CHARLOTTE-MECKLENBURG STORM WATER DESIGN MANUAL

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1.1 PURPOSE

This manual has been developed to assist in the design and evaluation of storm water management facilities in the City of Charlotte and Mecklenburg County. It provides engineering design guidance to:

- local agencies responsible for implementing the Charlotte-Mecklenburg Storm Water Management Program
- engineers responsible for the design of storm water management structures
- developers involved in site planning and design
- others involved in storm water management at various levels who may find the manual useful as a technical reference to define and illustrate engineering design techniques

When accompanied by sound engineering judgment, application of the procedures and criteria presented in this manual should contribute toward the effective and economical solutions of local drainage and flooding problems.

Engineering design methods other than those included in this manual may be used if approved by the City or County Engineering Department. Complete documentation of these methods may required for approval.

1.2 CONTENTS

The manual presents technical and engineering procedures and criteria needed to comply with Charlotte and Mecklenburg County storm water regulations. Following are the chapters included in the manual.

- Chapter 1 Introduction
- Chapter 2 Hydrology
- Chapter 3 Open Channel Hydraulics
- Chapter 4 Storm Drainage Systems
- Chapter 5 Design of Culverts
- Chapter 6 Storage and Detention
- Chapter 7 Energy Dissipation

Each chapter contains the equations, charts and nomographs needed to design specific storm water management facilities. Example problems are used to illustrate the use of the procedures. Where appropriate, desktop procedures are developed for design application. In addition, available computer programs are referenced and users are encouraged to obtain certain computer programs for design application.

1.3 LIMITATIONS

The manual provides a compilation of readily available literature relevant to storm water management activities in the Charlotte-Mecklenburg area. Although it is intended to establish uniform design practices, it neither replaces the need for engineering judgment nor precludes the use of information not presented. Since material presented was obtained from numerous publications and has not been duplicated in its entirety, the user is encouraged to obtain original or additional reference material, as appropriate. References are included at the end of the manual.

1.4 UPDATING

The Charlotte-Mecklenburg Storm Water Design Manual is a dynamic document available at www.charmeck.org/stormwater/regulations/Pages/StormWaterDesignManual.aspx and is updated periodically. It is the responsibility of the user to make sure the most recent standards are being specified. Users may request to receive notification of future revisions. The current manual is available via download only from this website. Hard copies or CDs of the Design Manual are no longer available from the City.

1.5 DESIGN AND CONSTRUCTION CRITERIA

The following criteria will be used for the design and construction of all storm water facilities.

- Discharge leaves site in the same direction and relative location as pre-developed condition.
- Design and installation of all storm water detention facilities must comply with applicable federal, state and local laws. Attention should be given to the applicable Soil Erosion and Sediment Control Ordinances, Post Construction Stormwater Ordinance, and the North Carolina Dam Safety Law.
- In no case shall a structure be located within the impoundment area of any storm water storage facility or over a storm drainage line.
- No utilities (sewer lines, power line, water line, etc.) shall be located within or under any detention facility.
- Detention facilities located within code-required automobile parking areas shall not exceed a maximum water depth of 6 inches. Additional criteria are listed in Chapter 6, section 6.2.5.
- With the exception of erosion control facilities, all detention facilities shall be considered permanent.
- Routine maintenance of all detention facilities shall be the responsibility of the property owner or appointed designee.
- It is recommended that storm water detention facilities be located on the site from which the runoff to be controlled is generated.
- Off-site detention facilities are acceptable provided the land area involved with the facility is delineated on an acceptable map and officially recorded at the Mecklenburg County's Register of Deeds Office as a "Permanent Detention Easement". Also, an official commitment to maintenance of the facility will be required.

1.6 CITY AND COUNTY STORM WATER REGULATIONS

It is the responsibility of the user to be fully aware of all applicable federal, state, and local code requirements when using the Storm Water Design Manual such as, but not limited to:

- Floodplain Regulations
- Post Construction Stormwater Ordinance
- Soil Erosion and Sediment Control
- Stormwater Ordinance
- Subdivision Ordinance

- Zoning Ordinance

1.7 CONCEPT DEFINITIONS

Unless specifically defined in this section, words or phrases used in this manual shall be interpreted so as to give them the meaning they have in common usage and to give this manual its most reasonable application. The following words, terms and phrases, when used in this manual, shall have the meanings ascribed to them in this section and shall be italicized, except where the context clearly indicates a different meaning.

Antecedent Moisture
Antecedent soil moisture conditions are the soil moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably they affect the peak discharge only in the lower range of flood magnitudes - for example below about the 10-year event threshold. As the yearly storm event increases, antecedent moisture has a rapidly decreasing influence on runoff.

Bypass
Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade.

Combination Inlet
A drainage inlet composed of a curb-opening and a grate. This is a Charlotte-Mecklenburg standard catch basin on roadways.

CMP
The abbreviation for Corrugated Metal Pipe.

Critical Depth
Critical depth can best be illustrated as the depth at which water flows over a weir, this depth being attained automatically where no other backwater forces are involved. This is because it is the depth at which the energy content of flow is at a minimum. For a given discharge and cross-section geometry there is only one critical depth.

Cross-drain
Culverts and pipes that are used to convey runoff from one side of a roadway to another.

Crown
Crown refers to the highest part of the pipe (inside top).

Culvert
A structure that conveys any flow collected in an open ended pipe (i.e., headwall, flared end section, mitered end), a cross-drain.

Curb-Opening Inlet
A drainage inlet consisting of an opening in the roadway curb.

Depression Storage
Depression storage is the natural depressions within a watershed which store runoff. Generally, after the depression storage is filled runoff will commence.

Drop Inlet
A drainage inlet with a horizontal or nearly horizontal grate, generally located outside of the roadway.

Equivalent Cross Slope
An imaginary continuous cross slope having conveyance capacity equal to that of the given compound cross-slope.

Flanking Inlets
Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets is to intercept debris as the slope decreases and to act in relief of the inlet at the low point.

Freeboard
Freeboard is an additional depth (to the top of a basin, pond, ditch, etc.) regarded as a safety factor, above the peak design water elevation.

Free Outlets
Free outlets are those outlets whose tailwater is equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Frequency
Frequency is the average time interval between equal magnitude storm events. For example, a 25-year storm event has the probability of occurrence of once every 25 years on the average, or a 4 percent chance of occurrence in any given year.

Frontal Flow
The portion of the gutter flow which passes over the upstream side of a grate.

Grate Perimeter
The sum of the lengths of all sides of a grate. For grated inlets on curbs, neglect the side adjacent to the curb and use only three sides.

Gutter
That portion of the roadway section adjacent to the curb which is utilized to convey storm water runoff.

HDPE
The abbreviation for High Density Polyethylene.

Hydraulic Grade Line (HGL)
The hydraulic grade line is the set of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe system.

Hydraulic Roughness
Hydraulic roughness is a composite of the physical characteristics which influence the flow of water across the earth's surface, whether natural or channelized. It affects both the time response of a watershed and drainage channel as well as the channel storage characteristics.

Hydrograph
The hydrograph is a graph of the time distribution of runoff from a watershed.

Hyetograph
The hyetograph is a graph of the time distribution of rainfall over a watershed.

Improved Inlets
Flared, improved, or tapered inlets indicates a special entrance condition which decreases the amount of energy needed to pass the flow through the inlet and thus increases the capacity of culverts at the inlet.

Infiltration
Infiltration is a complex process of allowing runoff to penetrate the ground surface and flow through the upper soil surface. The infiltration curve is a graph of the time distribution at which this occurs.

Initial Abstractions
All losses (water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration) before surface water runoff begins.

Inlet Efficiency
The ratio of flow intercepted by an inlet to total flow in the gutter.

Interception
Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.

Invert
Invert refers to the flowline (lowest part) of the pipe (inside bottom) or channel.

Lag Time
The lag time is defined as the time from the centroid of the excess rainfall to the peak of the runoff hydrograph.

Peak Discharge
The peak discharge, sometimes called peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event.

Pressure Head
Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Primary Access Streets
A street serving as the only access to parcel(s).

Rainfall Excess
Rainfall excess is the excess water available to runoff after interception, depression storage, and infiltration have been satisfied.

RCP
The abbreviation for Reinforced Concrete Pipe.

Scupper
A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.

Side-Flow Interception
Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.

Splash Over
Portion of the frontal flow at a grate which skips or splashes over the grate and is not intercepted.

Spread
The width of flow measured perpendicularly from the roadway pavement edge or the lip of the gutter.

Steep and Mild Slope
A steep slope culvert operation is where the computed critical depth is greater than the computed uniform depth. A mild slope culvert operation is where critical depth is less than uniform depth.

Submerged Inlets
Submerged inlets are those inlets having a headwater greater than 1.5 times the pipe diameter (1.5D).

