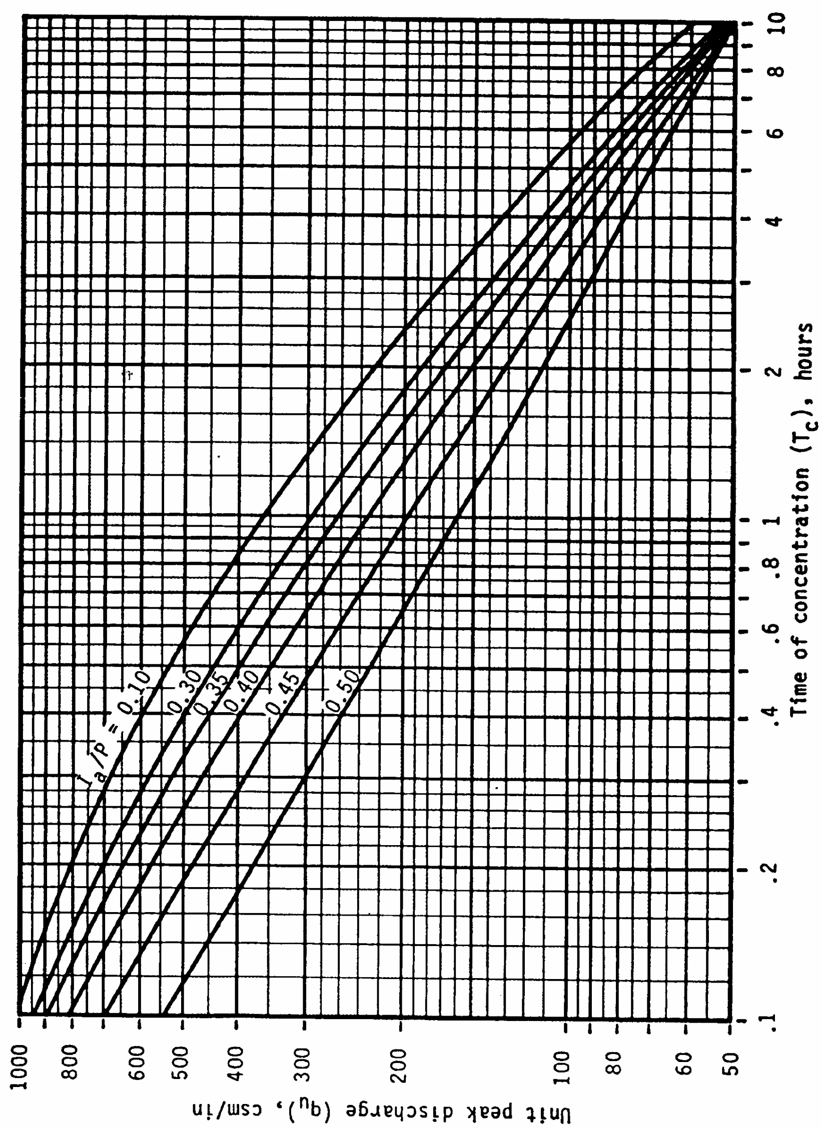
$Q =$ runoff depth (in)

$Fp =$ pond and swamp adjustment factor from Table 3.5

Table 3.5. Swamp Correction Factors	
Percentage of pond or swamp areas	Fp
0	1.00
0.2	0.97
1.0	0.87
3.0	0.87
5.0	0.72

Note that swamp correction factors should only be used if the area will remain in place. If there is a possibility the area will be re-graded in the future, no correction should be made.

Figure 3.1 NRCS Type II Unit Peak Discharge Graph



(Source: NRCS TR-55 Urban Hydrology for Small Watersheds, Second Edition, June 1986)

3.3.3 Time of Concentration

Use of the rational formula and the NRCS Unit Hydrograph requires the time of concentration (t_c) for each design point within the drainage basin. The time of concentration is considered the longest time for which the stormwater runoff has to travel to the design point. The time of concentration typically consists of an overland flow (sheet flow) time, shallow concentrated time and channel flow time. The overland flow time is sheet flow and generally does not last more than 200 feet in an undisturbed wooded or grass area. After 200 feet or in some basins shorter, the flow becomes shallow concentrated in a gutter section, vegetated swale, pipe, etc. The flow may then be conveyed to a larger system or stream where it is conveyed to the design point.

Some general guidelines when performing time of concentrations are given below:

- The minimum time of concentration for drainage area is 5 minutes.
- In some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge.
- When designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 100 feet in urban areas and 300 feet in rural areas should be done only after careful consideration. Except in very flat areas, overland flow time should not be greater than the pipe or channel flow time.

There are several acceptable methods for calculating the time of concentration. The City of Rocky Mount prefers the use of either the Kirpich or NRCS methodology as described below. If the designer has reason to use another method the designer must submit the method and supporting technical information for review and approval by the Director of Engineering.

3.3.3.1 Kirpich Equation

The Kirpich equation is based on empirical data and observation and does not break down the time of concentration flow path into different segments. Although it has no analytical basis, it has proven an effective method in many years of use. It is therefore widely considered an acceptable method for estimating time of concentration for small drainage areas of up to 10 acres. The basic form of the equation is:

Kirpich Equation

$$Tc = \frac{\left(\frac{L^3}{H}\right)^{0.385}}{128}$$

Where:

$Tc =$ time of concentration (min)
$H =$ height of the most remote point on the watershed above the outlet (ft)
$L =$ length of flow from the most remote point on the watershed to the outlet (ft)

(Civil Engineering, Vol. 10, No. 6, June 1940, p.362.)

The following adjustments are commonly made to Kirpich Equation to compensate for channelization.

- For well-defined natural channels, use T_C.
- For overland flow on grassy surfaces, use T_C * 2.
- For overland flow on paved surfaces, use T_C * 0.4.
- For concrete channels, use T_C * 0.2.

3.3.3.2 NRCS Method

The time of concentration can be broken into three types of flow, sheet flow, shallow concentrated flow, and channel flow (or pipe flow). Sheet flow is assumed to be no longer than a few hundred feet and can be described by Manning's kinematic solution:

Sheet Flow Equation

$$Tsf = \frac{(0.007)(nL)^{0.8}}{(P2)^{0.5}(S)^{0.4}}$$

Where:

$Tsf =$ travel time for sheet flow(hours)
$n =$ Manning roughness coefficient
$L =$ flow length (ft)- Maximum length = 100 feet
$P2 =$ 2-yr 24 hour rainfall (in)
$S =$ ground slope (ft/ft)

Table 3.6 Manning's "n" Value for Sheet Flow

(Source: North Carolina Erosion and Sediment Control Planning and Design Manual)

Description	"n"
Smooth surfaces:	
Concrete, asphalt	0.011
Bare soil, gravel	0.011
Sparse grasses	0.150
Dense grasses	0.240
Bermuda grass	0.410
Woods, light underbrush	0.40
Dense underbrush	0.80

<u>Sheet Flow Equation</u>

Shallow concentrated flow travel time is best estimated by calculating the average flow velocity from the figure on the following page. The travel time is estimated as the average flow velocity multiplied by the flow length.

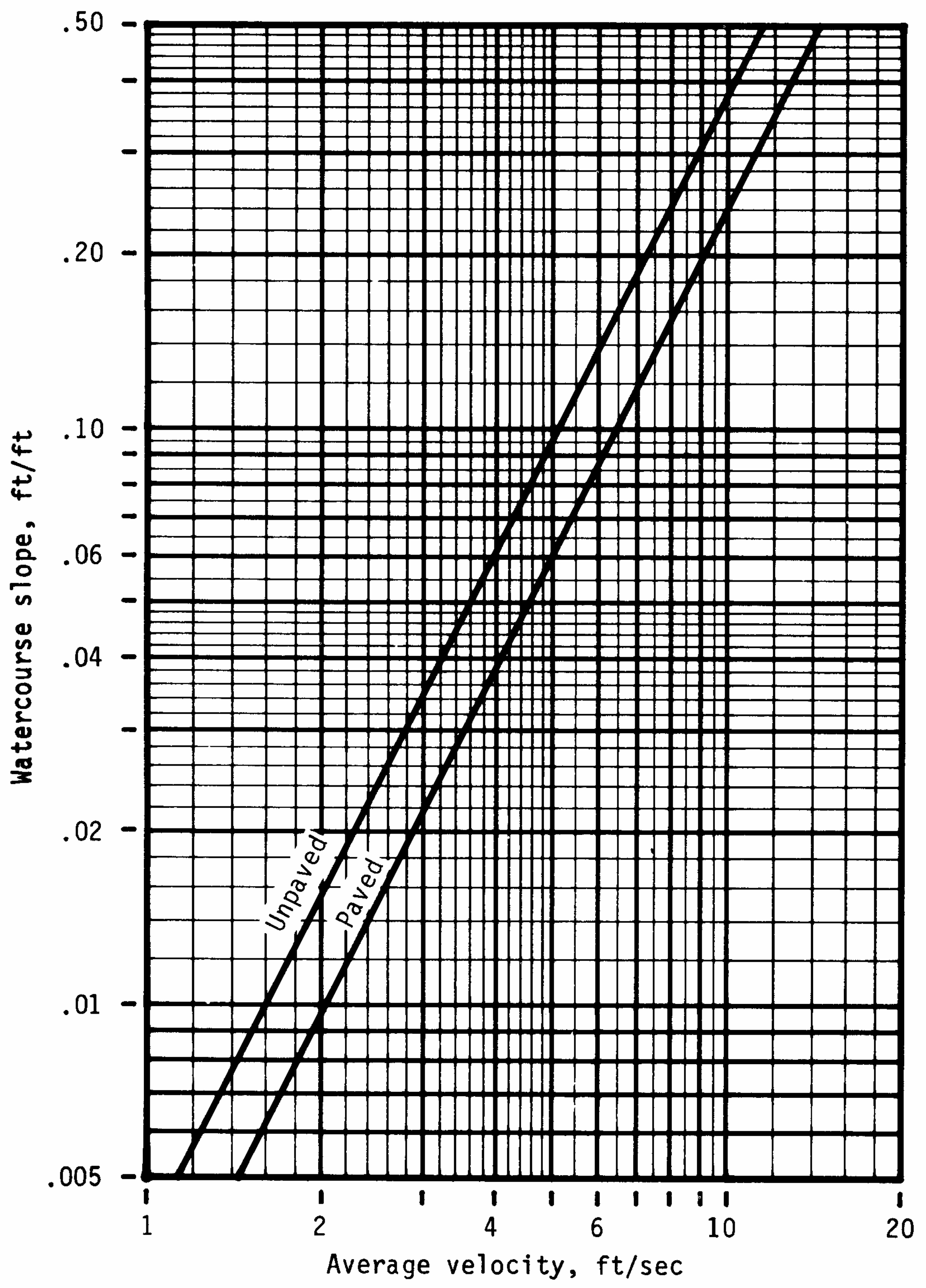
$$Tscf = \frac{(L)}{(V)3600}$$

Where:

$Tscf =$ travel time for shallow concentrated flow (hours)
$L =$ length of shallow concentrated flow path (ft)
$V =$ velocity (fps)

Figure 3.2 Average Velocity

(Source: NRCS TR-55 Urban Hydrology for Small Watersheds, Second Edition, June 1986)



At the point where a defined channel or pipe system begins, the flow velocity can be estimated by the Manning equation. For open channels the equation has the form:

Open Channel and Pipe Flow Equation

$$Toc = \frac{(L)}{(V)3600}$$

Where:

$Tof =$ travel time for open channel and pipe flow (hours)
$L =$ length of open channel and pipe flow path (ft)
$V =$ velocity (fps)

Flow in an Open Channel

$$V = \frac{(1.49)(R)^{0.667}(S)^{0.5}}{n}$$

Where:

$V =$ average flow velocity (fps)
$R =$ hydraulic radius (ft)
$S =$ channel slope (ft/ft)
$n =$ Manning's roughness coefficient

Hydraulic Radius Equation

$$R = \frac{A}{P}$$

Where:

$A =$ cross-sectional area (sq. ft)
$P =$ wetted perimeter (ft)

For pipe systems the flow velocity can be estimated by the Manning equation as well. Assuming the pipe is circular and is flowing just full, the equation simplifies to the form:

Flow in a Pipe

$$V = \frac{(0.59)(D)^{0.667}(S)^{0.5}}{n}$$

Where:

$D =$ pipe diameter (ft)
Other variables are as defined previously

Time of Concentration

The Tc is the total travel time and the sum of the sheet flow, shallow concentrated flow, open channel and pipe flow.

$$Tc = Tsf + Tscf + Tof$$

3.4 Hydrograph Generation

Hydrographs are a graph of the discharge at a particular location in a watershed. The hydrograph represents the discharge rate versus time for a given rainfall event and the volume underneath the hydrograph equals the runoff volume. The highest discharge rate on the hydrograph is the peak discharge.

