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:

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

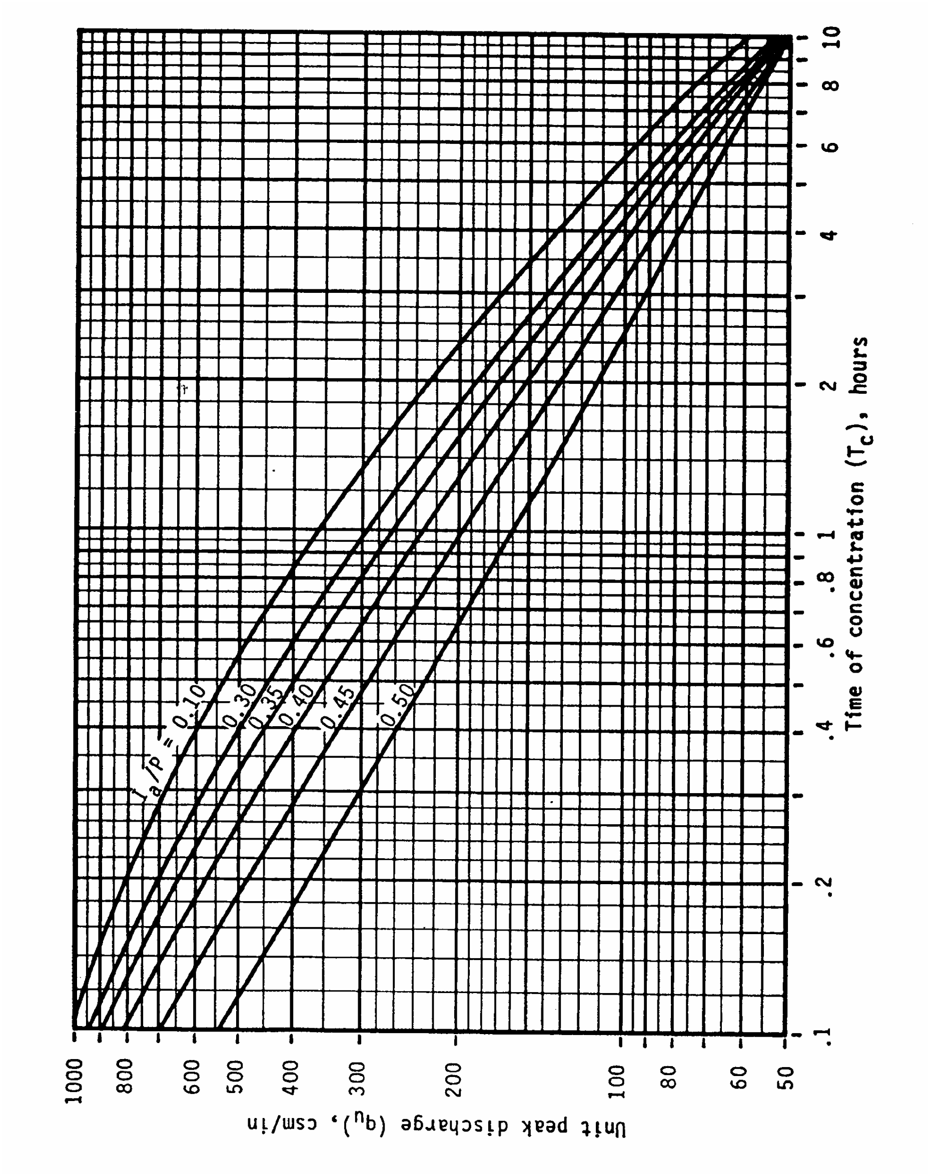
Where:

$Qp =$ 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

Sheet Flow Equation

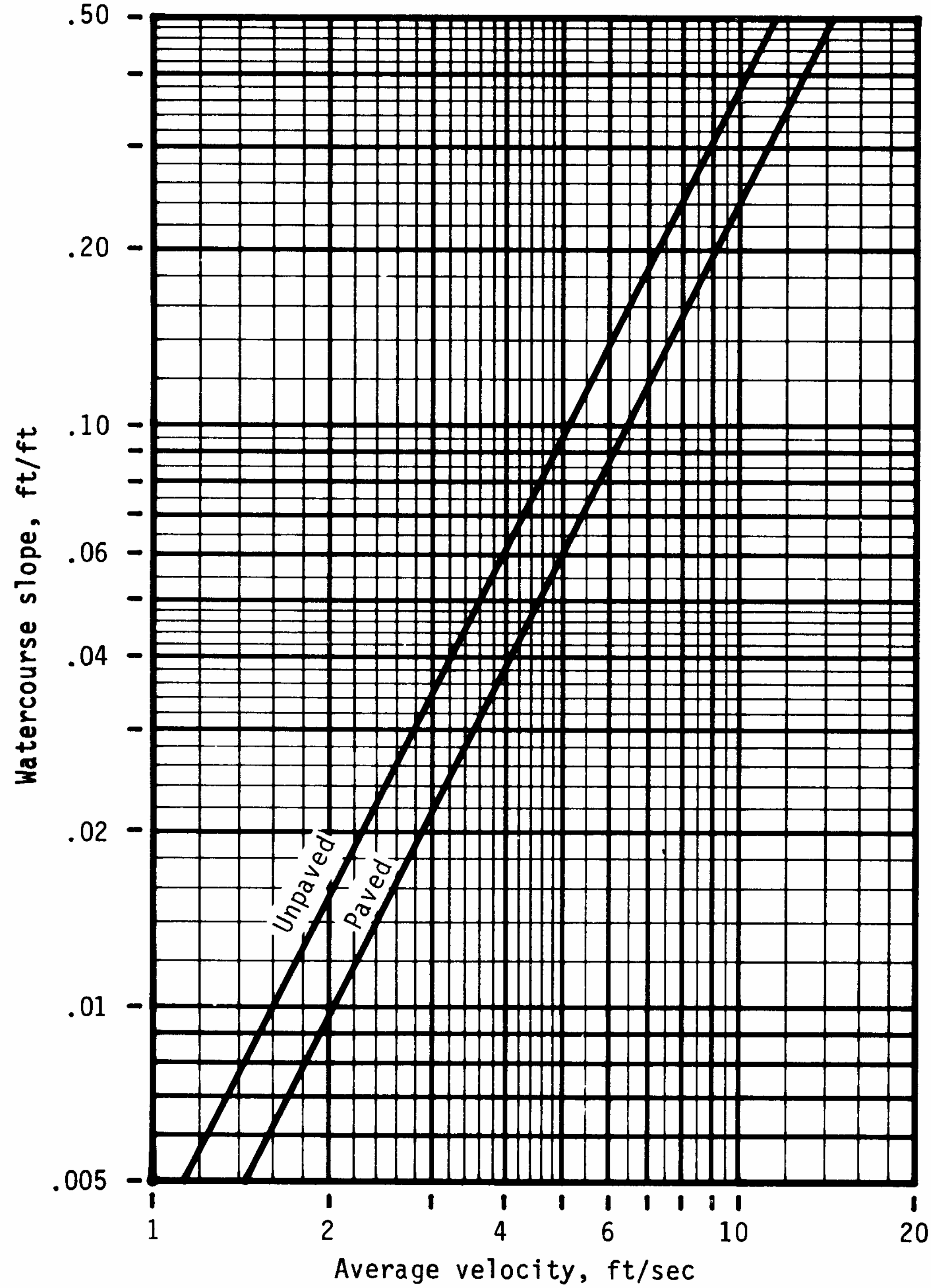
Shallow concentrated flow travel time is best estimated by calculating the average flow velocity from the figure on the following page. The travel time is estimated as the average flow velocity multiplied by the flow length.

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

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

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

Time to Peak

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

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.

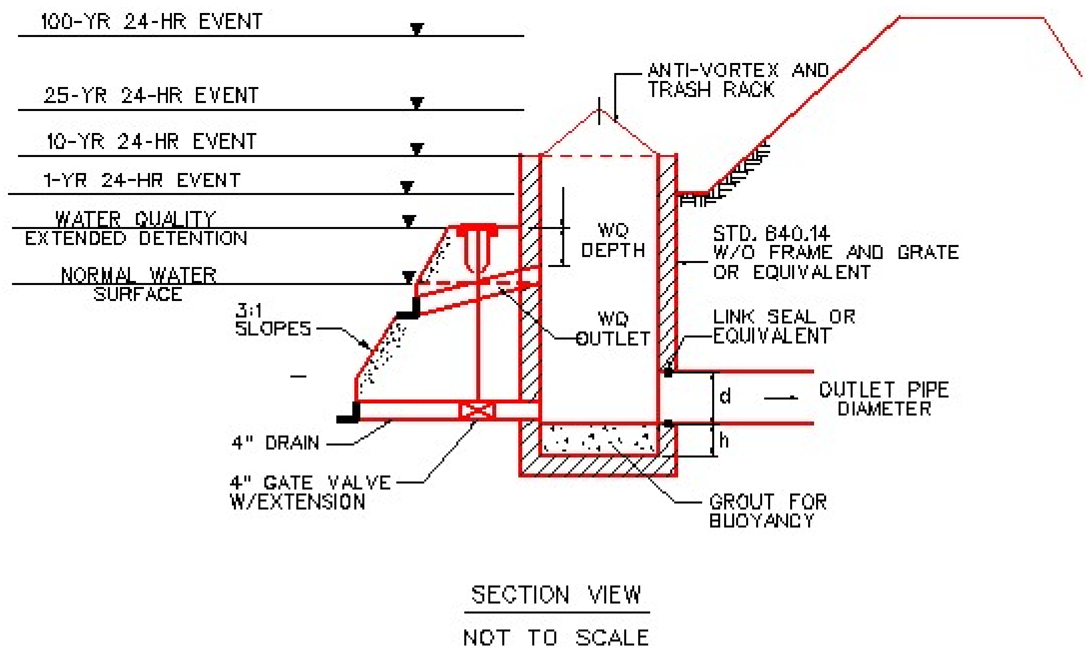
Because controlling multiple design storms may be required, some rather imaginative outlet devices may result. To the extent possible, outlet devices should be kept simple. This may require an optimal design for one storm frequency and an over design for the other storm event.