Submerged Outlets
Partially submerged outlets are those outlets whose tailwater is higher than critical depth and lower than the height of the culvert. Submerged outlets are those outlets having a tailwater elevation higher than the crown of the culvert. Lowering of the tailwater may have significant effect on the discharge or the backwater profile upstream of the tailwater.

Stage
The stage of a channel or detention facility is the elevation of the water surface above some elevation datum.

Storm Drain
A pipe used for the conveyance of storm water.

Thoroughfare
A street designated as a major or minor thoroughfare as shown on the Charlotte-Mecklenburg Thoroughfare Plan.

Time of Concentration
The time of concentration is the time required for water to flow from the most hydraulically remote point of the basin to the location being analyzed. Thus the time of concentration is the maximum time for water to travel through the watershed, which is not always the maximum distance from the outlet to any point in the watershed.

Uniform Flow
Uniform flow is flow in a prismatic channel of constant cross section having a constant discharge, velocity and depth of flow throughout the reach.

Unit Hydrograph
A unit hydrograph is the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution and which lasts for a specific duration of time (thus there could be a 5-, 10-, 15-minute, etc., unit hydrograph for the same drainage area). The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area.

Usable and Functional Part of the Structure

"Usable and functional part of the structure" shall be defined as being inclusive of living areas, basements, sunken dens, basement utility rooms, crawlspaces, attached carports, and mechanical appurtenances such as furnaces, air conditioners, water pumps, electrical conduits and wiring, but shall not include water lines or sanitary sewer traps, piping and clean-outs.

Velocity Head

A quantity proportional to the kinetic energy of flowing water expressed as a height or head of water.

CHAPTER 2

HYDROLOGY

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2.1 HYDROLOGIC DESIGN POLICIES

2.1.1 Factors Affecting Flood Runoff

For all hydrologic analysis, the following factors shall be evaluated and included when they will have a significant effect on the final results.

Drainage Basin Characteristics
Size
Shape
Slope
Ground Cover
Land Use (Existing Conditions, Existing Zoning)
Geology
Soil Types
Surface Infiltration
Ponding and Storage
Watershed Development Potential (Future Land Use Plans)
Other Characteristics

Stream Channel Characteristics
Geometry and Configuration
Natural Controls
Artificial Controls
Channel Modifications
Aggradation – Degradation
Debris
Hydraulic roughness (Manning's *n*)
Slope
Other Characteristics

Flood Plain Characteristics
Slope
Vegetation
Alignment
Storage
Location of Structures
Obstructions to Flow
Other Characteristics

Meteorological Characteristics
Precipitation Amounts
Time Rate of Precipitation
Historical Flood Heights
Storm Frequency Events
Other Characteristics

2.1.2 Hydrologic Method

Many hydrologic methods are available. Recommended methods and the circumstances for their use are listed in Table 2-1. The recommended methods have been selected for use in the Charlotte-Mecklenburg area based on several considerations, including:

- Verification of their accuracy in duplicating local hydrological estimates of a range of design storms
- Availability of equations, nomographs, and computer programs for the methods
- Use and familiarity with the methods by local governments and consulting engineers

Table 2-1
Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational (see Section 2.4)	0 – 200 Acres	Method can be used for estimating peak flows and the design of small sub-division type storm drainage systems.
NRCS Method (TR-55) (see Section 2.6)	0 – 2,000 Acres	Method can be used for estimating peak flows from developed areas.
HEC-1/HEC–HMS	None	Method can be used for estimating peak flows and hydrographs. Application of HEC-1 is limited by a maximum of 2,000 ordinates in the hydrograph, and of the particular hydrograph generation technique (NRCS Unit Hydrograph, Kinematic Wave, etc.) including the minimum time step interval expressed in the HEC-1 manual. Also, see section 2.1.4

[1]Size limitations refers to the sub-watershed size to the point where storm water management facility (i.e., culvert, inlet) is located

In using these methods, the procedures outlined in this chapter should be followed.

If other methods are used, they must first be calibrated to local conditions and tested for accuracy and reliability by the user. Third party computer software not identified in this table must be independently verified and calibrated to the recommended methods by the professional prior to its use. If other software is used, it will be compared to HEC-1/HEC-HMS to make sure it reproduces equivalent results. In addition to verifying results, complete source documentation for the software must be submitted for approval.

2.1.3 Storm Water Conveyance Design Policy

All storm water conveyances shall be designed based on fully developed land use conditions as shown on current County and City Land Use Plans and Zoning Maps or existing land use, whichever generates the higher runoff rate.

2.1.4 HEC-1 Limitations

The following are limitations of the HEC-1 model hydrograph generation routine using the NRCS unit dimensionless hydrograph. In addition to the items in the list, the user of the HEC-1 model

must be knowledgeable of the limitations of the hydrologic and hydraulic methodologies which are being applied by the model.

- The computation interval must not be significantly less than the minimum rainfall increment on the "PH" record, otherwise a portion of the rainfall is lost because the program cannot perform the logarithmic interpolation necessary for the development of the complete hyetograph. Standard HEC-1 model input uses a 5-minute "worst" precipitation increment. Therefore, the model may not be used with a computation interval less than 5 minutes unless the rainfall hyetograph is input with "PC" or "PI" records. The computation interval, when multiplied by the number of hydrograph ordinates, must also be greater than the storm duration which is planned to be studied (6 hour, 24 hour, etc.). Not having the program set to allow the storm to run causes hydrographs to be inappropriately peaked due to the lack of necessary time to fit in the needed runoff hydrograph.

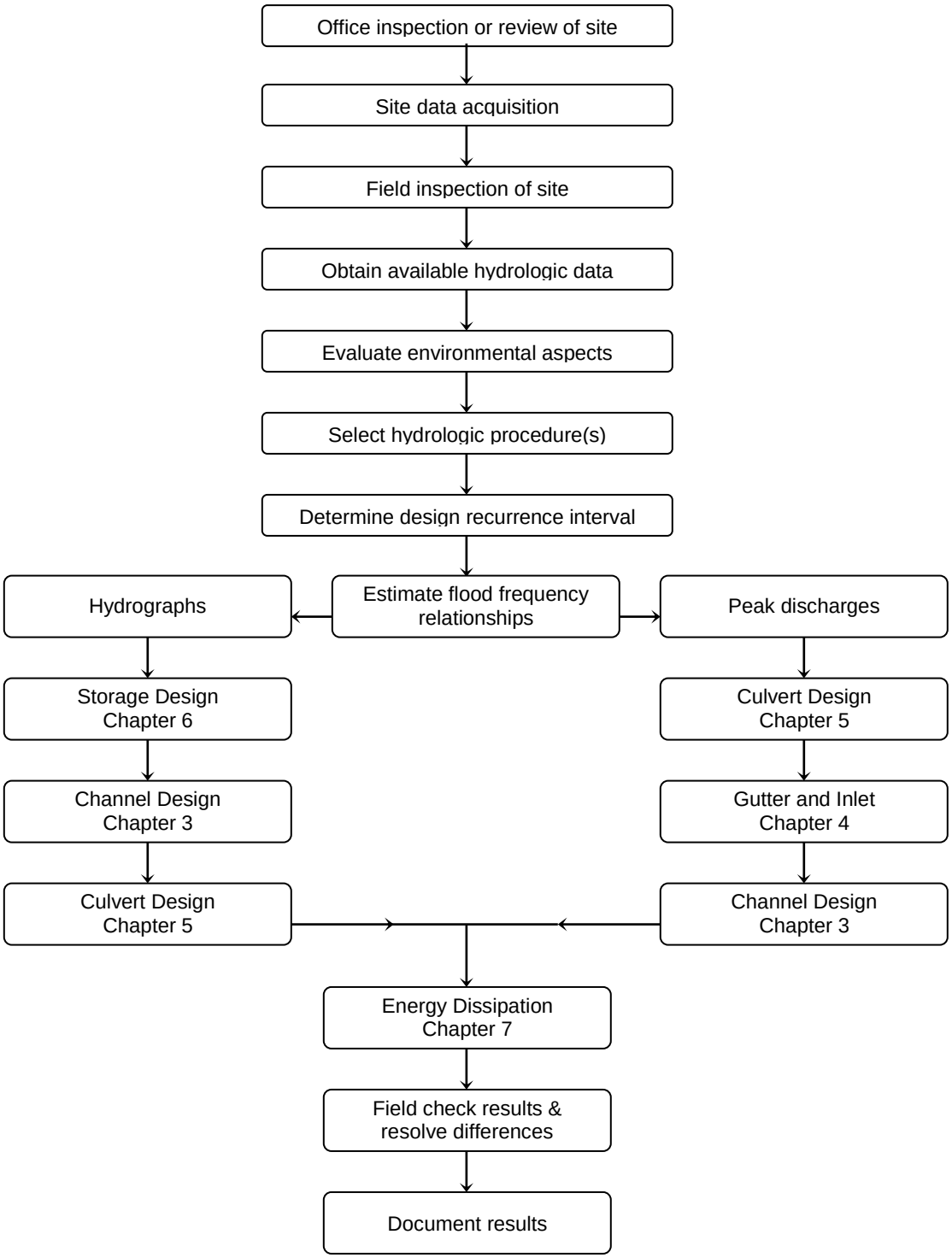
- The NRCS unit dimensionless hydrograph may not be used when the computation interval is greater than 0.29 times the lag time of the watershed. This limitation translates into a minimum time of concentration of 5.75 minutes which typically occurs in watersheds of 3 acres or less. The result of exceeding this limitation is that the resulting hydrograph may underestimate the peak flow by computing the peak flow values on either side of the peak of the hydrograph. However, the volume under the resulting hydrograph is correct and all volume computation such as detention storage is correct.