The City of Rocky Mount prefers hydrographs be generated using standard computer software with the NRCS Unit Hydrograph methodology, appropriate design storms presented in the Peak Discharge section of this chapter with the NRCS Type II rainfall distribution. This method uses the NRCS dimensionless unit hydrograph, rainfall distribution, time of concentration converted to lag time and the runoff excess from the NRCS RCN method to generate a hydrograph. The unit hydrograph was developed by the NRCS based on multiple gage sites and represents the runoff pattern from a typical watershed. The designer shall not modify the peak rate factor unless it can be demonstrated through model calibration that this adjustment is warranted.

However, if the computer model limitations do not apply to the site condition then the following method, developed by Dr. Rooney Malcolm and presented in Elements of Urban Drainage Design, can be utilized. This method calculates the discharge at a given time (t) based on the step function given below and can be readily incorporated into a spreadsheet.

Step Function Equation (1)

For 0 < t < 1.25tp

$$Q = \frac{Qp}{2}\left(1 - \cos\frac{\Pi \times t}{tp}\right) \text{ in radians}$$

Step Function Equation (2)

For t > 1.25 tp

$$Q = 4.34Qpe^{-1.3\left(t/tp \right)} \quad \text{in radians}$$

<u>Time to Peak</u>

$$Tp = \frac{Volume}{(1.39)(Qp)(Tp)}$$

Where:

$Tp =$ time to peak of the hydrograph

$Volume =$ volume of runoff from design storm in cubic feet

= runoff from section 3.3.2.2 multiplied by drainage area

$Qp =$ peak discharge from section 3.1

3.5 Stream Routings

On large complex basins where multiple subbasins and hydrographs are required to adequately reflect the watersheds, hydrographs from individual subbasins need to be routed from one design point to the next taking into account the affects of the floodplain storage. This is referred to in this manual as stream routing.

The City of Rocky Mount requires the use of acceptable computer programs to generate, route and combine the hydrographs. Acceptable stream routing methods include Muskingum-Cunge, Modified Puls and ATT-KIN.

3.6 Impoundment Routings

Hydrograph routing is required when City standards require that some form of impoundment, either detention or retention, be used for new developments. The type and size of facility required will usually depend on the size of the proposed development, its impact on the downstream watercourse and whether or not downstream water quality is of primary concern. This type of routing is referred to as impoundment routing.

When an impoundment is required to control peak discharges, the impoundment outlet device controls the rate at which water can leave the impoundment. When the inflow discharge is greater than the outlet device discharge, the excess water is stored in the impoundment. As water is stored and released, the water surface in the impoundment increases until the hydrograph is completed. The quantity of water that must be detained or stored in order to adequately reduce the peak discharge is referred to as the Required Storage Volume. This is the volume that must be available in the facility without exceeding the maximum permissible release rate.

Although the required volume can only be found by routing the design storm hydrograph through the proposed facility, it is adequate to estimate the volume by subtracting the permissible outflow peak flow from the basin from the inflow peak flow for the critical storm duration.

<u>Approximate Storage Volume Equation</u>

$$S = (Qp - MPRR)(Tp)$$

Where:

$S =$	estimated storage volume (cf)
$Qp =$	peak inflow (cfs)
$MPRR =$	maximum permissible release rate (cfs)
$Tp =$	time to peak (seconds)

This is only a good initial estimate and must be verified by routing the design storm through the proposed facility.

3.6.1 Impoundment Outlet Devices

For purposes of this manual, a stormwater impoundment is a facility that is constructed to pond the stormwater during all storm events either temporarily or permanently. These impoundments typically have outlet devices that consist of a principal spillway and a separate emergency spillway. The principal spillway is the outlet device that controls the peak flows of the design storm events whereas the emergency spillway is designed to pass the 100-year or larger storm event in a manner which minimizes the impoundment failure. In special conditions the principal spillway will be designed to pass all of the anticipated storm events and a separate emergency spillway will not be provided.

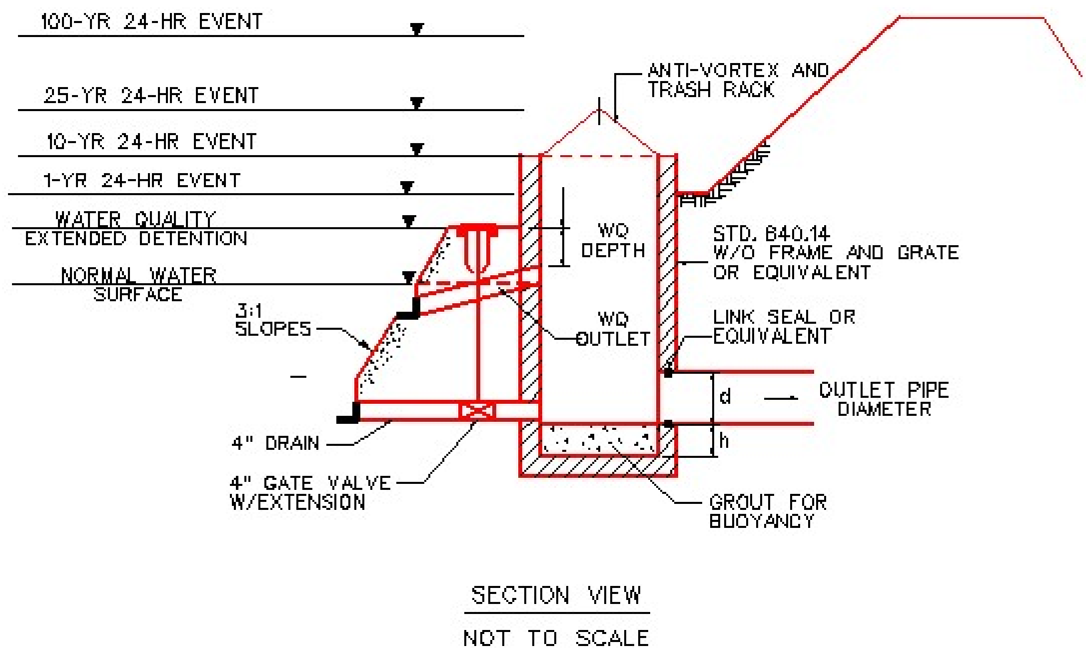
In order to route a hydrograph through an impoundment, an elevation-discharge rating curve is required to represent the principal and emergency spillway capacities. This section is a general description of some common outlet control devices and acceptable parameters used in impoundment facilities to generate the typical elevation-discharge rating curve.

Because controlling multiple design storms may be required, some rather imaginative outlet devices may result. To the extent possible, outlet devices should be kept simple. This may require an optimal design for one storm frequency and an over design for the other storm event.

Riser- Barrel Outlet
A riser-barrel outlet, as shown in Figure 3.3, is the most common type of principal spillway. The riser controls the water surface elevations in the impoundment and the barrel conveys the water through the impoundment structure. The riser-barrel can be a combination of several types of outlet devices. At different water surface elevations, different parameters will control the discharge. Small pipes or outlet holes in the riser will typically act as orifices, substantially limiting the amount of water that can be discharged through the barrel. These small openings are used to control the WQv and 1-year 24-hour peak discharge. When the water reaches the top of the riser, the water will spill over the edge, which acts as a weir. The length of the weir will control how much water passes over the edge for a given depth. This length will be set to control the 10-year, 24-hour peak discharge. As the water rises, more water passes over the edge of the riser and through the orifice openings and through the barrel. At some point the barrel starts to flow full and begins controlling how much water can pass through the impoundment. The barrel is typically sized to control the 25-year, 24-hour storm event. When the water surface rises above the 25-year elevation, the emergency spillway will then convey a majority of the larger events.

Each of the components summarized above work to control the peak discharges. The remaining part of this section presents the equations and acceptable parameters for orifices, weirs, and barrels used to develop the elevation discharge rating curve.

Figure 3.3 Cross Section of a Riser Barrel



3.6.1.1 Orifices

Small openings or pipes are the most common method of controlling the release of small amounts of water and are typically used to draw down the WQv in the required 48-hour period and also may be used to control the 1-year, 24 hour storm event. This is because as the depth of water increases over the orifice, the amount of water that passes through the orifice doesn't substantially change.

The discharge through an orifice can be described by an energy balance analysis. Assuming the upstream velocity is negligible (i.e. a reservoir) and the water surfaces both upstream and downstream are free surfaces, the energy balance can be simplified to what is referred to as the orifice equation.

<u>Orifice Equation</u>

$$Q = (Cd)(A)(2gh)^{0.5}$$

Where:

$Q =$	discharge (cfs)
$A =$	cross-sectional area of the orifice (sq ft.)
$g =$	gravitational acceleration
$h =$	driving head to the centroid of the orifice (where h > D/2)
$Cd =$	coefficient of discharge (usually 0.50-0.70)

The orifice equation is only appropriate when the headwater depth is above the top of the orifice (HW>D). When the flow through the orifice is lower than the top of the orifice, other forms of analysis such as a modified Weir Equation are required. For manual computations of discharge, the charts used for the inlet control for culverts may also be helpful. These charts are similar to the orifice equation but were developed using empirical data. In many cases they include discharges for depths as low as half the orifice diameter (HW/D = 0.5).

3.6.1.2 Weirs

Weirs control water by limiting the available length allowed for water to spill over. However, unlike the orifice, as the water rises the weir allows substantially more water to pass. Most weirs used in impoundments will fall into one of two categories; sharp-crested weirs such as flow over a standpipe, or broad-crested weirs such as emergency overflows in basins. Although considerable research has been conducted in the modeling of weirs, a simple expression can be applied to most weirs used in stormwater impoundments. The equation is usually expressed as:

Weir Equation

$$Q = (Cw)(L)(H)^{1.5}$$

Where:

$Q =$	discharge (cfs)
$Cw =$	weir coefficient
$L =$	length (ft)
$H =$	height of water above the crest of the weir (ft)

For sharp-crested weirs, Cw is usually taken to be about 3.33. For broad-crested weirs, 3.0 is generally used. Cw is not a true constant, but rather a function of flow depth and geometry. For horizontal weirs used in storm drainage, these values will usually suffice.