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:

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

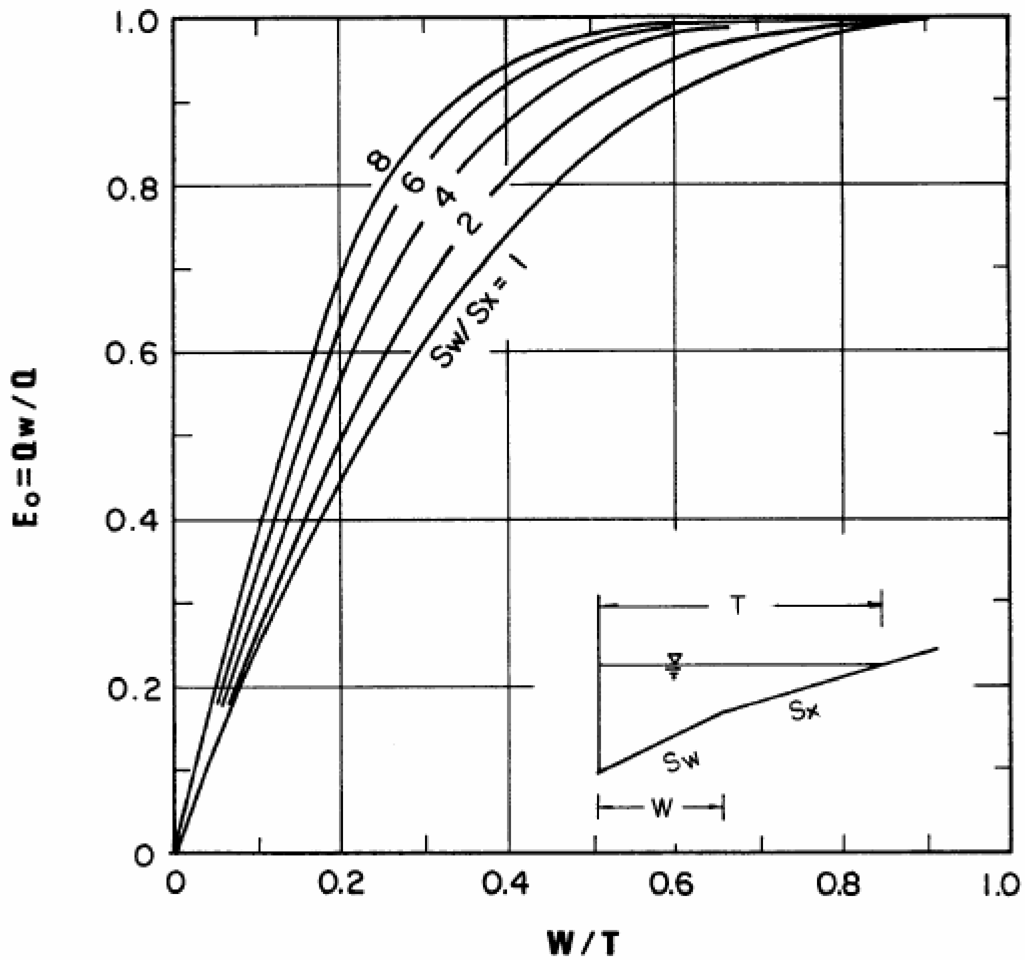
Where:

$Eo =$	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:

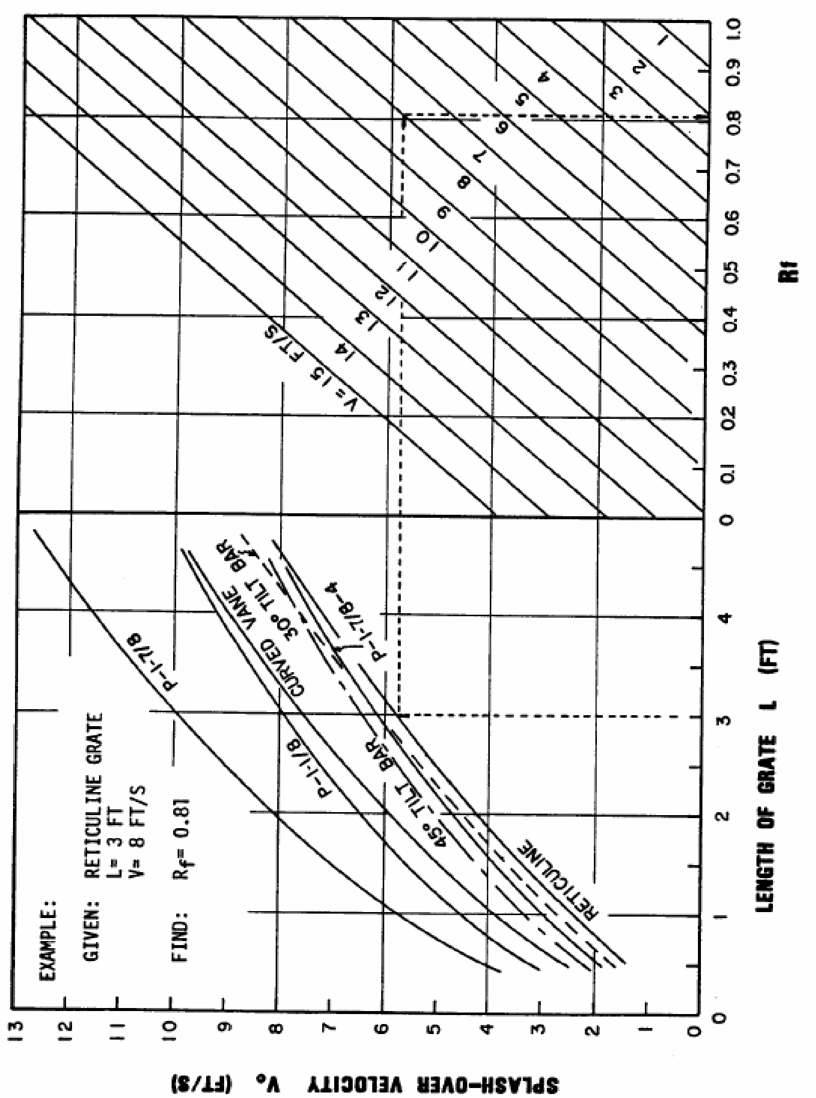
$$Rf = 1 - (0.09)(V - Vo)$$

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:

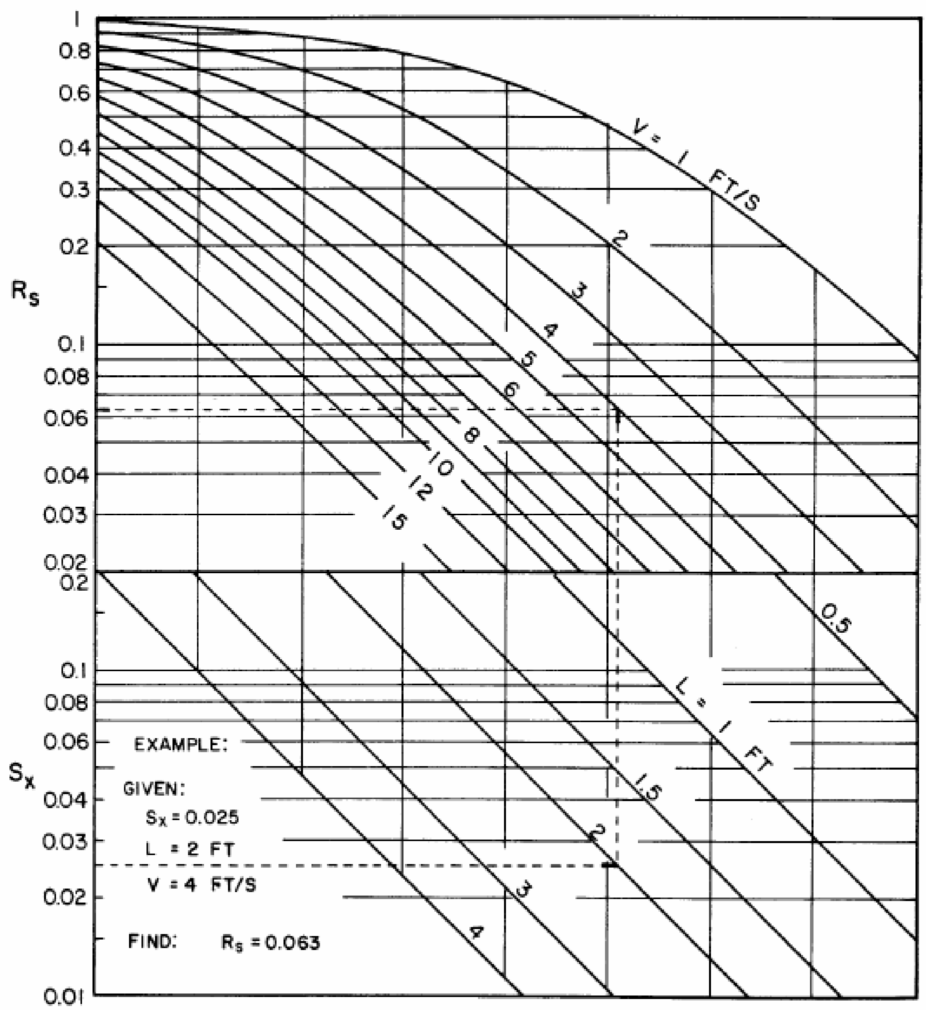
$$Rs = \frac{1}{\left[1 + \left(\frac{(0.15)(V)^{1.8}}{(Sx)(L)^{2.3}}\right)\right]}$$