2.2 HYDROLOGIC ANALYSIS PROCEDURE FLOWCHART

2.2.1 Purpose and Use

The purpose of the hydrologic analysis procedure flowchart is to show the steps or elements which need to be completed for the hydrologic analysis, and the different designs that will use the hydrologic estimates.

2.2.2 Design Flowchart



2.3 DESIGN FREQUENCY

2.3.1 Design Frequencies

Description	***Design Storm***
Storm system pipes	10 year
Ditch systems	10 year
Culverts/Cross-drain (subdivision streets)	25 year
Culverts/Cross-drain (thoroughfare roads)	50 year
Culverts (over regulated floodways)	100 year
Culverts/Cross-drain (primary access streets)	No overtopping in 100 year
Usable and functionable part of structure or building (as defined in the Subdivision Ordinance)	100 year + 1 foot

2.3.2 Rainfall Intensity

The following rainfall intensities (Table 2-2) shall be used for all hydrologic analysis.

Table 2-2
Rainfall Intensities - Charlotte, North Carolina

IDF variables for equation:

$$\textbf{Intensity (I)} = \frac{a}{(t+b)^n} \qquad (2.1)$$

t = duration of rainfall (minutes - min)
I = intensity (inches/hour - in/hr)
a, b, n = storm fitting parameters

a	44.7516	61.3997	83.3331	97.3148	104.2990	116.4790
b	10	12	15	15	15	15
n	0.8070	0.8035	0.8256	0.8254	0.8179	0.8223

Time		**Recurrence interval (years)**					
Hours	**Minutes**	**2**	**5**	**10**	**25**	**50**	**100**
0	**5**	5.03	6.30	7.03	8.21	9.00	9.92
	6	4.78	6.02	6.75	7.89	8.65	9.53
	7	4.55	5.76	6.49	7.59	8.32	9.17
	8	4.34	5.53	6.26	7.31	8.03	8.84
	9	4.16	5.32	6.04	7.06	7.75	8.54
	10	3.99	5.12	5.84	6.83	7.50	8.26
	15	3.33	4.35	5.03	5.87	6.46	7.11
	16	3.23	4.22	4.89	5.72	6.29	6.92
	17	3.13	4.10	4.77	5.57	6.13	6.74
	18	3.04	3.99	4.65	5.43	5.97	6.57
	19	2.96	3.89	4.53	5.30	5.83	6.41
	20	2.88	3.79	4.43	5.17	5.69	6.26
	21	2.80	3.70	4.32	5.05	5.56	6.12
	22	2.73	3.61	4.23	4.94	5.44	5.98
	23	2.66	3.53	4.14	4.83	5.32	5.85
	24	2.60	3.45	4.05	4.73	5.21	5.73
	25	2.54	3.37	3.96	4.63	5.10	5.61
	26	2.48	3.30	3.88	4.54	5.00	5.50
	27	2.43	3.23	3.81	4.45	4.90	5.39
	28	2.38	3.17	3.73	4.36	4.81	5.29
	29	2.33	3.11	3.66	4.28	4.72	5.19
	30	2.28	3.05	3.60	4.20	4.64	5.09
	40	1.90	2.57	3.05	3.56	3.93	4.32
	50	1.64	2.23	2.66	3.10	3.43	3.76

Table 2-2
Rainfall Intensities - Charlotte, North Carolina (continued)

Time		Recurrence interval (years)					
Hours	**Minutes**	**2**	**5**	**10**	**25**	**50**	**100**
1	**60**	1.45	1.98	2.36	2.76	3.05	3.34
2	**120**	0.88	1.21	1.45	1.70	1.89	2.06
3	**180**	0.65	0.90	1.07	1.25	1.40	1.52
6	**360**	0.38	0.53	0.62	0.73	0.82	0.89
12	**720**	0.22	0.31	0.36	0.42	0.47	0.51
24	**1440**	0.13	0.18	0.20	0.24	0.27	0.29
P_{24} (inches – in.)		**3.12**	**4.32**	**4.80**	**5.76**	**6.48**	**6.96**

2.4 RATIONAL METHOD

2.4.1 Introduction

When using the rational method some precautions should be considered.

- In determining the C value (land use) for the drainage area, hydrologic analysis should take into account future land use changes. Drainage facilities shall be designed for future land use conditions as specified in the County and City Land Use Plans and Zoning Maps (or existing land use, whichever generates the higher runoff rate).
- Since the rational method uses a composite C value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis, then the basin should be divided into two or more sub-drainage basins for analysis.
- The charts, graphs, and tables included in this section are given to assist the engineer in applying the rational method. The engineer should use good engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

2.4.2 Runoff 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, frequency factor, 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:

$$\mathbf{Q = C_f CIA} \qquad \textbf{(2.2)}$$

Where: Q = maximum rate of runoff (cubic feet/second - cfs)

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration (in/hr)

A = drainage area contributing to the design point location (acres)

C_f = frequency factor

The C_f values that can be used are listed in Table 2-3. The product of C_f multiplied by C shall not exceed 1.0.

Table 2-3
Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
2	1
10	1
25	1.1
50	1.2
100	1.25

2.4.3 Time of Concentration

Use of the rational formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I) from Table 2-2. The time of concentration is interpreted as the longest time of flow from points on the watershed ridge to the point of interest. The most common method for determining time of concentration is outlined in section 2.6.5.

Although not commonly used, the Kirpich Equation is an acceptable method for calculating time of concentration.

$$t_c = 0.0078 \times \frac{L^{0.77}}{S^{0.385}} \qquad (2.3)$$

Where: t_c = Time of Concentration (min)

L = Longest hydraulic flow length (foot - ft)

S = Surface slope (foot/foot - ft/ft)

This formula can be used to estimate the time of concentration for basins with well defined channels, for overland flow on grassed, concrete or asphalt surfaces, and for concrete channels. In the case where the flow is overland on grassed surfaces, multiply t_c by 2. For overland flow over concrete/asphalt surfaces or concrete channels, multiply the t_c by 0.4 and 0.2, respectively. Within the City of Charlotte, the Kirpich Equation can be used in the design of storm water conveyance systems. The Kirpich Equation cannot be used to design storm water control measures where a comparison of pre-development and post-development hydrology is necessary.

For each drainage area, flow length is determined from the inlet to the most hydrologically remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. Other formulas or charts may be used to calculate overland flow time if

approved by the City/County Engineering Departments. **Note: time of concentration cannot be less than 5 minutes.**

A common error should be avoided when calculating t_c. 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.

2.4.4 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in inches/hour for a duration equal to the time of concentration for a selected return period. 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 Rainfall-Intensity-Duration data given in Table 2-2. Straight-line interpolation can be used to obtain rainfall intensity values for storm durations between the values given in Table 2-2.

2.4.5 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 2-4 gives the recommended runoff coefficients for the Rational Method.

Table 2-4
Recommended Runoff Coefficient Values

Description of Area	Runoff Coefficient (C)
Lawns	0.30
Wooded	0.25
Streets	0.95
Gravel Areas	0.55
Drives, walks, roofs	0.95
Bare soils	0.45
Residential (including streets):	
Single-Family (Lot < 20,000 square feet - SF)	0.60
Single-Family (Lot > 20,000 square feet - SF)	0.50
Multi-family, Attached	0.70
Industrial:	
Light areas	0.70
Heavy areas	0.80
Office Parks	0.75
Shopping Centers	0.80

Note: The above runoff coefficients are valid for 2-year to 10-year storm frequencies only. Coefficients must be accompanied with a C_f factor when used for less frequent, higher intensity storms.

2.4.6 Composite Coefficients

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 2-4 by using percentages of different land uses. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area.

It should be remembered that the rational method 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 area.

2.5 EXAMPLE PROBLEM - RATIONAL METHOD

Introduction

Following is an example problem which illustrates the application of the Rational Method to estimate peak discharges.

Problem

Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert for a 25-year and 100-year return period.