In some situations it is necessary to notch a riser and create multiple weirs. The notch is one weir and when the water rises and begins to spill over the top this becomes a second weir. Another situation where multiple weirs occur is when the emergency spillway can be defined as a weir. When multiple weirs are used, each of the weirs will have different depths of flows and therefore should be calculated separately and then added together.

As water rises above the top of the riser and more water passes over the edge, at some point the riser cross sectional opening area may become more restrictive than the weir length. Calculating the allowable discharge using the orifice equation and the allowable discharge using the weir equation at the same water surface elevation should be performed to determine which condition controls. It is strongly recommended not to allow the riser to control as an orifice because this indicates that there is a pocket of air trapped between the headwater created by the barrel and the water surface in the pond. This trapped air has the potential for creating destructive forces on the riser-barrel structure.

3.6.1.3 Barrel

The barrel is just like a culvert except that instead of a headwall, there is a riser section on the upstream side. The headwater elevation for a given discharge will be the height of the water inside the riser barrel. Once the headwater for the barrel is above the top of riser, the controlling flow out of the impoundment will be either the riser or the barrel.

The barrel sections are typically short sections of pipe and the capacity and headwater can be calculated using the orifice flow equation:

$$Q = (Cd)(A)(2gh)^{0.5}$$

Where:

$Q =$ discharge (cfs)
$A =$ cross-sectional area of the orifice (sq ft.)
$g =$ gravitational acceleration
$h =$ driving head to the centroid of the barrel opening (ft) (where h > D/2)
$Cd =$ coefficient of discharge (usually 0.60-0.70)

In some situations, the downstream tailwater conditions during higher discharges can control the barrel. Therefore, the designer should consider checking the barrel for outlet control as described in the culvert section.

3.6.1.4 Elevation-Discharge Rating Curve

The elevation-discharge rating curve for an impoundment defines the discharge that will be conveyed through the impoundment when the water surface is at a given elevation. This rating curve is used to route the hydrograph through the impoundment and determine the impoundment water surface elevation for the various storm events. For a conservative estimation of the impoundment water surface elevation, the elevation can be determined from the rating curve using the unrouted peak discharge.

The elevation-discharge rating curve should compile all the various components of the impoundment outlet devices and determine the controlling discharge at a given impoundment water surface elevation. A sample rating curve table is provided below.

Table 3.7 – Typical Elevation-Discharge Curve

Elevation	Low flow Orifice		Principal Weir				Qp total	Barrel			Emergency Spillway				Rating Curve	
			First Notch		Top of Riser			Inlet		Outlet		Top weir	Avg. weir			
Feet	h (feet)	Q (cfs)	h	Q	h	Q		h	Q	Q	h	length (ft.)	length (ft.)	Q	Elevation	Q (cfs)
89.125	0.0	0.0	0.0	0.0	0.0	0.0	0.0	NA	0.0	0.0	NA	NA	NA	0.0	**89.1**	**0.0**
89.500	0.4	0.1	0.0	0.0	0.0	0.0	0.1	NA	0.0	0.0	NA	NA	NA	0.0	**89.5**	**0.1**
90.000	0.9	0.2	0.0	0.0	0.0	0.0	0.2	NA	0.0	0.0	NA	NA	NA	0.0	**90.0**	**0.2**
90.500	1.4	0.3	0.0	0.0	0.0	0.0	0.3	NA	0.0	0.0	NA	NA	NA	0.0	**90.5**	**0.3**
91.000	1.9	0.3	0.0	0.0	0.0	0.0	0.3	4.3	48.9	51.7	NA	NA	NA	0.0	**91.0**	**0.3**
91.500	2.4	0.4	0.5	0.0	0.5	10.9	11.3	4.8	51.7	55.6	NA	NA	NA	0.0	**91.5**	**11.3**

92.000	2.9	0.4	1.0	0.0	1.0	30.9	31.3	5.3	54.3	59.2	0.0	10.0	10.0	0.0	**92.0**	**31.3**
92.500	3.4	0.4	1.5	0.0	1.5	56.8	57.2	5.8	56.8	62.7	0.5	13.0	11.5	10.8	**92.5**	**67.6**
93.000	3.9	0.5	2.0	0.0	2.0	87.4	87.9	6.3	59.3	65.9	1.0	16.0	13.0	34.5	**93.0**	**93.7**
93.500	4.4	0.5	2.5	0.0	2.5	122.1	122.6	6.8	61.6	69.0	1.5	19.0	14.5	70.6	**93.5**	**132.2**
94.000	4.9	0.5	3.0	0.0	3.0	160.6	161.1	7.3	63.8	72.0	2.0	22.0	16.0	119.9	**94.0**	**183.7**
94.500	5.4	0.5	3.5	0.0	3.5	202.3	202.9	7.8	65.9	74.8	2.5	25.0	17.5	183.3	**94.5**	**249.3**

3.6.2 Elevation Storage Rating Curve

The elevation storage rating curve defines the available storage at a given elevation. These curves can be generated by a computer program which typically uses the conic method to calculate the volume between two elevations with known surface areas. Another acceptable method is to use the average end area. In this method average surface area between two elevations multiplied by the difference in elevations determines the incremental volumes.

3.6.3 Storage Indication Routing

The storage indication method is used by most standard computer software applications to route the hydrograph through an impoundment. This method uses the inflow hydrograph, elevation-discharge and elevation-storage rating curves to determine the outflow hydrograph and elevation within the impoundment. The City of Rocky Mount prefers hydrographs be routed through impoundments using standard computer software.

3.6.4 Chain Saw Routing

For simple impoundments and conditions when the computer model limitations apply to the site the Chain Saw Routing developed by Dr. Rooney Malcolm and presented in Elements of Urban Drainage Design can be utilized. This method uses the hydrograph defined by the step function described earlier.

For routing a storm by the Chain Saw Routing method in a spreadsheet or by hand, it is necessary to formulate an expression for the stage-storage relationship. For routing by hand, a plot of the relationship is adequate. For computer application, the relationship can usually be expressed by a power curve. The simplest ways to determine the volume is to planimeter (or digitize) a topographic map of the basin and calculate the storage using the average end areas method.

The resulting plot of stage vs. storage may be used for routing by hand or a "best fit" equation of the points may be used. The best fit is usually of the form:

$$Storage = (K)(Stage)^b$$

Where:

$Storage =$ accumulated volume at the stage (ft^3)

$Stage =$ Depth or elevation in the impoundment

$K =$ constant for the best fit line. Typically determined using the spread sheet function

$b =$ constant exponent of the best fit line. Typically determined using the spreadsheet function

Therefore, to calculate the stage for the associated storage volume:

$$Stage = (K)(Storage)^{1/b}$$

The components required for the Chain Saw Routing method are similar to those of storage-indication method. The method is an incremental tabular application of the same differential equation but simplified to the form:

$$Si = (Ii - Oi)(Ti - Tj)$$

Where:

$Si =$ incremental change in storage at time i (sec)

$Ii =$ inflow at time i (cfs), using the step function defined in Section 3.3.

$Oi =$ outflow at time i (cfs)

$(Ti - Tj) =$ time step (sec)

The Chain Saw Routing method may not be as intuitively satisfying as other methods since the outflow at any time is based on the storage volume prior to that time step. The method does however lend itself to spreadsheet application and with sufficiently short time steps provides reasonable results. Here again the method is best explained by example.

3.7 Street and Gutters

Effective drainage of street and roadway pavements is essential to the maintenance of the roadway service level and to traffic safety. Water on the pavement can interrupt traffic flow, reduce skid resistance, increase potential for hydroplaning, and limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface.

This section presents design guidance for gutter flow hydraulics originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual. For more complex gutter sections, the design should refer to the manual for appropriate methodologies.

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = \left(\frac{0.56}{n}\right)(Sx)^{5/3}(S)^{1/2}(T)^{8/3}$$

Where:

$Q =$ gutter flow rate, cfs
$Sx =$ pavement cross slope, ft/ft
$n =$ Manning's roughness coefficient
$S =$ longitudinal slope, ft/ft
$T =$ width of flow or spread, ft

Table 3.8 Manning’s n Values for Gutter Sections
City of Rocky Mount, NC

Type of Gutter or Pavement	Range of Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002
Note: Estimates are by the Federal Highway Administration Source: USDOT, FHWA, HDS-3 (1961).	

3.8 Catch Basins and Drop Inlets

The City of Rocky Mount requires the completion Catch Basin Design Data Sheet provided in Appendix B. This section presents the equations and charts necessary to complete the sheet and demonstrate that the catch basins are properly located and designed according to Chapter 1 requirements.

The capacity of a catch basin or drop inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small. A parallel-bar grate is the most efficient type of gutter inlet. However, when crossbars are added for bicycle safety, the efficiency is greatly reduced.

The ratio of frontal flow to total gutter flow, Eo, for straight cross slope is expressed by the following equation:

$$Eo = Qw/Q = 1-\left(1-\frac{W}{T}\right)^{2.67}$$

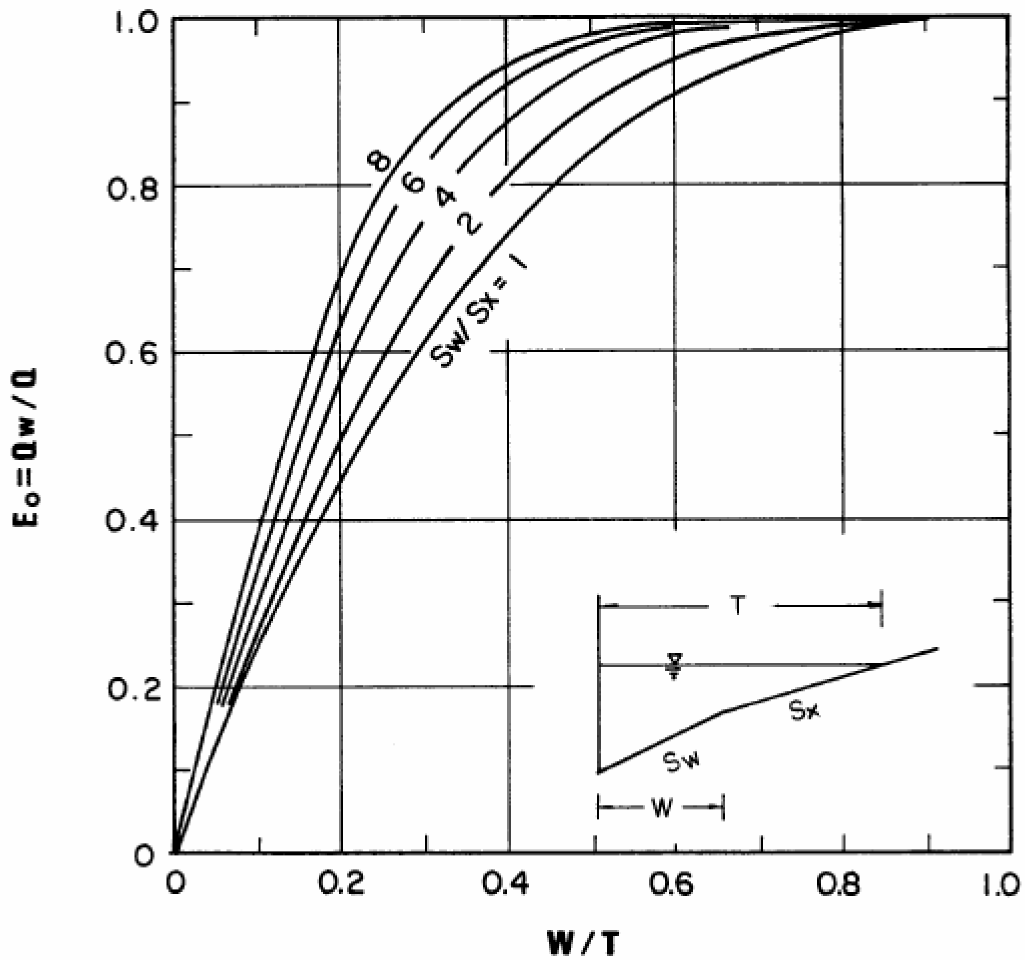
Where:

$Eo =$	ratio of the frontal flow to total gutter flow
$Q =$	total gutter flow, cfs
$Qw =$	flow in width W, cfs
$W =$	width of depressed gutter or grate, ft
$T =$	total spread of water in the gutter, ft

Figure 3.4 can be used to determine Eo.