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

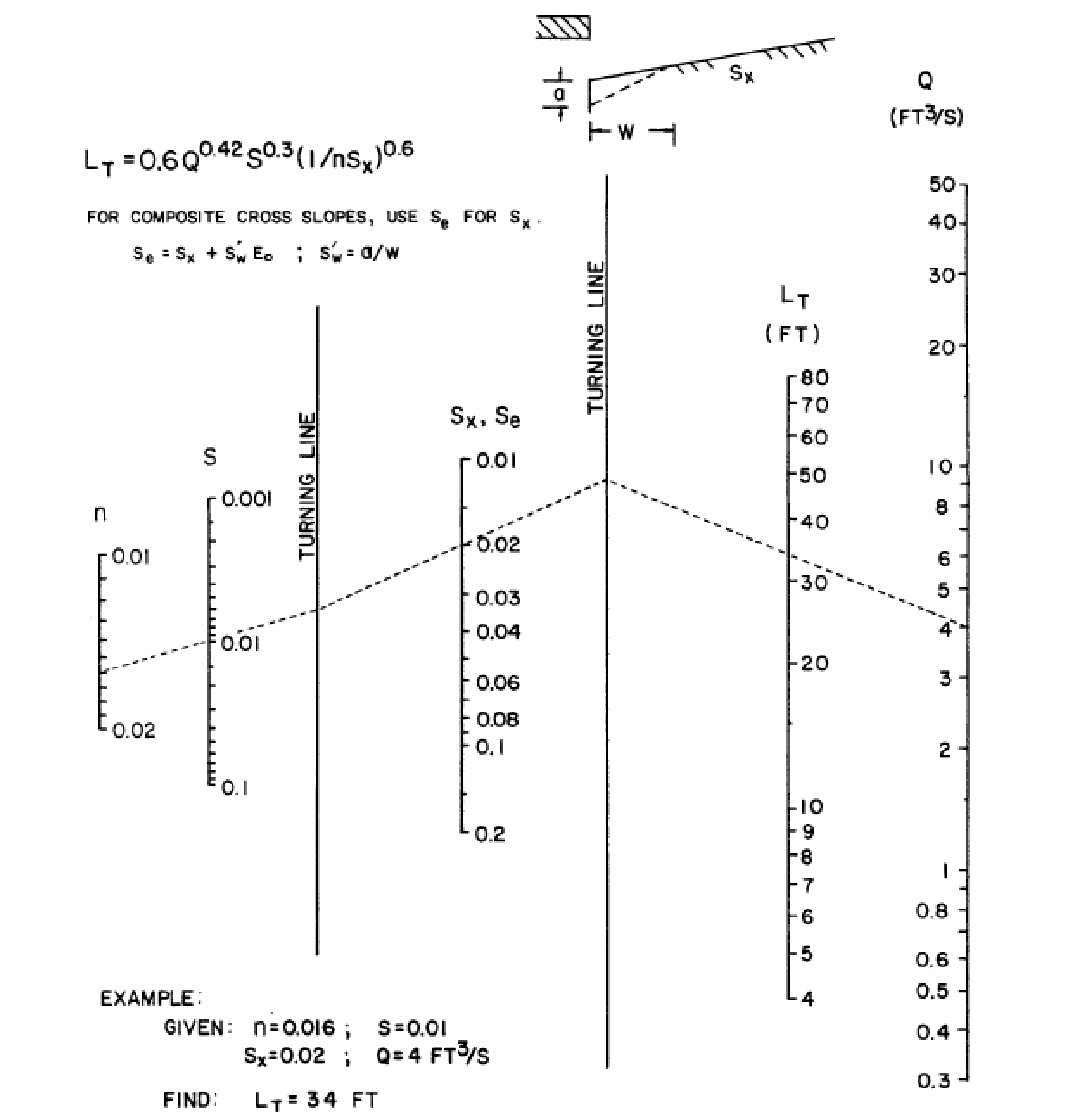
$$S'w = \frac{a}{(W)(12)}$$

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.