Site Data

From a topographic map field survey, the area of the drainage basin upstream from the point in question is found to be 18 acres. In addition the following data were measured:

Flow Path
Average slope = 2.0%
Length of flow in well defined channel = 1,000 ft

Land Use

From existing land use maps, land use for the drainage basin was estimated to be:

Single Family (< 20,000 SF)	80%
Light Industrial	20%

Time of Concentration

Since this problem involves determining the flows for a storm water conveyance system, utilization of the conservative and simplistic Kirpich method may be appropriate:

Using Equation 2.3 with a flow length of 1,000 ft and slope of 2.0%

$$t_c = 0.0078 \times \frac{1{,}000^{0.77}}{0.02^{0.385}} = 7.2 \text{ minutes}$$

Rainfall Intensity

From Table 2-2, with a duration equal to 7.2 minutes, the intensity can be selected by interpolation.

I_{25} (25-year return period) = 7.53 in/hr
I_{100} (100-year return period) = 9.10 in/hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined in the following table by utilizing the values from Table 2-4.

Land Use	(1) Percent Of Total Land Area	(2) Runoff Coefficient	(3) Weighted Runoff Coefficient*
Residential (Single-Family, < 20,000 SF)	.80	.60	.48
Light Industrial	.20	.70	.14
Total Weighted Runoff Coefficient			.62

** Column 3 equals column 1 multiplied by column 2.*

Peak Runoff

From the rational method equation 2.2:

$Q_{25} = C_f CIA = 1.1 \times .62 \times 7.53 \text{ in/hr} \times 18 \text{ acres} = 92.4 \text{ cfs}$
$Q_{100} = C_f CIA = 1.25 \times .62 \times 9.10 \text{ in/hr} \times 18 \text{ acres} = 126.9 \text{ cfs}$

These are the estimates of peak runoff for a 25-yr and 100-yr design storm for the given basin.

2.6 NRCS UNIT HYDROGRAPH

2.6.1 Introduction

The Natural Resources Conservation Service (NRCS) hydrologic method requires basic data similar to the Rational Method; drainage area, a runoff factor, time of concentration, and rainfall. The NRCS approach; however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the *NRCS National Engineering Handbook, Section 4*.

The NRCS method includes the following basic steps:

1. Determination of a composite curve number which represents and considers different land uses within the drainage area.

2. Calculation of time of concentration to the design point location.

3. Using the Type II rainfall distribution or the balanced storm distribution and peaking factor 484, total and excess rainfall amounts are determined.

4. Using the unit hydrograph approach, triangular and composite hydrographs are developed for the drainage area.

2.6.2 Equations and Concepts

The following discussion outlines the equation and basic concepts utilized in the NRCS method.

Drainage Area—the drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within drainage area, and route flows to design study points of interest.

Rainfall—The NRCS method applicable to the Charlotte-Mecklenburg area is based on a storm event which has a Type II time distribution. Figure 2-1 shows this distribution. To use this distribution for the one year storm event it is necessary for the user to obtain the 24-hour rainfall intensity from Table 2-2 and multiply by the 24-hour duration to obtain the depth (P_{24} in Figure 2-1) from Table 2-5. This depth is then distributed according to Figure 2-1. To use this distribution for other storm events it is necessary for the user to obtain the 6-hour rainfall intensity from Table 2-2 and multiply by the 6-hour duration to obtain the depth (P_6 in Figure 2-1) from Appendix 2B for the frequency of the design storm. This depth is then distributed according to Figure 2-1. Rainfall may also be distributed using a center weighted "balanced" distribution. Tables 2-5 thru 2-10 show a balanced distribution for various storm events.

Rainfall-Runoff Equation—A relationship between accumulated rainfall and accumulated runoff was derived by NRCS from experimental plots for numerous soils and vegetative cover conditions. The following NRCS runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$\mathbf{Q = \frac{(P - I_a)^2}{(P - I_a) + S}} \qquad \textbf{(2.4)}$$

Where: Q = accumulated direct runoff (inches)
P = accumulated rainfall or potential maximum runoff (inches)
I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff (inches)
S = potential maximum soil retention (inches)

The empirical relationship used in the NRCS runoff equation for estimating I_a is :

$$\mathbf{I_a = 0.2S} \qquad \textbf{(2.5)}$$

Substituting 0.2S for I_a in equation 2.4, the NRCS rainfall-runoff equation becomes:

$$\mathbf{Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}} \qquad \textbf{(2.6)}$$

Where: S = (1,000/CN) – 10
CN = NRCS curve number (see section 2.6.3)

Figure 2-2 shows a graphical solution of this equation which enables the precipitation excess from a storm to be obtained if the total rainfall and watershed curve number are known. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurs on a watershed with a curve number of 85.

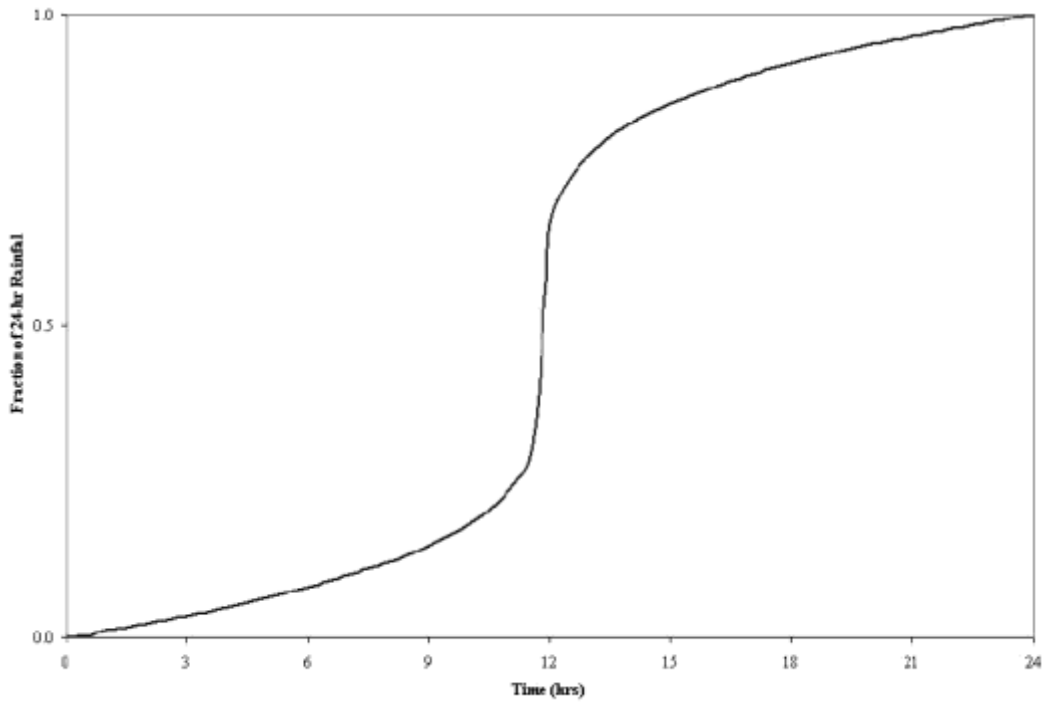


Figure 2-1 NRCS (SCS) Type II Rainfall Distribution
Source: http://www.seda-cog.org/union/lib/union/appendix_p_-_stormwater_tables_calculations,_cns.pdf

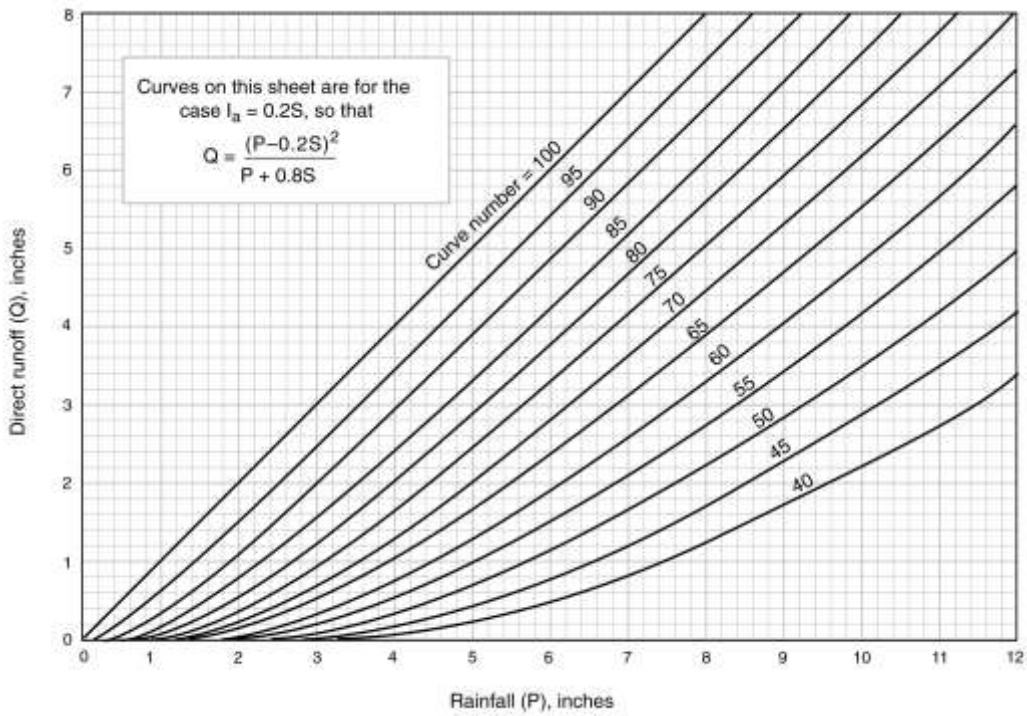


Figure 2-2 Solution of Runoff Equation
Source: ftp://ftp.wcc.nrcs.usda.gov/downloads/hydrology_hydraulics/tr55/tr55.pdf