Figure 3.4 Ratio of Frontal Flow to Total Gutter Flow

(Source: AASHTO Model Drainage Manual, 1991)



The ratio of frontal flow intercepted to total frontal flow, Rf, is expressed by the following equation:

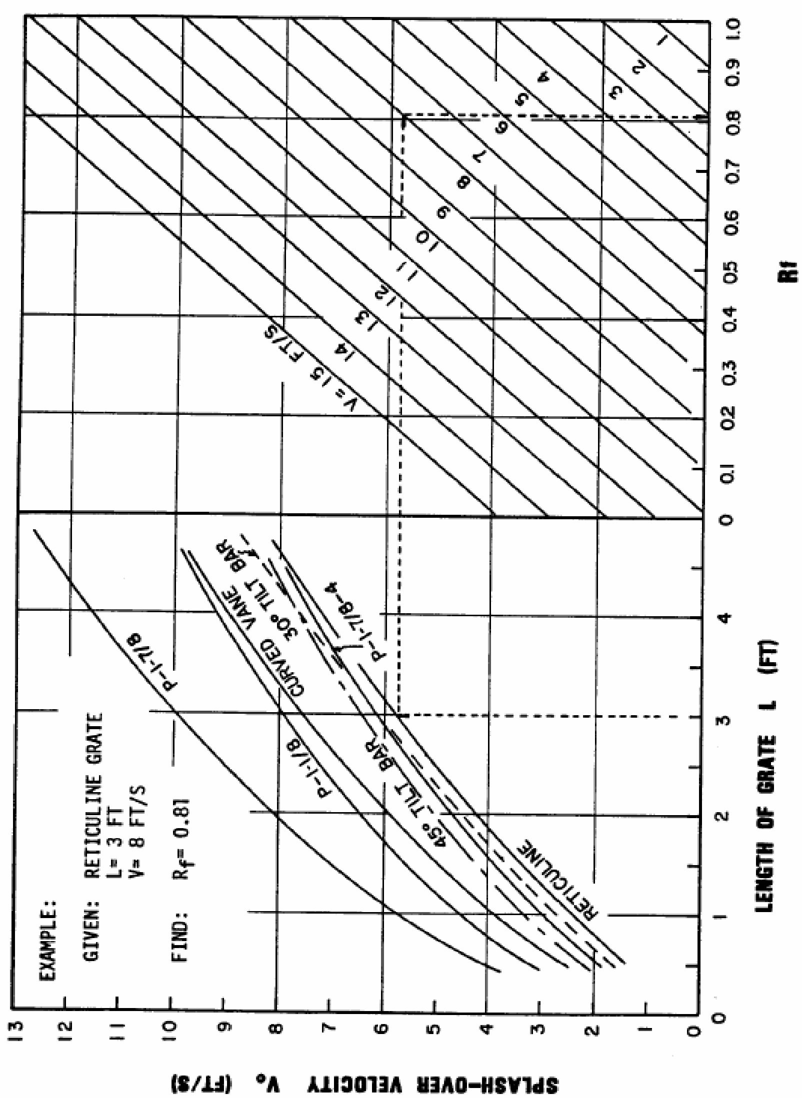
$$Rf = 1 - (0.09)(V - Vo)$$

Where:

Rf =	ratio of frontal flow intercepted by the catch basin grate
V =	velocity of flow in the gutter, ft/s
Vo=	gutter velocity where splash-over first occurs, ft/s (from Figure 3.5)

Figure 3.5 Grate Inlet Frontal Flow Interception Efficiency

(Source: HEC-12, 1984)



The ratio of side flow intercepted to total side flow, Rs, or side flow interception efficiency, is expressed by:

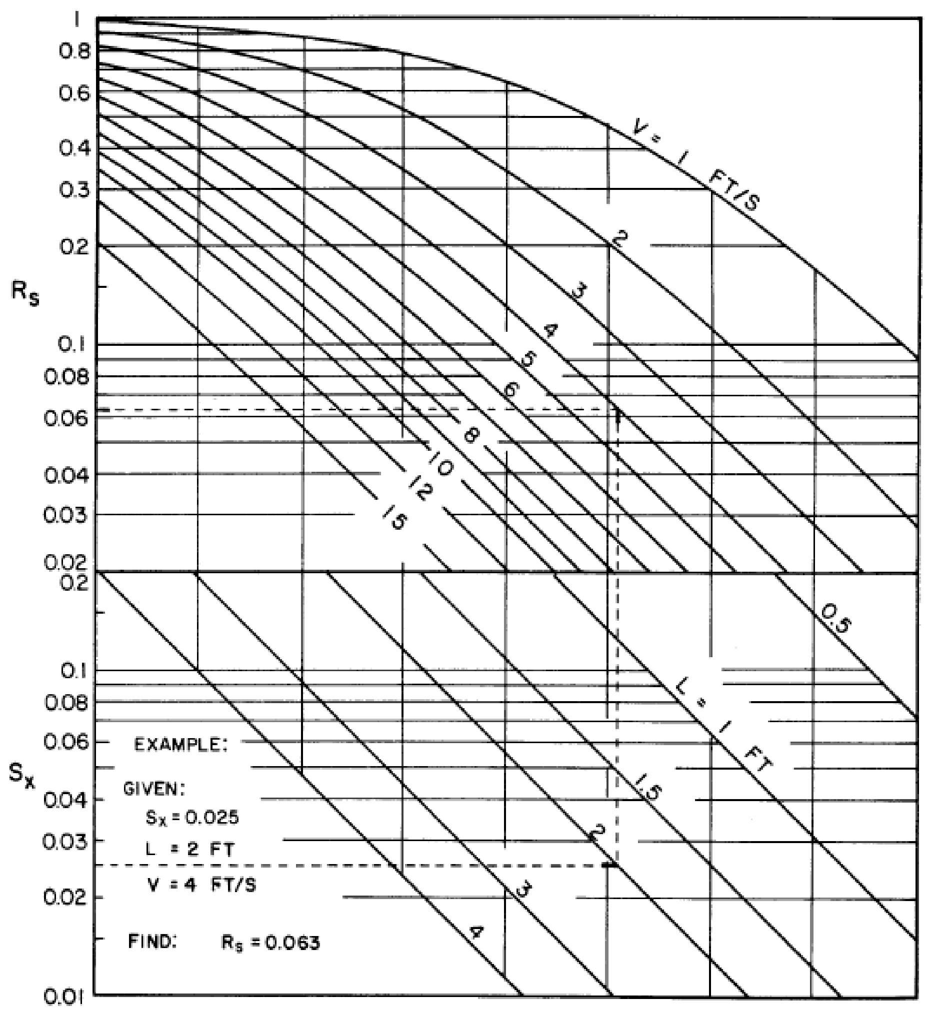
$$Rs = \frac{1}{\left[1 + \left(\frac{(0.15)(V)^{1.8}}{(Sx)(L)^{2.3}}\right)\right]}$$

Where:

Rs = ratio of the side flow intercepted by the catch basin grate
L = length of the grate, ft

Figure 3.6 provides a solution to the equation.

Figure 3.6 Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)



The efficiency, E, of a grate is expressed as:

$$E = (Rf)(Eo) + (Rs)(1 - Eo)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Qi = (E)(Q) = Q[(Rf)(Eo) + (Rs)(1 - Eo)]$$

3.8.1 Grate Inlets in Sag

A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.4 feet above the top of grate and when depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 feet and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of grate inlets operating as a weir is:

$$Qi = (C)(P)(d)^{1.5}$$

Where:

P = perimeter of grate excluding bar widths and the side against the curb, ft
C = 3.0
d = depth of water above grate, ft

The capacity of grate inlets operating as an orifice is:

$$Qi = (C)(A)(2gd)^{0.5}$$

Where:

C = 0.67 orifice coefficient
A = clear opening area of the grate, ft2
g = 32.2 ft/s2
d = depth of water above grate, ft

Both calculations should be computed at given depths. The lowest Qi will control the depth of ponding above the grate.

3.8.2 Curb Inlets on Grade

Following is a discussion of the procedures for the design of curb inlets on grade. Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is determined using Figure 3.7.

The efficiency of curb opening on grade with inlets shorter than the length required for total interception is determined using Figure 3.7.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, Se, in the following equation:

$$Se = Sx + (S'w)(Eo)$$

Where:

Eo = ratio of flow in the depressed section to total gutter flow
$S'w$= cross slope of gutter measured from the cross slope of the pavement, Sx

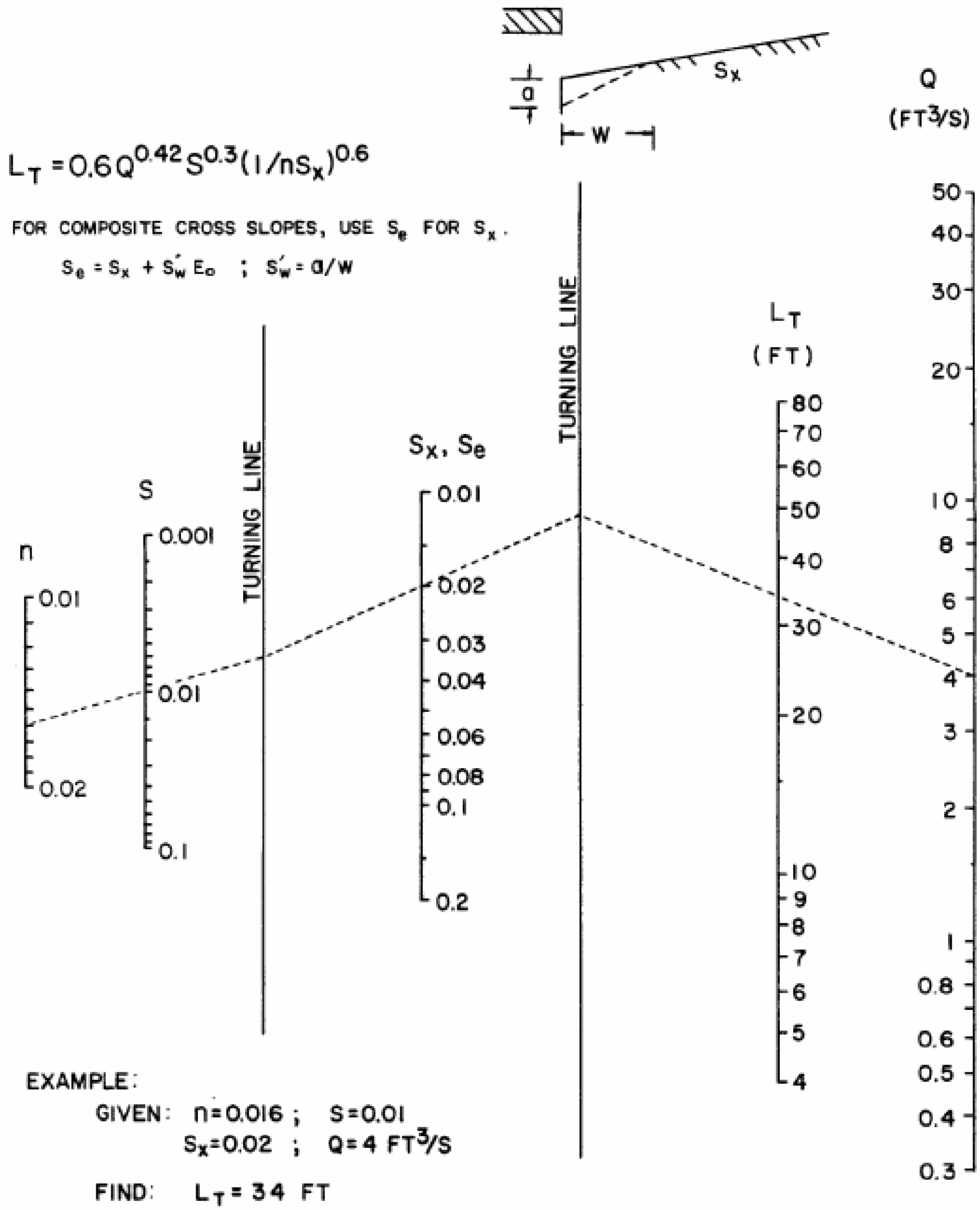
$$S'w = \frac{a}{(W)(12)}$$

Where:

a= gutter depression, in
W= width of depressed gutter, ft

It is apparent from examination of Figure 3.7 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Figure 3.7 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)



3.8.3 Curb Inlets in Sump

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

3.8.4 Drop Inlets

Drop Inlets are typically located in natural or graded sump locations. The capacity and depth of water shall be calculated using the weir and orifice equations identified in the Grate Inlets In Sags section above.

3.9 Storm Drainage Pipes

Storm Drainage Pipes are located between the catch basins and drop inlets and ultimately convey the water to a receiving channel or stream. The City of Rocky Mount requires the Storm Drain Design Computations Sheet, provided in Appendix B, be completed for all pipe systems to demonstrate the storm drainage pipe has the capacity to convey the design discharge assuming normal depth. The City of Rocky Mount also requires the Hydraulic Grade Line Calculation Sheet, provided in Appendix B, be completed for all pipe systems to demonstrate the storm drainage system is adequate when considering all of the tailwater conditions and energy losses. This section presents the equations and allowable parameters to complete these calculation sheets.

3.9.1 Storm Drain Calculation Sheet

The storm drain calculation sheet requires the designer to calculate the design peak discharge at each catch basin, drop inlet and junction box using the rational equation and accumulated time of concentration. For each storm drainage system, the sheet should be completed beginning at the furthest upstream inlet. When a storm drainage system includes multiple branches, then each branch should be treated as a separate system.

The pipe diameter shall be designed to handle the design discharge assuming normal depth and full flow capacity of the pipe. The most widely used formula for determining the hydraulic capacity of storm drainage pipes for gravity and pressure flows is the Manning's Formula, expressed by the following equation:

$$V = \frac{(1.486)(R)^{2/3}(S)^{1/2}}{n}$$

Where:

V = mean velocity of flow, ft/s
R= the hydraulic radius, ft – defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)
S = the slope of hydraulic grade line, ft/ft
n = Manning's roughness coefficient, see Table 3.9

In terms of discharge, the above formula becomes:

$$Q = \frac{(1.486)(A)(R)^{2/3}(S)^{1/2}}{n}$$

Where:

Q = rate of flow, cfs
A = cross sectional area of flow, ft2

For pipes flowing full, the above equations become:

$$V = \frac{(0.590)(D)^{2/3}(S)^{1/2}}{n}$$

$$Q = \frac{(0.463)(D)^{8/3}(S)^{1/2}}{n}$$

Where:

D = diameter of pipe, ft

Table 3.9 Manning's n Values

Type of Conduit	Wall & Joint Description	Manning's n
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
Corrugated Metal Pipes and Boxes Annular Corrugations	2 2/3- by ½-inch corrugations	0.024
	6- by 1-inch corrugations	0.025
	5- by 1-inch corrugations	0.026
	3- by 1-inch corrugations	0.028
	6-by 2-inch structural plate	0.035
	9-by 2-1/2 inch structural plate	0.035
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3-by ½-inch corrugated 24-inch plate width	0.012
Spiral Rib Metal Pipe	3/4 by 3/4 in recesses at 12 inch spacing, good joints	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.015
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011

Source: HDS No. 5, 1985

When sizing pipe diameters, the following general rules shall apply:

- The pipe diameter shall not be reduced downstream regardless of the hydraulic capacity.
- Minimum pipe diameters shall be as identified in Chapter 1.
- Minimum Time of Concentration shall be 5 minutes.

3.9.2 Hydraulic Grade Line Calculation Sheet

The storm drain calculation sheets assume that the flow in the storm drain system is not affected by downstream conditions, such as tailwater or hydraulic losses through the structures. These conditions are considered in the Hydraulic Grade Line Calculation Sheet. The hydraulic grade line begins at the outlet of the storm drainage system and progresses upstream to the first inlet. When a system consists of multiple branches, separate hydraulic grade lines are calculated for each branch beginning at the common structure and progressing upstream to the branch first inlet. The hydraulic grade line computes the potential water surface elevations, under design conditions in the various inlets, catch basins, manholes, junction boxes, etc. If the potential water surface elevation in the structures does not satisfy the design criteria identified in Chapter 1, then adjustments to the storm drain design will be required. These adjustments may include increasing the pipe diameter or relocating the system.

The hydraulic grade line calculations begin at the outlet of the system from a known water surface elevation, which is typically the depth of flow in the receiving channel or water surface elevation within a structure. The hydraulic losses to the next upstream structure are then added to the known water surface elevation to determine the potential water surface elevation. These hydraulic losses are the friction loss of the pipe and the junction losses within the upstream structure. These losses are summarized below.

3.9.2.1 Friction Loss

The hydraulic loss caused by the roughness of the pipe material.

$$Hf = (Sf)(L)$$

Where:

$Hf =$ friction loss (ft.)
$L =$ length of the pipe between structures (ft.)
$Sf =$ friction slope

Where:

$$Sf = \frac{Q^2}{K}$$

Where:

Q = the design peak discharge (cfs)
K = pipe conveyance

Where:

$$K = \frac{1.486}{n}(A)(R)^{2/3}$$

3.9.3 Contraction Loss

The hydraulic loss caused by the contraction of flow within the structure to the outlet pipe opening.

$$Ho = (0.25)\left(\frac{Vo^2}{2g}\right)$$

Where:

Ho = contraction loss (ft.)
Vo = velocity in the outlet pipe assuming full fow (ft/s)
g = 32.2 (ft/s$^{2)}$

3.9.4 Expansion Loss

The hydraulic loss is caused by expansion of the flow within the structure. When multiple pipes enter the structure, the system with the largest momentum will be used to determine the expansion loss. Pipe with inflows of less than 10% of the mainline outflow can be neglected.

$$He = (0.35)\left(\frac{Vi^2}{2g}\right)$$

Where:

He = expansion loss (ft.)
Vi = velocity in the inlet pipe with the largest momentum M (ft/s)
g = 32.2 (ft/s$^{2)}$

Where:

$$M = p(Qi)(Vi)$$

Where:

Qi = discharge for the influent pipe (ft.)
Vi = velocity in the inlet pipe with the largest momentum M (ft/s)
p = density of water. This can be ignored for the purpose of comparing inflows.

3.9.5 Bend Loss

The hydraulic loss caused by the change in direction within the structure.

$$Hb = (K)\left(\frac{Vi^2}{2g}\right)$$

Where:

Hb = bend loss (ft.)
Vi = velocity in the inlet pipe (ft/s)
g = 32.2 (ft/s$^{2)}$

K = bend loss coefficient based on the bend angle. Provided on the Hydraulic Grade Line Calculation Sheet.

3.10 Culverts

A *culvert* is a short, closed (covered) conduit that conveys stormwater runoff under an embankment, usually a roadway. The primary purpose of a culvert is to convey surface water through the embankment and protect the embankment from failure, protect traffic during the design storms events and protect the downstream channel from the contraction of the floodplain flows. Culverts shall be designed per the FHWA Hydraulic Design Series No. 5 – Hydraulic Design of Highway Culverts (HDS-5). This section provides a brief summary of culvert designs and supporting calculations for typical culvert design. The designer is expected to be aware of when the typical design assumptions and methods are not appropriate and utilize the appropriate design methodology presented in HDS-5.

The City of Rocky Mount requires the Culvert Design Form provided in Appendix B be completed for each culvert. The City of Rocky encourages the use of computer programs such as HEC-RAS to perform the calculations because this software readily computes the tailwater conditions and roadway overtopping depths. If the designer uses an acceptable computer program to perform the calculations, the designer shall complete the form using the results from the computer program and submit the computer program with the supporting calculations. If the designer performs the calculations by hand, then the designer shall submit all of the supporting calculations including normal depth, weir flow, nomographs, friction losses, etc.

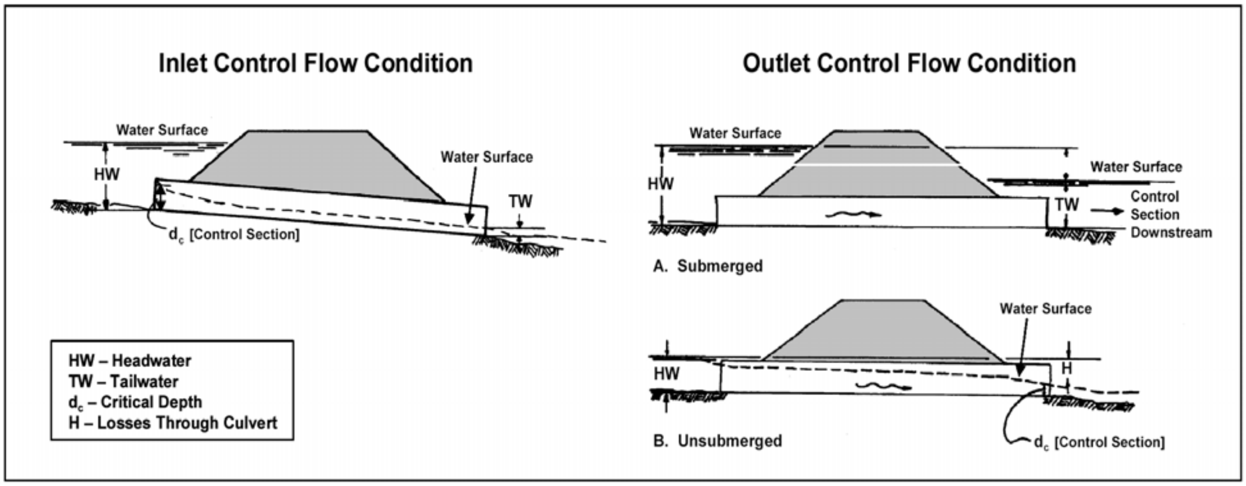
3.10.1 Types of Flow Control

There are two types of flow conditions for culverts that are based upon the location of the control section and the critical flow depth. Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs.

Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

Figure 3.8 Culvert Flow Conditions
(Adapted from: HDS-%, 1985)



3.10.2 Inlet Control

HDS –5 includes numerous nomographs for the inlet condition for various pipe sizes and entrance conditions. The City of Rocky Mount prefers these nomographs be used to determine the inlet control HW/D column in the Culvert Design Form. The two most common inlet forms are provided below. If other types of inlets are used, the designer shall include a copy of the inlet nomograph with the Culvert Design Form.

Figure 3.9 Headwater Depth for Concrete Pipe Culvert with Inlet Control

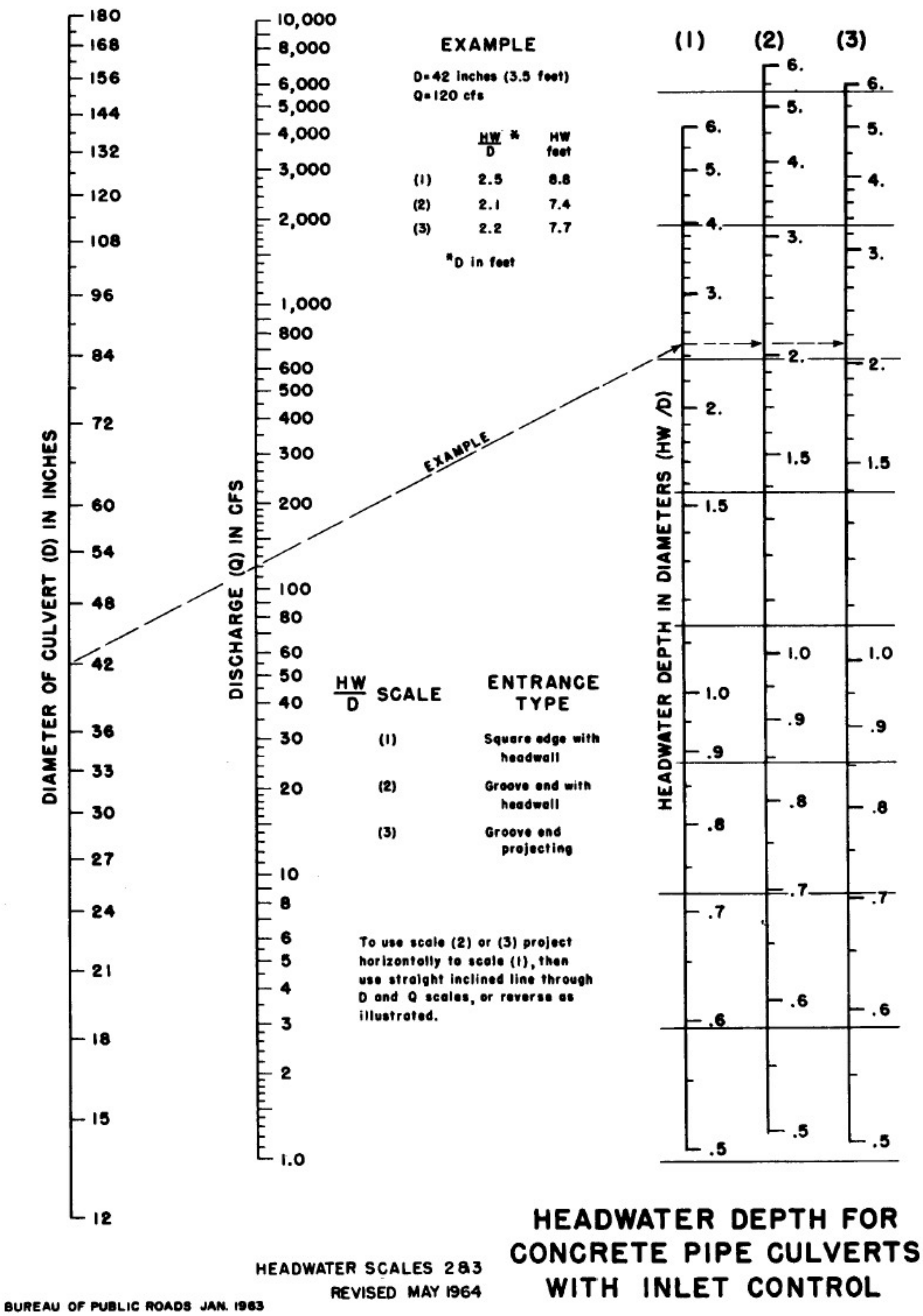
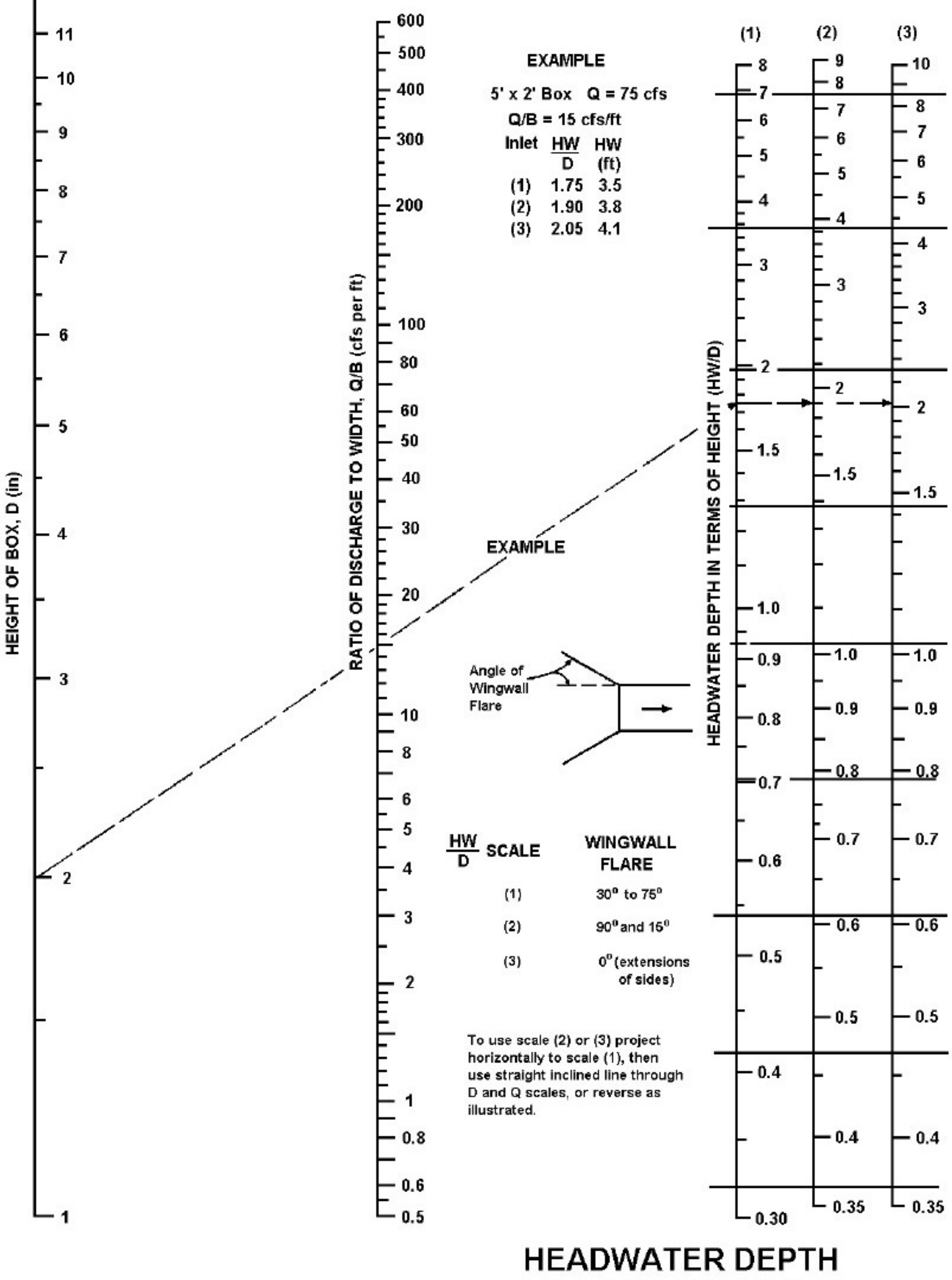


Figure 3.10 Headwater Depth for Concrete Box Culvert with Inlet Control



HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

3.10.3 Outlet Control

HDS –5 includes numerous nomographs for the full flow condition for various pipe sizes and entrance conditions. The City of Rocky Mount does not want these nomographs to be used and requires the outlet control conditions be calculated using the acceptable entrance losses and Manning's "n" values. Outlet Control requires the calculation of the tailwater depth and the energy losses associated with the culvert. These losses are similar to the storm drainage pipe losses required to establish the hydraulic grade line. The tailwater depth should reflect the expected water surface elevation in the downstream channel and floodplain. The tailwater depth can be calculated using the methods identified in the open channel section or taken from an existing flood study. The designer shall determine if the tailwater depth will be affected by downstream conditions and perform the necessary calculations to reflect these conditions. The basic equations for the outlet control are provided below:

$$Hw = H + ho - (L)(S)$$

Where:

$Hw =$	headwater depth above the upstream invert elevation (ft)
$H =$	energy loss in feet through the culvert (ft)
$ho =$	tailwater depth above the downstream invert elevation (ft)
$L =$	culvert length (ft)
$S =$	culvert slope (ft/ft)

Where:

$$H = \left[1 + ke + \frac{(29)(n)^2(L)}{(R)^{2/3}}\right]\left(\frac{V^2}{2g}\right)$$

Where:

$1 =$	exit loss coefficient
$ke=$	entrance loss coefficient from the table

When the tailwater depth is below the top of the culvert, then the tailwater depth should be compared to the following equation and the higher depth used to calculate outlet control:

$$ho = dc + \frac{D}{2}$$

Where:

$dc =$	critical depth in the culvert (ft)
$D =$	inside depth of the culvert (ft)