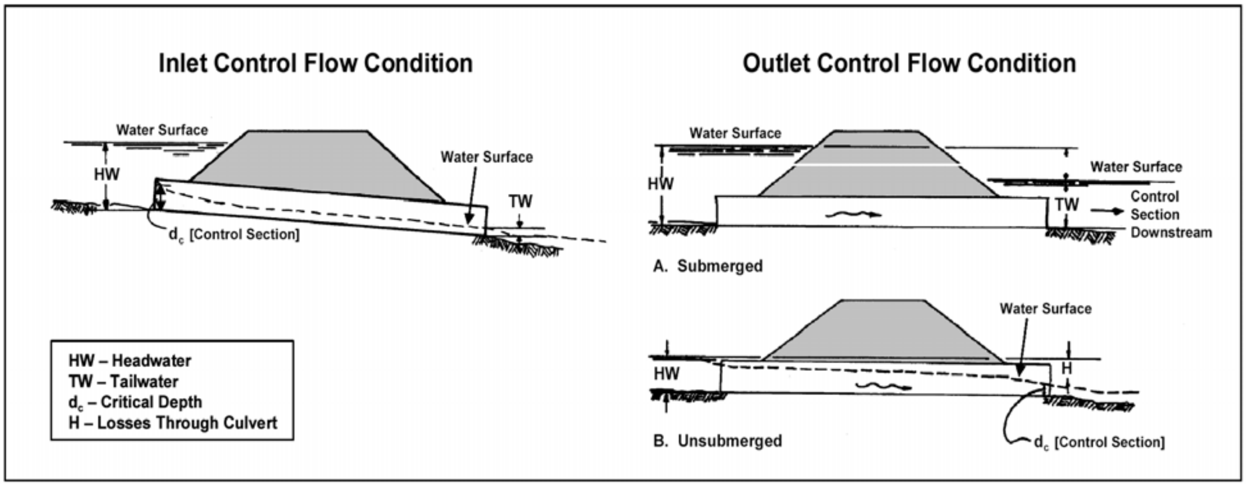
3.10.1 Types of Flow Control

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