Table 2-5
1-Year, 24-hour Storm Event, 6-Minute Time Increment

IN	6									
PB	2.58									
PI	.0000	.0010	.0010	.0010	.0011	.0010	.0011	.0010	.0011	.0011
PI	.0011	.0011	.0011	.0011	.0012	.0011	.0012	.0011	.0012	.0012
PI	.0012	.0012	.0012	.0013	.0012	.0012	.0013	.0012	.0013	.0013
PI	.0013	.0013	.0013	.0013	.0014	.0013	.0014	.0014	.0013	.0014
PI	.0014	.0014	.0014	.0015	.0015	.0015	.0015	.0015	.0015	.0016
PI	.0016	.0016	.0016	.0017	.0017	.0016	.0018	.0017	.0017	.0018
PI	.0018	.0018	.0018	.0019	.0019	.0018	.0020	.0019	.0019	.0020
PI	.0020	.0020	.0020	.0021	.0021	.0021	.0021	.0021	.0021	.0022
PI	.0022	.0022	.0024	.0024	.0026	.0026	.0028	.0029	.0029	.0030
PI	.0032	.0032	.0032	.0032	.0032	.0032	.0033	.0034	.0036	.0038
PI	.0039	.0041	.0044	.0046	.0048	.0051	.0054	.0058	.0062	.0066
PI	.0070	.0077	.0086	.0096	.0106	.0115	.0238	.0476	.0764	.1371
PI	.0951	.0190	.0166	.0144	.0122	.0098	.0084	.0080	.0074	.0068
PI	.0064	.0060	.0056	.0054	.0052	.0048	.0046	.0044	.0042	.0040
PI	.0038	.0037	.0036	.0035	.0034	.0034	.0033	.0033	.0032	.0031
PI	.0030	.0030	.0029	.0028	.0027	.0027	.0026	.0026	.0025	.0024
PI	.0023	.0023	.0022	.0023	.0022	.0022	.0022	.0021	.0021	.0021
PI	.0021	.0020	.0020	.0020	.0019	.0020	.0019	.0019	.0018	.0018
PI	.0018	.0018	.0017	.0018	.0017	.0017	.0016	.0017	.0016	.0016
PI	.0015	.0016	.0015	.0015	.0015	.0014	.0014	.0014	.0013	.0014
PI	.0013	.0013	.0013	.0013	.0013	.0012	.0013	.0013	.0012	.0013
PI	.0012	.0013	.0012	.0013	.0012	.0012	.0013	.0012	.0012	.0012
PI	.0012	.0012	.0012	.0012	.0012	.0011	.0012	.0012	.0011	.0012
PI	.0011	.0012	.0011	.0012	.0011	.0011	.0012	.0011	.0011	.0011
PI	.0011	.0000								

Table 2-6
2-Year Precipitation Data

2-year, 6-Hour Balanced Storm Rainfall Distribution

Time Interval	5 min	15 min	1 hour	2 hour	3 hour	6 hour
Rainfall depth (in)	0.42	0.83	1.45	1.76	1.95	2.28

2-Year, 6-Hour Storm Event, 5-Minute Time Increment

PI	.000	.007	.007	.008	.008	.008	.008	.009	.009	.009
PI	.009	.009	.010	.010	.010	.011	.011	.011	.012	.012
PI	.014	.015	.016	.017	.018	.021	.022	.024	.026	.028
PI	.031	.044	.050	.058	.089	.115	.242	.420	.168	.100
PI	.064	,054	.047	.033	.029	.027	.025	.023	.021	.018
PI	.017	.016	.016	.015	.014	.012	.012	.011	.011	.010
PI	.010	.009	.009	.009	.009	.008	.008	.008	.008	.008
PI	.007	.007	.007	.000						

Table 2-7
10-Year Precipitation Data

10-Year, 6-Hour Balanced Storm Rainfall Distribution

Time Interval	5 min	15 min	1 hour	2 hour	3 hour	6 hour
Rainfall depth (in)	0.59	1.26	2.36	2.90	3.21	3.72

10-Year, 6-Hour Storm Event, 5-Minute Time Increment

PI	.000	.011	.011	.011	.012	.012	.012	.013	.013	.013
PI	.014	.014	.015	.015	.016	.016	.017	.018	.018	.023
PI	.024	.025	.026	.027	.029	.036	.039	.042	.045	.049
PI	.054	.079	.089	.103	.161	.201	.395	.590	.275	.177
PI	.112	.095	.084	.057	.051	.047	.043	.040	.038	.030
PI	.028	.027	.025	.024	.023	.019	.018	.017	.017	.016
PI	.016	.015	.015	.014	.014	.013	.013	.012	.012	.012
PI	.011	.011	.011	.000						

Table 2-8
25-Year Precipitation Data

25-Year, 6-Hour Balanced Storm Rainfall Distribution

Time Interval	5 min	15 min	1 hour	2 hour	3 hour	6 hour
Rainfall depth (in)	0.68	1.47	2.76	3.40	3.75	4.38

25-Year, 6-Hour Storm Event, 5-Minute Time Increment

PI	.000	.014	.014	.015	.015	.015	.016	.016	.016	.017
PI	.017	.018	.019	.019	.020	.020	.022	.022	.023	.026
PI	.027	.028	.029	.031	.033	.043	.046	.049	.053	.058
PI	.064	.093	.104	.120	.188	.234	.461	.680	.321	.207
PI	.131	.112	.098	.067	.061	.056	.051	.048	.045	.034
PI	.032	.030	.029	.027	.026	.023	.022	.021	.021	.020
PI	.019	.019	.018	.017	.017	.017	.017	.016	.015	.015
PI	.015	.014	.014	.000						

Table 2-9
50-Year Precipitation Data

50-Year, 6-Hour Balanced Storm Rainfall Distribution

Time Interval	5 min	15 min	1 hour	2 hour	3 hour	6 hour
Rainfall depth (in)	0.75	1.62	3.05	3.78	4.20	4.92

50-Year, 6-Hour Storm Event, 5-minute Time Increment

PI	.000	.016	.016	.016	.017	.018	.018	.019	.019	.019
PI	.020	.020	.021	.022	.022	.023	.024	.025	.026	.031
PI	.032	.034	.035	.037	.039	.050	.053	.056	.061	.066
PI	.073	.103	.116	.134	.208	.259	.508	.750	.354	.229
PI	.146	.124	.109	.077	.069	.063	.059	.055	.051	.040
PI	.038	.036	.034	.033	.031	.026	.025	.024	.024	.023
PI	.022	.021	.021	.020	.020	.019	.019	.018	.018	.017
PI	.017	.016	.016	.000						

Table 2-10
100-Year Precipitation Data

100-Year, 6-Hour Balanced Storm Rainfall Distribution

Time Interval	5 min	15 min	1 hour	2 hour	3 hour	6 hour
Rainfall depth (in)	0.83	1.77	3.34	4.12	4.56	5.34

100-Year, 6-Hour Storm Event, 5-Minute Time Increment

PI	.000	.017	.017	.018	.018	.019	.020	.020	.020	.021
PI	.022	.022	.023	.023	.024	.025	.026	.027	.028	.032
PI	.034	.035	.037	.039	.041	.053	.056	.060	.035	.071
PI	.078	.113	.126	.147	.226	.282	.555	.830	.386	.250
PI	.160	.136	.119	.082	.074	.068	.063	.058	.055	.042
PI	.040	.038	.036	.034	.033	.029	.027	.026	.025	.025
PI	.024	.023	.022	.022	.021	.021	.020	.020	.019	.019
PI	.018	.018	.017	.000						

2.6.3 Runoff Factor

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land cover, soil types and land slope. The NRCS uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential.

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the NRCS has divided soils into four hydrologic soil groups as follows:

> **Group A** - Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well drained sand and gravels.
>
> **Group B** - Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.
>
> **Group C** - Soils having moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water of soils with moderately fine to fine texture.
>
> **Group D** - Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils for Charlotte and Mecklenburg County and their hydrologic classifications are presented in Table 2-11 below. Soil survey maps can be obtained from local NRCS offices.