Table 3.9 Inlet Coefficients

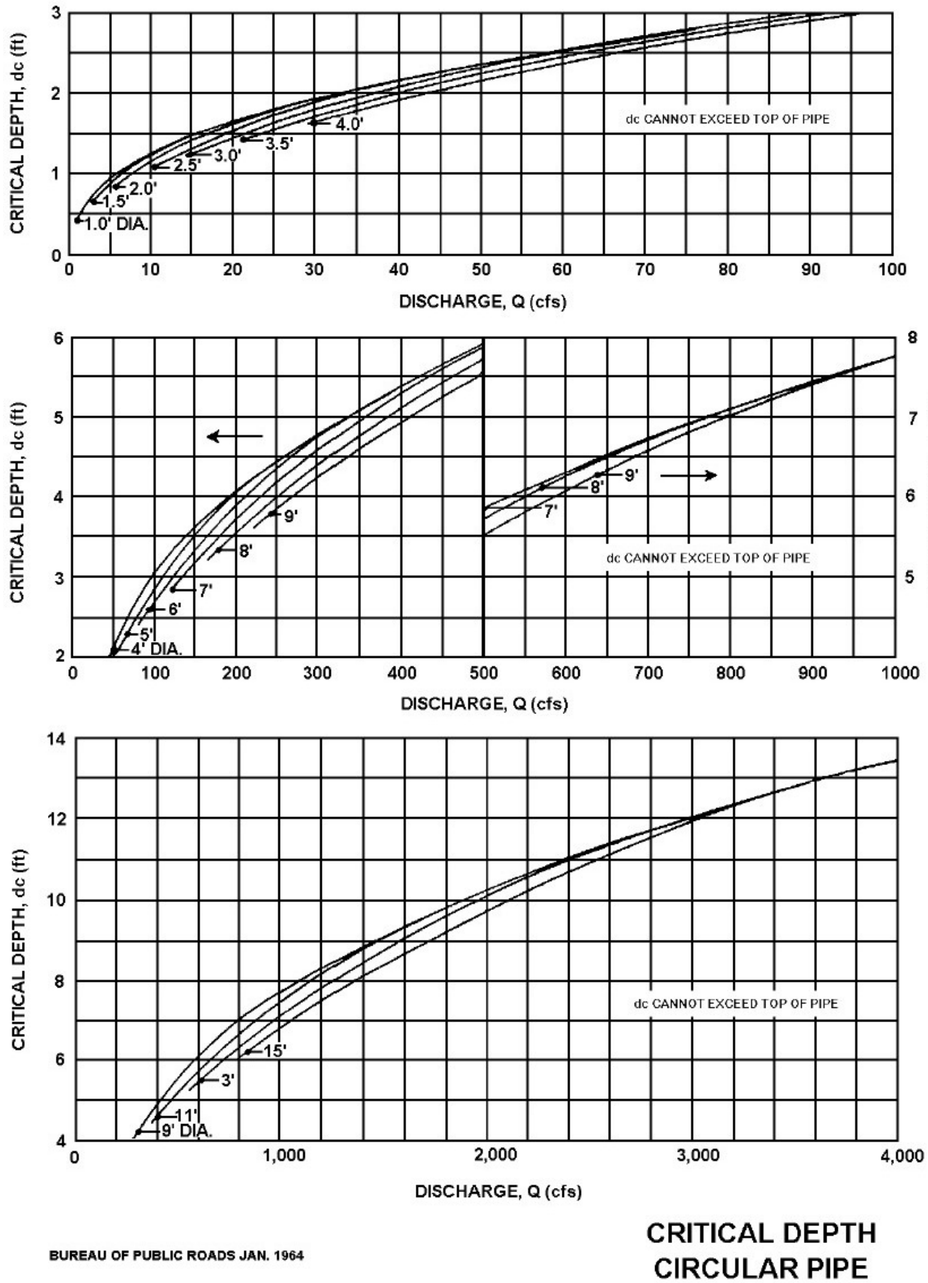
Type of Structure and Design of Entrance	Coefficient Ke
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7^o or 45^o bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal[1]	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7 $_o$ or 45 $_o$ bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or beveled edges on 3 sides	0.2
Wingwalls at 30^o to 75^o to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10^o to 25^o to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

1 Although laboratory tests have not been completed on Ke values for High-Density Polyethylene (HDPE) pipes, the Ke values for corrugated metal pipes are recommended for HDPE pipes.

* Note: End Section conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control.

Source: HDS No. 5, 1985

Figure 3.11 Critical Depth for Circular Pipes



3.10.4 Roadway Overtopping

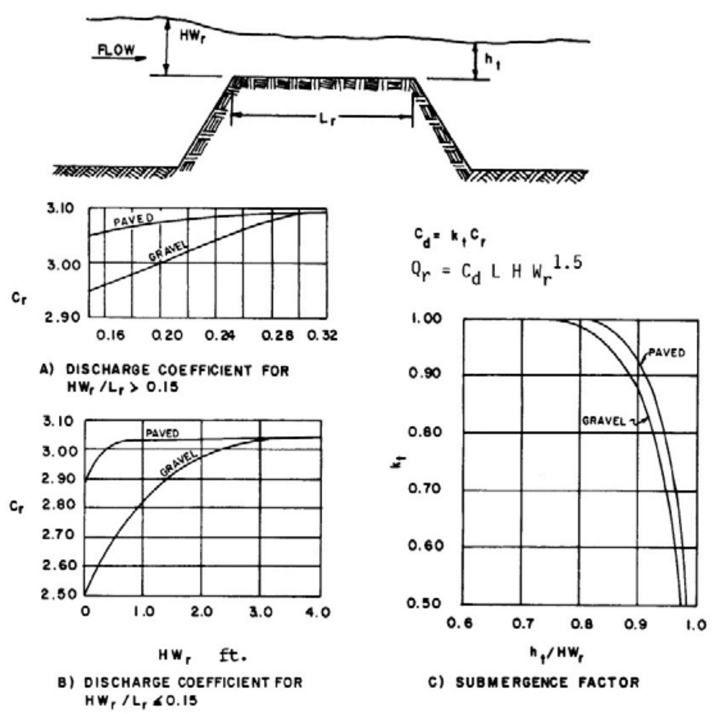
To complete the Culvert Design Form, the roadway overtopping should be analyzed for the larger events. The elevation over the road for a given discharge will be when the discharge over the road plus the discharge through the culvert add up to the given discharge. This can be done through a trial and error process of assuming a headwater elevation and calculating the two discharges or can be determine by generating an elevation-discharge curve similar to the curve used for outlet devices. This curve can then be used graphically or through interpretation to determine the headwater elevation. Discharge over the road can be determined using the weir flow equation.

$$Qr = (Cd)(L)(H)^{1.5}$$

Where:

Qr = overtopping flow rate (cfs)
Cd = overtopping discharge coefficient
L = length of roadway (ft)
H = depth of water above the road (ft)

Figure 3.10 Discharge Coefficients for Roadway Overtopping
(Source HDS No. 5, 1985)



3.11 Open Channels

Vegetative and rip rap open channels shall be designed in accordance with the procedures identified in the North Carolina Erosion and Sediment Control Planning and Design Manual. For irregular shaped channels, the City of Rocky Mount prefers the use of standard computer programs such as HEC-RAS.

City of Rocky Mount, North Carolina
Stormwater Design Manual

Appendices

October 2006

Rocky Mount Engineering Department
One Government Plaza
Rocky Mount, NC 27802

APPENDIX A: ACRONYMS AND DEFINITIONS

Bioretention - An engineered means of managing stormwater runoff, using chemical, biological and physical processes via a natural, terrestrial-based community of plants, microbes and soil. Bioretention provides two important functions: (1) water quantity (flood) controls; and (2) improves water quality through removal of pollutants and nutrients associated with runoff.

Catch Basin - A structure located within a curb and gutter section that allows water to enter into the storm drainage system. The catch basin has an opening in the curb and may or may not have an opening in the gutter section covered by a grate.

Design Storm - A theoretical storm of a given frequency that will produce a simulated runoff peak and volume having the same return frequency. Thus, a 100-year design storm should produce a 100-yr runoff and volume.

Drop Inlet - A vertical inlet to a buried culvert or storm drainage pipe with a flat grate inlet.

DWQ – North Carolina Division of Water Quality.

Easement - A right to use the land of another for a specific purpose, such as for a right-of-way or utilities.

Forebay - Excavated settling basin or a section separated by a low weir at the head of the primary impoundment. The forebay serves as a repository for a large portion of sediment and facilitates draining and excavating the basin.

Grass Swales - A series of vegetated, open channels that are designed to treat and attenuate stormwater runoff for a specified water quality volume. As stormwater runoff flows through the channels, it is treated through filtering by vegetation in the channel, filtering through a subsoil matrix, and/or infiltration into the underlying soils.

Grate Inlet - Depressions or cavities in the pavement or ground that are covered by a steel grate and designed to collect and covey stormwater. Grate inlets can be found in parking lots, roadway medians and along town streets.

Illicit Connection - Any discharge to a municipal separate storm sewer that is not composed entirely of stormwater (some discharges may be authorized by an NPDES permit) and discharges resulting from fire fighting activities.

Impervious Surface - Surfaces providing negligible infiltration such as pavement, buildings, recreation facilities(e.g. tennis courts, etc.), and covered driveways. This will include porous pavement, gravel roads, parking areas and precast concrete, but does not include wooden slatted decks or the water surface area of swimming pools.

Junction Box – Where stormwater drain lines join or intersect, a box installed to accommodate changes in flow direction, pipe diameter and elevation.

Level Spreader - A device used to spread out stormwater runoff uniformly over the ground surface as sheetflow (i.e., not through channels). The purpose of a level spreader is to prevent concentrated, erosive flows from occurring and to enhance infiltration.

NCDENR – North Carolina Department of Environment and Natural Resources.

New development - shall be defined as to include the following: 1) any activity that disturbs greater than one acre of land in order to establish, expand or modify a single family or duplex residential development or a recreational facility; 2) any activity that disturbs greater than one-half an acre of land in order to establish, expand or modify a multifamily residential development or a commercial, industrial or institutional facility; and 3) does NOT include agriculture, mining or forestry activities. Land disturbance is defined as grubbing, stump removal and/or grading.

NPDES – National Pollutant Discharge Elimination System.

Open Channel - A long, narrow, open trench dug into the ground usually at the side of a road or field, which is used especially for supplying or removing water, or for dividing land.

Plug Flow - Fluid particles pass through the basin and are discharged in the same sequence in which they enter. The particles remain in the system €or a time equal to the theoretical detention time. This type of flow is especially appropriate for basins with high length-to-width ratios (Metcalf and Eddy, Inc., 1979).

Record Drawings - The primary outlet is often constructed of a rised barrel assembly and

Principal Spillway - The primary outlet is often constructed of a rised barrel assembly and provides flood protection (ie. for the 10-yr. storm) or reduces the frequency of the operation of the emergency spillway.

Riparian Buffer - an area of trees, usually accompanied by shrubs and other vegetation, that is adjacent to a body of water and which is managed to maintain the integrity of stream channels and shorelines, to reduce the impact of upland sources of pollution by trapping, filtering, and converting sediments, nutrients, and other chemicals, and to supply food, cover, and thermal protection to fish and other wildlife.

Spillway - A sluiceway or passage for excess water in a reservoir, to prevent too much pressure on the dam.

Storm Drainage System – Natural or man-made individual structures, designed in combination, with the express purpose of conveying stormwater to larger water bodies.

Storm Event - A rainfall event that produces more than 0.1 inch of precipitation and is separated from the previous storm event by at least 72 hours of dry weather.

Stormwater Wetlands - Manmade structure that is regularly saturated by surface or groundwater and subsequently characterized by a prevalence of vegetation that is adapted for life in saturated soil conditions.

Travel Lane - A strip of roadway intended to accommodate the forward movement of a single line of vehicles. A solid or broken line is used to separate individual traffic lanes from each other and from the shoulder of the road.

Vegetated Filter Strips - Strips of vegetation separating a water body from a land use that could act as a non-point pollution source. Vegetated buffers are variable in width and can range in function from vegetated filter strips to wetlands or riparian areas.

Wet Detention Pond – Detention basins are excavated areas or natural depressions designed to detain stormwater runoff. These structures detain or impede flow by storing runoff and releasing the stored volume at a reduced rate.