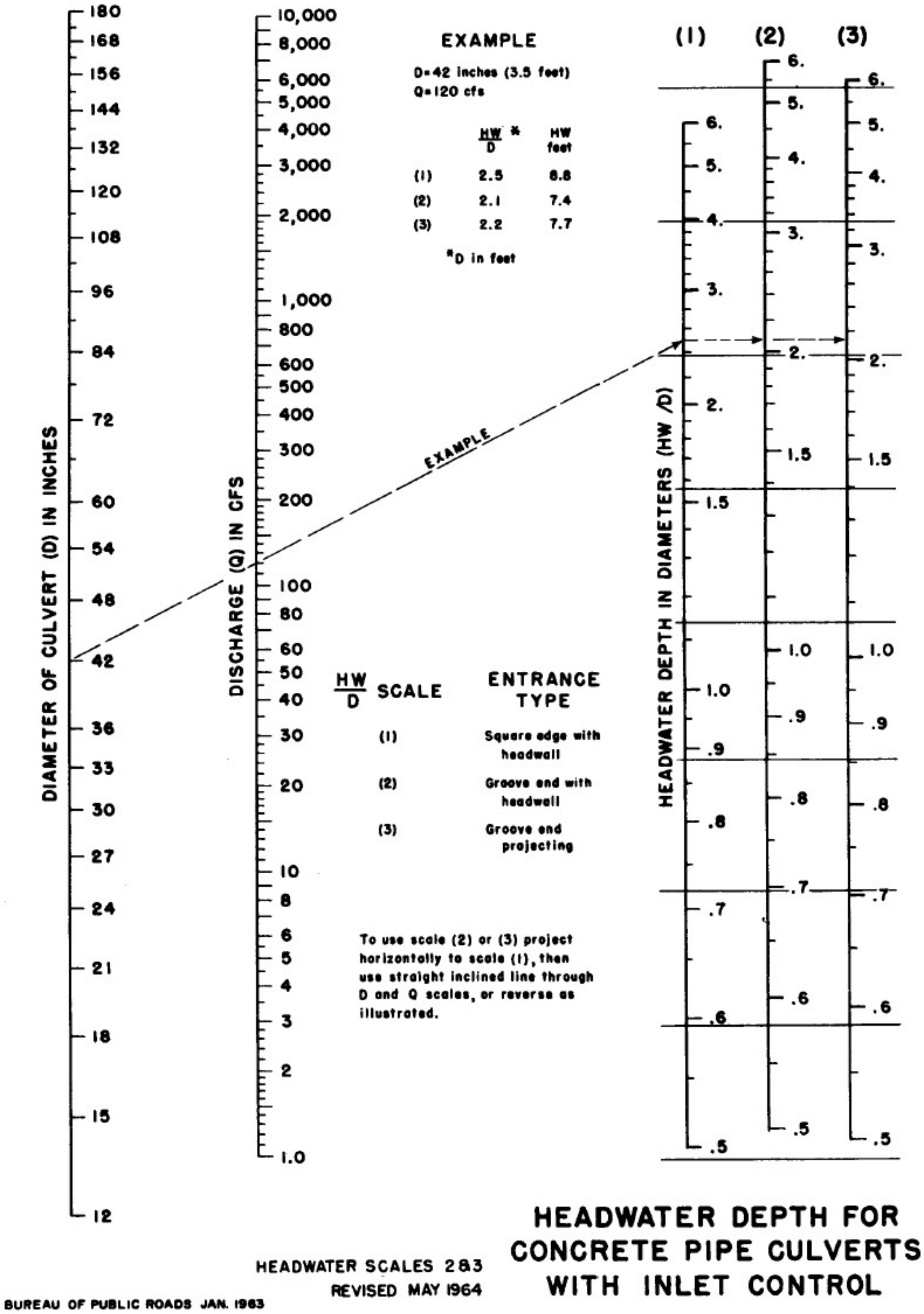
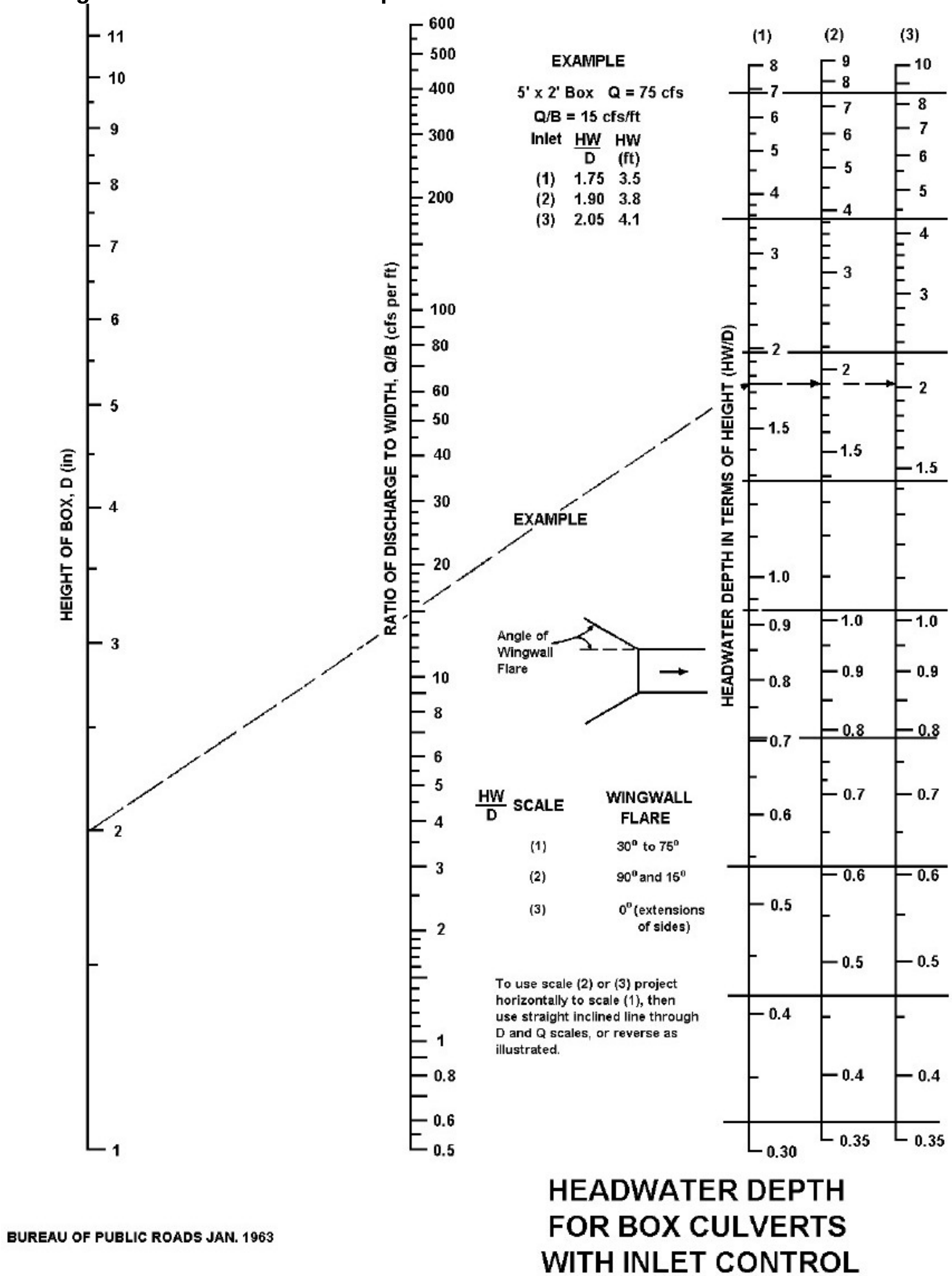