Table 2-11
Hydrologic Soil Groups for Charlotte-Mecklenburg

Series Name	Hydrologic Group	Series Name	Hydrologic Group
Appling	B	Lignum	C
Cecil	B	Mecklenburg	C
Davidson	B	Monacan	C
Enon	C	Pacolet	B
Georgeville	B	Pits	D
Goldston	C	Vance	C
Helena	C	Wilkes	C
Iredell	D		

Consideration should be given to the effects of soil compaction due to development on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with

the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for all hydrologic analysis.

Table 2-12 gives recommended curve number values for a range of different land uses.

2.6.4 Modifications for Developed Conditions

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, consider whether the impervious areas connect directly to the drainage system, or to lawns or other pervious areas where infiltration can occur.

The curve number values given in Table 2-12 on the following page are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into a drainage system.

It is possible that curve number values from developed areas could be reduced by not directly connecting impervious surfaces to the drainage system. For a discussion of connected and unconnected impervious areas and their effect on curve number values see Appendix A at the end of this chapter.

Table 2-12
Runoff Curve Numbers[1]

Cover type and hydrologic condition	Average percent impervious area [2/]	Curve numbers for hydrologic soil group A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3/]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 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 sewers (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
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Agricultural Lands					
Woods					
poor hydrologic condition		45	66	77	83
fair hydrologic condition		36	60	73	79
good hydrologic condition		30	55	70	77
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

1 Average runoff condition, and Ia = 0.2S.

2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas area 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.

3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

Source: Source: 210-VI-TR-55, Second Edition, June 1986

2.6.5 Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times of consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed.

Following is a discussion of related procedures and equations.

2.6.5.1 Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$\mathbf{T_t = \frac{L \ x \ 0.0167}{V}} \qquad \textbf{(2.7)}$$

Where: T_t = travel time (min)
L = flow length (ft)
V = average velocity (feet/second - ft/s)

2.6.5.2 Time of Concentration

The time of concentration is the sum of T_t values for the various consecutive flow segments along the path extending from the hydraulically most distant point in the watershed to the point of interest.

$$\mathbf{t_c = T_{t1} + T_{t2} \ldots T_n} \qquad \textbf{(2.8)}$$

Where: t_c = time of concentration (hour - hr)
n = number of flow segments

2.6.5.3 Sheet Flow

Sheet flow is flow over plane surfaces. It occurs in the headwater of streams. With sheet flow, the friction value (Manning's *n*) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These *n* values are for very shallow flow depths of about 0.1 foot or so. Also please note, when designing a drainage system, the sheet flow path is not necessarily the same before and after development and grading operations have been completed. **Selecting sheet flow paths in excess of 100 feet in developed areas and 300 feet in undeveloped areas should be done only after careful consideration.**

For sheet flow less than 300 feet in undeveloped areas and less than 100 ft in developed areas use Manning's kinematic solution (Overton and Meadows 1976) to compute T_t:

$$\mathbf{T_t = \frac{0.42\,(nL)^{0.8}}{(P_2)^{1/2}(S)^{0.4}}} \qquad \textbf{(2.9)}$$

Where: T_t = travel time (min)
n = Manning's roughness coefficient, reference Table 2-13
L = flow length (ft)
P_2 = 2-year, 24 hour rainfall = 3.12 inches
S = slope of hydraulic grade line (land slope – ft/ft)

Substituting the constant rainfall amount the travel equation becomes:

$$\mathbf{T_t = \frac{0.238\,(nL)^{0.8}}{(S)^{0.4}}} \qquad \textbf{(2.10)}$$

Thus the final equations for paved and unpaved areas are:

Paved (n = .011)

$$\mathbf{T_t = 0.0065\,[(L)^{0.8} / (S)^{0.4}]} \qquad \textbf{(2.11)}$$

$$\mathbf{V = 2.68(S)^{0.4}(L)^{0.2}} \qquad \textbf{(2.12)}$$

Unpaved (n = .24)

$$\mathbf{T_t = 0.076\,[\,(L)^{0.8} / (S)^{0.4}\,]} \qquad \textbf{(2.13)}$$

$$\mathbf{V = 0.22(S)^{0.4}(L)^{0.2}} \qquad \textbf{(2.14)}$$

Where: V = Velocity (ft/s)
T_t = Travel time (min)

Table 2-13
Roughness Coefficients (Manning's *n*)[1] for Sheet Flow

Surface Description	n
Smooth surfaces (concrete, asphalt gravel, or bare soil	0.016
Fallow (no residue)	0.05
Cultivated Soils:	
Residue Cover < 20%	0.06
Residue Cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1]The *n* values are a composite of information by Engman (1986).

[2]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue gamma grass, and native grass mixture.

[3]When selecting *n*, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Source: NRCS, TR-55, Second Edition, June 1986

2.6.5.4 Shallow Concentrated Flow

After a maximum of 300 feet in undeveloped areas or 100 feet in developed areas, sheet flow usually becomes shallow concentrated flow. Average velocities for estimating travel time for shallow concentrated flow can be computed using the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $\mathbf{V = 16.1345(S)^{1/2}}$ **(2.15)**

Paved $\mathbf{V = 20.3282(S)^{1/2}}$ **(2.16)**

Where: V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse, slope, ft/ft)

These two equations are based on the solution of Manning's equation with different assumptions for *n* (Manning's roughness coefficient) and r (hydraulic radius, ft). For unpaved areas, *n* is 0.05 and r is 0.4; for paved areas, *n* is 0.025 and r is 0.2.

After determining average velocity using equations 2.15 or 2.16, use equation 2.7 to estimate travel time for the shallow concentrated flow segment.

2.6.5.5 Channelized Flow

Open channel flow is assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Flow within pipes and culverts not under pressure is considered closed channel flow. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation. Manning's velocity for pipes assumes a fully flowing condition.

Manning's equation is $$\mathbf{V = \left[\frac{1.49\,(r)^{2/3}\,(s)^{1/2}}{n}\right]} \qquad \textbf{(2.17)}$$

Where: V = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of the hydraulic grade line (ft/ft)
n = Manning's roughness coefficient for open channel flow

After average velocity is computed using equation 2.17, T_t for the channel segment can be estimated using equation 2.7.

Velocity in channels should be calculated from the Manning's equation. Cross sections from all channels that have been field checked should be used in the calculations. This is particularly true of areas below dams or other flow control structures.

2.6.5.6 Reservoirs and Lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero. If the travel time through the reservoir or lake is important to the analysis then the hydrograph should be routed through the storage facility. A reservoir can have an impact in reducing peak flows which can be accounted for by routing.

2.6.5.7 Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet in undeveloped areas and 100 feet in developed areas.

- For storm conveyance systems, carefully identify the appropriate hydraulic flow path to estimate t_c.

APPENDIX 2A
IMPERVIOUS AREA CALCULATIONS

2A.1 Urban Modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas. For example, consider whether the impervious areas connect directly to the drainage system, or to lawns or other pervious areas where infiltration can occur.

The curve number values given in Table 2-12 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system.

It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system. The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

Connected Impervious Areas

Urban CN's given in Table 2-12 were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

(a) pervious urban areas are equivalent to pasture in good hydrologic condition, and

(b) impervious areas have a CN of 98 and are directly connected to the drainage system.

Some assumed percentages of impervious area are shown in Table 2-12.

If all the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2-12 are not applicable, use figure 2A-1 to compute composite CN. For example, Table 2-12 gives a CN of 70 for a ½ acre lot in hydrologic soil, group B, with an assumed impervious area of 25 percent. However, if the lot has 20 percent impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2A-1 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