APPENDIX B: DESIGN FORMS AND CHECKLISTS

Design Checklist: Wet Detention Pond

Project: ______________________

Form Completed By: ______________ Date: ______________

Form Checked By: ______________ Date: ______________

PRELIMINARY HYDROLOGIC CALCULATIONS

1. Water Quality Volume	
Runoff Coefficient, R_v	R_v = ________
WQv	WQ_v = ________ acre-ft
Average release rate over 48-hour period	Rate = ________ cfs
2. 1-Year Detention Requirements	
Existing Condition 1-year discharge	1-year = ________ cfs
3. Flood Detention Requirements	
Existing Condition 10-year discharge	10-year = ________ cfs
Existing Condition 25-year discharge	25-year = ________ cfs

POND DESIGN

1. Surface Area of Normal Pool	
Drainage Area	DA = ________ acre(s)
Percent Impervious	Impervious = ________ %
Depth	Depth = ________ ft
Surface Area of Normal Pool	SA = ________ acre
2 Sediment Forebay	
Volume	Vol_{pre} = ________ cu. ft.
3. Pond Design Characteristics	
Normal Pool Elevation	Elevation = ________ ft
Normal Pool Volume	Volume = ________ cu. ft.
Top of Embankment	Top = ________ ft
WQv Elevation	WQv Elev. = ________ ft
WQv Volume	WQv Vol. = ________ cu. ft.
1-year peak elevation	________ ft
1-year outlet discharge	________ cfs
10-year peak elevation	________ ft
10-year outlet discharge	________ cfs
25-year peak elevation	________ ft
25-year outlet discharge	________ cfs
100-year peak elevation	________ ft
100-year outlet discharge	________ cfs
4. Elevation-Discharge Rating Curve	Separate Sheet
5. Elevation-Storage Rating Curve	Separate Sheet
6. Hydrograph Routing	Separate Sheet

Notes: ____

Design Checklist: Stormwater Wetland

Project: ______________________

Form Completed By: ______________ Date: ______________

Form Checked By: ______________ Date: ______________

PRELIMINARY HYDROLOGIC CALCULATIONS

1 Surface Area Required for Wetland	
% imperviousness of drainage area	% = ______
Drainage Area	DA = ______ acres
SA/DA from Table	SA/DA = ______
Surface Area Required for Wetland	SA = ______ sq. ft.
2. Water Quality Volume	
Runoff Coefficient, R_v	R_v = ______
WQv	WQ_v = ______ acre-ft
Average Release Rate Over 48-hour Period	Rate = ______ cfs
3. 1-Year Detention Requirements	
Existing Condition 1-year discharge	1-year = ______ cfs
4. Flood Detention Requirements	
Existing Condition 10-year discharge	10-year = ______ cfs
Existing Condition 25-year discharge	25-year = ______ cfs

STORMWATER WETLAND DESIGN

1. Wetland Design	
Micropool Area	$Area_{mp}$ = ______ sq. ft., % = ______
Sediment Forebay Area	$Area_{mp}$ = ______ sq. ft., % = ______
Pool/Deepwater Wetland Zone (1.5 - 6 feet deep)	$Area_{dw}$ = ______ sq. ft., % = ______
Low Marsh Wetland Zone (6-12 inches deep)	$Area_{low}$ = ______ sq. ft., % = ______
High Marsh Wetland Zone (0-6 inches deep)	$Area_{high}$ = ______ sq. ft, % = ______
	Σ = 100.00%
2. Sediment Forebay	
Volume	Vol_{pre} = ______ cu. ft.
Drainage Area	DA = ______ acres
Impervious Area	Imperv. = ______ acres
3. Wetland Final Design Characteristics	
Normal Pool Elevation	______ ft
Top of Embankment	______ ft
WQv Elevation	______ ft
WQv Volume	______ cu. ft.
1-year peak elevation	______ ft
1-year outlet discharge	______ cfs
10-year peak elevation	______ ft
10-year outlet discharge	______ cfs
25-year peak elevation	______ ft
25-year outlet discharge	______ cfs
100-year peak elevation	______ ft
100-year outlet discharge	______ cfs
4. Elevation-Discharge Rating Curve	Separate Sheet
5. Elevation-Storage Rating Curve	Separate Sheet
6. Hydrograph Routing	Separate Sheet

Notes: ______________________

Design Checklist: Riparian Buffer

Project: ____________________

Form Completed By: __________ Date: __________

Form Checked By: __________ Date: __________

1. Computed WQv	
WQ_v	WQ_v = ______ acre-ft
Q_p	Q_p = ______ cfs
2. Drainage Area	A = ______ acre(s)
3. Diversion structure	
Low Flow Orifice - Orifice Equation	A = ______ ft^2
Orifice Diameter	D = ______ in
4 Level Spreader	______
Entrance Width	Enter W = ______ ft
End Width	Exit W = ______ ft
Depth	Depth = ______ ft

Notes: ____________________

Design Checklist: Grassed Swales

Project: ______________________________

Form Completed By: ______________ Date: ______________

Form Checked By: ______________ Date: ______________

1. Computed WQv	WQ_v = ________ acre-ft
2. Drainage Area	A = ________ acre(s)
	________
3. Peak Runoff	
Peak Runoff, 10-year event	$Q_{p\text{-}10}$ = ________ acre-ft
Velocity, 10-year event	$V_{p\text{-}10}$ = ________ ft/s
4. Swale Dimensions	
Length	Length = ________ ft
Width	Width = ________ ft
Longitudinal Slope	S = ________ ft/ft
Side Slopes	Side Slopes = ________ (h:v)

Notes: ______________________________

Design Checklist: Water Quality Swale

Project: ____________________

Form Completed By: ____________ Date: ____________

Form Checked By: ____________ Date: ____________

1. Computed WQv	
WQv	WQ_v = ________ acre-ft
2. Computed Q_{p-10}	
Q_{p-10}	Q_{p-10} = ________ acre-ft
V_{p-10}	V_{p-10} = ________ ft/s
3. Sediment Forebay Volume	
Volume	Vol_{pre} = ________ acre-ft
4. Swale Dimensions	
Length	Length = ________ ft
Width	Width = ________ ft
Side Slopes	Side Slopes = ________ (h:v)
Area	Area = ________ ft^2
Longitudinal Slope	S = ________ ft/ft
5. Check Dams	
Depth	Depth = ________ ft
Spacing Distance	Distance = ________ ft
Number of Check Dams	No. = ________
6. Filter	
Area	A_F = ________ ft^2
Depth	Depth = ________ in
Draw Down Time	Time = ________ hr
Permeability	Fc = ________ in/hr

Notes: ____________________

Design Checklist: Vegetated Filter Strip With Level Spreader

Project: ______________________________

Form Completed By: ______________ Date: ______________

Form Checked By: ______________ Date: ______________

1. Computed WQv	
WQ_v	WQ_v = ________ acre-ft
Q_p	Q_p = ________ cfs
2. Drainage Area	
Area	A = ________ acre(s)
3. Diversion structure	
Low Flow Orifice - Orifice Equation	A = ________ ft^2
Orifice Diameter	diam. = ________ in
4. Filter Strip	
Length	L_f = ________ ft
Width	W = ________ ft
Slope	S = ________ ft/ft
Level Spreader Width	W_f = ________ ft
5. Level Spreader	
Length	L = ________ ft
Depth	D = ________ ft

Notes: ______________________________

Design Checklist: Bioretention (Rain Gardens)

Project: ______________________

Form Completed By: ______________ Date: ______________

Form Checked By: ______________ Date: ______________

BIORETENTION DESIGN

1. Compute WQ_v volume requirements	WQ_v = ________	acre-ft
2. Drainage Area	A = ________	acre
3. Bioretention Filter	A_f = ________	ft^2
Filter Depth	Depth = ________	in
Filter Length	Length = ________	ft
Filter Width	Width = ________	ft
4. Engineered Soil		
Depth of Soil	Depth = ________	in
Clay Content	Clay = ________	%
Infiltration Rate	Rate = ________	in/hr
pH	pH = ________	
Organic Content (%)	Organics = ________	%
Soluble Salts	Salts = ________	ppm
Phosphorus Index	P = ________	
5 Conveyance to Bioretention Facility	online or offline	(circle one)
6 Depth of Pond for _____-year Event		
Ponding Depth Above Filter	Depth = ________	in
Design Year Event	Design = ________	- year event
7 Sediment Forebay Volume (if required)	Vol_{pre} = ________	acre-ft

Notes: ______________________

Design Checklist: Sand Filter

Project: ______________________________

Form Completed By: ______________ Date: ______________

Form Checked By: ______________ Date: ______________

1. Computed WQv
   WQ_v — WQ_v = ________ acre-ft

2. Drainage Area
   Area — A = ________ acre(s)

3. Diversion Structure
   Low Flow Orifice - Orifice Equation — A = ________ ft^2
   Orifice Diameter — diam. = ________ in

4. Filtration Area
   Area - Darcy's Law — A_f = ________ ft^2
   Filter Length — L = ________ ft
   Filter Width — W = ________ ft
   Sand Depth — D = ________ in

5. Sediment Forebay
   Area - Camp-Hazen Equation — A_s = ________ ft^2
   Length — L = ________ ft

Notes: ______________________________

Hydraulic Grade Line Computations

Designed by ____________________
Checked by ____________________
QA /QC Verification ____________________

Project Name ____

Location ____

Catch Basin, Drop Inlet or Junction No.	Outlet W.S. Elevation	Pipe diameter (inches)	Discharge (cfs)	Pipe Length (feet)	Head Losses (feet)					Inlet W.S. Elevation	RIM Elevation	Remarks
					H(f)	H(c)	H(e)	H(b)	Total			

BEND LOSS COEFFICENTS

Degrees	K	Degrees	K	Degrees	K
0	0.00	30	0.28	70	0.61
15	0.10	40	0.38	80	0.66
20	0.16	50	0.47	90	0.70
25	0.22	60	0.55		

Storm Drain Design Computations

Designed by ______________________
Checked by ______________________
QA /QC Verification ______________________

Project Name ______________________________

Location ______________________________

STRUCTURE		DISCHARGE								Pipe Design								
From	To	Cum. D.A.	Sum (C)(D.A.)	Pipe length	Inlet time	Flow time	Design time	Rainfall intensity	Discharge	Inlet elevation	Outlet elevation	Slope	Diameter	Material	Pipe capacity	Flow velocity	Upstream box depth	Remarks
		(Acres)		(feet)	[min.]	[min.]	[min.]	(in/hr)	(cfs)			(ft/ft)	(inches)		(cfs)	[fps]	(feet)	

Culvert Design Form

Date

Project Name:.

Designed By

Checked By

Culvert Location

Skew

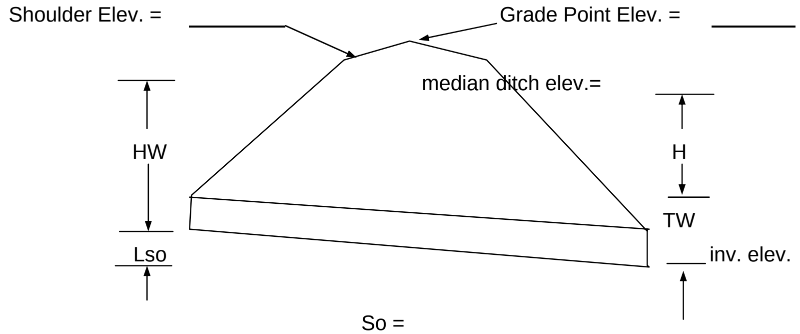
Size/Type Pipe

Type Entrance

Direction of Flow

Hydrologic Method

H.W. Control Elev.



So =

L=

SUMMARY DATA

Drainage Area =

Design Frequency =

Design Discharge =

Design H.W. Elev =

Q100 Discharge =

Q100 H.W. Elev. =

Depth above road =

Size (ft)		No.	"n"	Type	Freq.	Total Q	Nat.	Allow.	T.W.	INLET CONTROL		OUTLET CONTROL							H.W.	Vo	Comments
D	B			Pipe	(yrs)	(cfs)	H.W.	H.W.	Elev.	HW/D	H.W.	Ke	dc	dc+D/2	ho	H	Lso	H.W.	Elev.	(fps)	

SUMMARY AND RECOMMENDATIONS:

APPENDIX C: REFERENCES

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