Figure 3.10 Headwater Depth for Concrete Box Culvert with Inlet Control



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

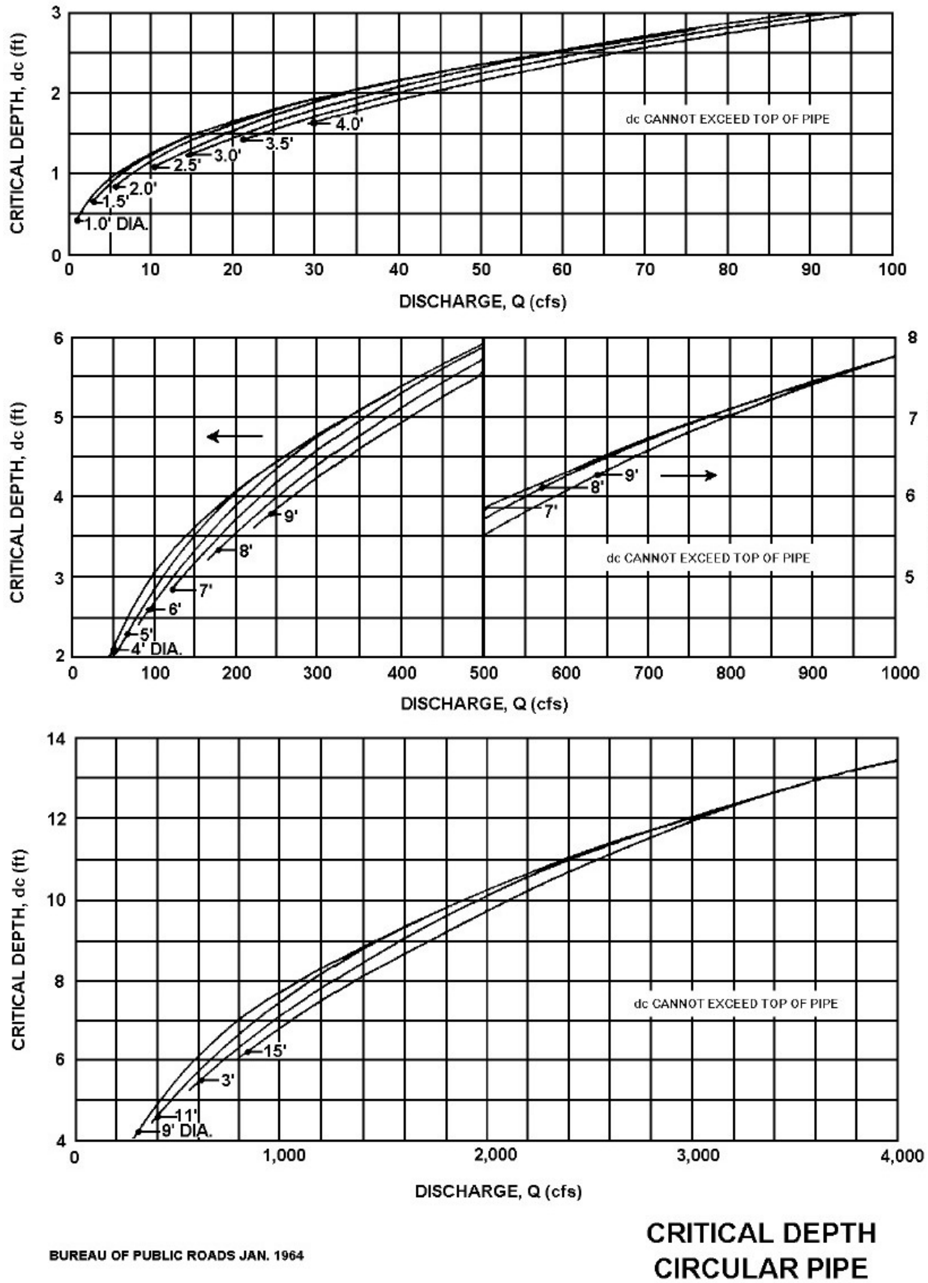
Type of Structure and Design of Entrance	Coefficient Ke
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)	0.2
Mitered to conform to 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

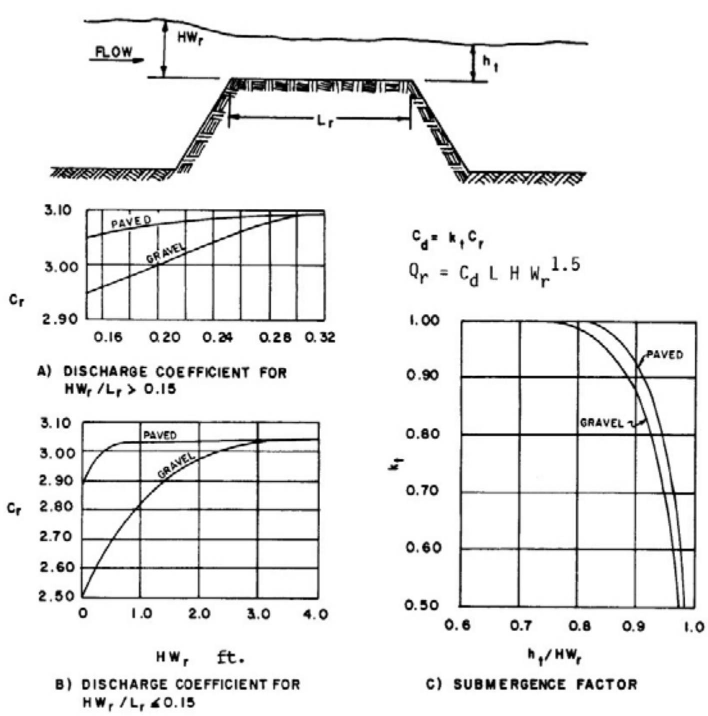
To complete the Culvert Design Form, the roadway overtopping should be analyzed for the larger events. The elevation over the road for a given discharge will be when the discharge over the road plus the discharge through the culvert add up to the given discharge. This can be done through a trial and 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.