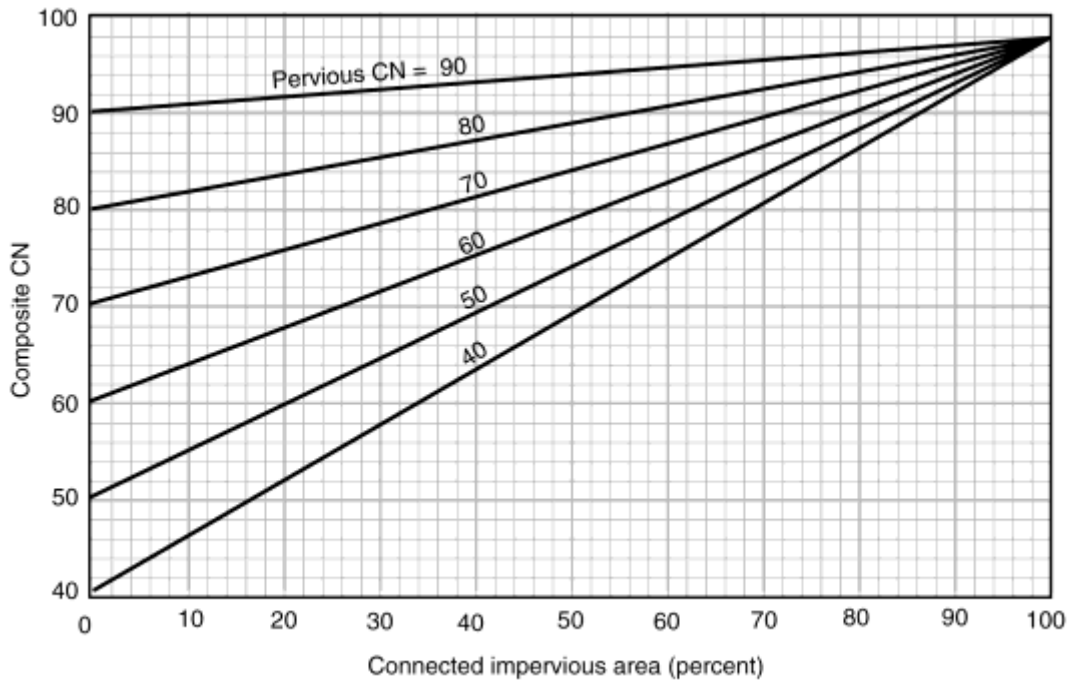


Figure 2A-1 Composite CN with Connected Impervious Area

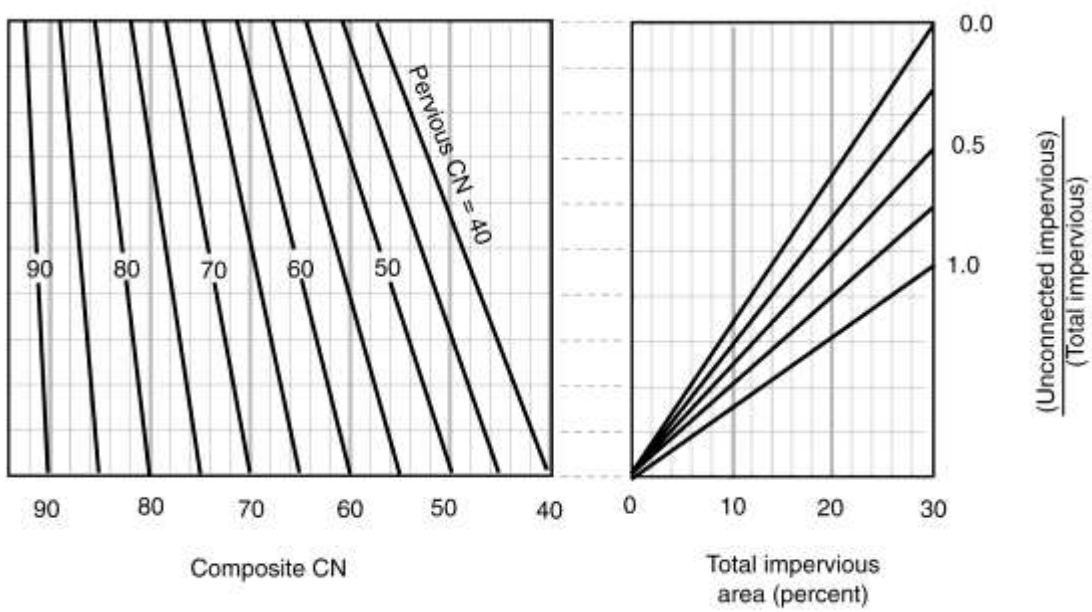


Figure 2A-2 Composite CN With Unconnected Impervious Area
(Total Impervious Area Less than 30%)
Source: ftp://ftp.wcc.nrcs.usda.gov/downloads/hydrology_hydraulicstr55/tr55.pdf

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1)

use Figure 2A-2 if total impervious area is less than 30 percent or (2) use Figure 2A-1 if the total impervious area is equal to or greater than 30 percent, because the absorptive capacity of the remaining pervious area will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of Figure 2A-2 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a ½ acre lot with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from Figure 2A-2 is 66. If all of the impervious area is connected, the resulting CN (from Figure 2A-1) would be 68.

2A.2 Composite Curve Numbers

When a drainage area has more than one land use, a weighted composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but rather sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by entering the required data into a table such as the example presented in Table 2A-1.

Table 2A-1
Composite Curve Numbers

Acreage	Land Use	Soil Type	Hydrologic Group	CN	Weighted CN (Acreage/Total Area) x (CN)
3.41	Pavement and Buildings	CuB/CeD2	B	98	41.10
1.70	Pavement and Buildings	EnD	C	98	20.49
0.65	Open Space—Good Condition	CeD2	B	61	4.88
0.78	Open Space—Good Condition	WuD	C	74	7.10
0.57	Woods—Good Condition	CeD2	B	55	3.86
1.02	Woods—Good Condition	EnD/WuD	C	70	8.78
8.13				CN_{post}=	86.21

The different land uses within the basin should represent a uniform hydrologic group represented by a single curve number. Any number of land uses can be included but if their spatial distribution is important to the hydrologic analysis the sub-basins should be developed and separate hydrographs developed and routed to the design point location.

APPENDIX 2B
ACCUMULATED PRECIPITATION DATA

2-Year, 6-Hour Storm
1 Minute Time Increment

Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)
0	0.0014	36	0.0576	72	0.1260	108	0.2214	144	0.3821
1	0.0029	37	0.0594	73	0.1282	109	0.2246	145	0.3882
2	0.0043	38	0.0610	74	0.1303	110	0.2279	146	0.3944
3	0.0058	39	0.0627	75	0.1325	111	0.2313	147	0.4007
4	0.0072	40	0.0645	76	0.1347	112	0.2347	148	0.4072
5	0.0087	41	0.0662	77	0.1369	113	0.2381	149	0.4138
6	0.0101	42	0.0680	78	0.1391	114	0.2416	150	0.4225
7	0.0116	43	0.0698	79	0.1413	115	0.2451	151	0.4313
8	0.0131	44	0.0715	80	0.1436	116	0.2486	152	0.4403
9	0.0146	45	0.0733	81	0.1459	117	0.2522	153	0.4495
10	0.0161	46	0.0751	82	0.1482	118	0.2558	154	0.4590
11	0.0176	47	0.0769	83	0.1505	119	0.2595	155	0.4687
12	0.0191	48	0.0787	84	0.1528	120	0.2636	156	0.4787
13	0.0206	49	0.0806	85	0.1552	121	0.2678	157	0.4889
14	0.0221	50	0.0824	86	0.1575	122	0.2720	158	0.4995
15	0.0237	51	0.0843	87	0.1599	123	0.2763	159	0.5104
16	0.0252	52	0.0861	88	0.1623	124	0.2806	160	0.5216
17	0.0267	53	0.0880	89	0.1647	125	0.2850	161	0.5332
18	0.0283	54	0.0899	90	0.1675	126	0.2894	162	0.5452
19	0.0299	55	0.0918	91	0.1703	127	0.2939	163	0.5577
20	0.0314	56	0.0937	92	0.1731	128	0.2985	164	0.5707
21	0.0330	57	0.0956	93	0.1759	129	0.3031	165	0.5879
22	0.0346	58	0.0976	94	0.1787	130	0.3078	166	0.6057
23	0.0362	59	0.0995	95	0.1816	131	0.3125	167	0.6243
24	0.0378	60	0.1015	96	0.1845	132	0.3174	168	0.6438
25	0.0394	61	0.1034	97	0.1874	133	0.3223	169	0.6642
26	0.0410	62	0.1054	98	0.1903	134	0.3273	170	0.6858
27	0.0426	63	0.1074	99	0.1933	135	0.3323	171	0.7086
28	0.0442	64	0.1094	100	0.1963	136	0.3375	172	0.7331
29	0.0459	65	0.1114	101	0.1993	137	0.3427	173	0.7655
30	0.0475	66	0.1135	102	0.2024	138	0.3480	174	0.8002
31	0.0492	67	0.1155	103	0.2055	139	0.3534	175	0.8444
32	0.0508	68	0.1176	104	0.2086	140	0.3590	176	0.8924
33	0.0525	69	0.1197	105	0.2117	141	0.3646	177	0.9458
34	0.0542	70	0.1218	106	0.2149	142	0.3703	178	1.0298
35	0.0559	71	0.1239	107	0.2181	143	0.3761	179	1.1138

Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)
180	1.1978	216	1.9078	252	2.0644	288	2.1579	325	2.2274
181	1.2818	217	1.9136	253	2.0676	289	2.1600	326	2.2290
182	1.3658	218	1.9192	254	2.0707	290	2.1621	327	2.2307
183	1.4162	219	1.9249	255	2.0739	291	2.1641	328	2.2324
184	1.4621	220	1.9303	256	2.0770	292	2.1662	329	2.2340
185	1.4983	221	1.9357	257	2.0800	293	2.1682	330	2.2357
186	1.5318	222	1.9410	258	2.0831	294	2.1703	331	2.2373
187	1.5631	223	1.9462	259	2.0861	295	2.1723	332	2.2389
188	1.5867	224	1.9513	260	2.0891	296	2.1743	333	2.2405
189	1.6089	225	1.9563	261	2.0928	297	2.1763	334	2.2421
190	1.6299	226	1.9612	262	2.0949	298	2.1783	335	2.2437
191	1.6498	227	1.9661	263	2.0979	299	2.1802	336	2.2453
192	1.6688	228	1.9709	264	2.1007	300	2.1822	337	2.2469
193	1.6870	229	1.9756	265	2.1036	301	2.1841	338	2.2485
194	1.7045	230	1.9803	266	2.1064	302	2.1861	339	2.2501
195	1.7178	231	1.9849	267	2.1092	303	2.1880	340	2.2516
196	1.7305	232	1.9894	268	2.1120	304	2.1899	341	2.2532
197	1.7428	233	1.9939	269	2.1147	305	2.1918	342	2.2547
198	1.7546	234	1.9983	270	2.1172	306	2.1936	343	2.2563
199	1.7660	235	2.0026	271	2.1196	307	2.1955	344	2.2578
200	1.7771	236	2.0069	272	2.1220	308	2.1974	345	2.2593
201	1.7878	237	2.0112	273	2.1244	309	2.1992	346	2.2609
202	1.7982	238	2.0154	274	2.1267	310	2.2011	347	2.2624
203	1.8083	239	2.0195	275	2.1291	311	2.2029	348	2.2639
204	1.8181	240	2.0232	276	2.1314	312	2.2047	349	2.2654
205	1.8273	241	2.0269	277	2.1337	313	2.2065	350	2.2669
206	1.8371	242	2.0305	278	2.1360	314	2.2083	351	2.2684
207	1.8462	243	2.0340	279	2.1382	315	2.2101	352	2.2698
208	1.8551	244	2.0375	280	2.1405	316	2.2118	353	2.2713
209	1.8638	245	2.0410	281	2.1427	317	2.2136	354	2.2728
210	1.8706	246	2.0445	282	2.1449	318	2.2154	355	2.2742
211	1.8771	247	2.0479	283	2.1471	319	2.2171	356	2.5757
212	1.8835	248	2.0512	284	2.1493	320	2.2188	357	2.2771
213	1.8898	249	2.0546	285	2.1515	321	2.2206	358	2.2786
214	1.8959	250	2.0579	286	2.1536	322	2.2223	359	2.2800
215	1.9020	251	2.0611	287	2.1558	323	2.2240	360	2.2800
						324	2.2257		

10-Year, 6-Hour Storm
1 Minute Time Increment

Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)
0	0.0022	36	0.0887	72	0.1945	108	0.3469	144	0.6230
1	0.0044	37	0.0914	73	0.1976	109	0.3523	145	0.6336
2	0.0066	38	0.0940	74	0.2012	110	0.3577	146	0.6444
3	0.0089	39	0.0967	75	0.2046	111	0.3632	147	0.6554
4	0.0111	40	0.0994	76	0.2080	112	0.3687	148	0.6666
5	0.0134	41	0.1021	77	0.2114	113	0.3743	149	0.6781
6	0.0156	42	0.1048	78	0.2148	114	0.3799	150	0.6936
7	0.0179	43	0.1075	79	0.2183	115	0.3856	151	0.7094
8	0.0202	44	0.1102	80	0.2218	116	0.3914	152	0.7255
9	0.0225	45	0.1130	81	0.2253	117	0.3973	153	0.7420
10	0.0248	46	0.1158	82	0.2289	118	0.4032	154	0.7589
11	0.0271	47	0.1186	83	0.2325	119	0.4092	155	0.7763
12	0.0294	48	0.1214	84	0.2361	120	0.4164	156	0.7940
13	0.0317	49	0.1242	85	0.2397	121	0.4237	157	0.8122
14	0.0341	50	0.1271	86	0.2434	122	0.4311	158	0.8310
15	0.0364	51	0.1299	87	0.2471	123	0.4386	159	0.8502
16	0.0388	52	0.1328	88	0.2508	124	0.4461	160	0.8701
17	0.0412	53	0.1357	89	0.2546	125	0.4538	161	0.8906
18	0.0436	54	0.1386	90	0.2591	126	0.4615	162	0.9118
19	0.0460	55	0.1416	91	0.2636	127	0.4694	163	0.9338
20	0.0484	56	0.1445	92	0.2681	128	0.4774	164	0.9566
21	0.0508	57	0.1475	93	0.2727	129	0.4855	165	0.9876
22	0.0533	58	0.1505	94	0.2774	130	0.4936	166	1.0198
23	0.0557	59	0.1535	95	0.2820	131	0.5020	167	1.0531
24	0.0582	60	0.1565	96	0.2868	132	0.5104	168	1.0878
25	0.0606	61	0.1596	97	0.2915	133	0.5190	169	1.1240
26	0.0631	62	0.1627	98	0.2963	134	0.5276	170	1.1620
27	0.0636	63	0.1657	99	0.3012	135	0.5365	171	1.2021
28	0.0681	64	0.1689	100	0.3060	136	0.5454	172	1.2445
29	0.0707	65	0.1720	101	0.3110	137	0.5546	173	1.2976
30	0.0732	66	0.1751	102	0.3160	138	0.5638	174	1.3542
31	0.0758	67	0.1783	103	0.3210	139	0.5733	175	1.4281
32	0.0783	68	0.1815	104	0.3261	140	0.5829	176	1.5067
33	0.0809	69	0.1847	105	0.3312	141	0.5926	177	1.5919
34	0.0835	70	0.1880	106	0.3364	142	0.6026	178	1.7099
35	0.0861	71	0.1913	107	0.3416	143	0.6127	179	1.8279

Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)
180	1.9459	216	3.1144	252	3.3824	288	2.1579	325	2.2274
181	2.0639	217	3.1244	253	3.3876	289	2.1600	326	2.2290
182	2.1819	218	3.1343	254	3.3928	290	2.1621	327	2.2307
183	2.2635	219	3.1440	255	3.3979	291	2.1641	328	2.2324
184	2.3395	220	3.1535	256	3.4030	292	2.1662	329	2.2340
185	2.3982	221	3.1628	257	3.4080	293	2.1682	330	2.2357
186	2.4529	222	3.1720	258	3.4129	294	2.1703	331	2.2373
187	2.5045	223	3.1811	259	3.4178	295	2.1723	332	2.2389
188	2.5458	224	3.1900	260	3.4227	296	2.1743	333	2.2405
189	2.5847	225	3.1987	261	3.4275	297	2.1763	334	2.2421
190	2.6218	226	3.2073	262	3.4323	298	2.1783	335	2.2437
191	2.6575	227	3.2158	263	3.4370	299	2.1802	336	2.2453
192	2.6912	228	3.2242	264	3.4417	300	2.1822	337	2.2469
193	2.7240	229	3.2325	265	3.4464	301	2.1841	338	2.2485
194	2.7556	230	3.2406	266	3.4510	302	2.1861	339	2.2501
195	2.7788	231	3.2486	267	3.4556	303	2.1880	340	2.2516
196	2.8012	232	3.2565	268	3.4601	304	2.1899	341	2.2532
197	2.8227	233	3.2644	269	3.4646	305	2.1918	342	2.2547
198	2.8436	234	3.2721	270	3.4684	306	2.1936	343	2.2563
199	2.8638	235	3.2797	271	3.4721	307	2.1955	344	2.2578
200	2.8833	236	3.2872	272	3.4759	308	2.1974	345	2.2593
201	2.9023	237	3.2946	273	3.4795	309	2.1992	346	2.2609
202	2.9208	238	3.3020	274	3.4832	310	2.2011	347	2.2624
203	2.9388	239	3.3092	275	3.4868	311	2.2029	348	2.2639
204	2.9563	240	3.3152	276	3.4904	312	2.2047	349	2.2654
205	2.9734	241	3.3212	277	3.4940	313	2.2065	350	2.2669
206	2.9901	242	3.3271	278	3.4975	314	2.2083	351	2.2684
207	3.0065	243	3.3329	279	3.5011	315	2.2101	352	2.2698
208	3.0224	244	3.3387	280	3.5045	316	2.2118	353	2.2713
209	3.0381	245	3.3443	281	3.5080	317	2.2136	354	2.2728
210	3.0497	246	3.3499	282	3.5114	318	2.2154	355	2.2742
211	3.0611	247	3.3555	283	3.5148	319	2.2171	356	2.5757
212	3.0722	248	3.3610	284	3.5182	320	2.2188	357	2.2771
213	3.0831	249	3.3665	285	3.5216	321	2.2206	358	2.2786
214	3.0937	250	3.3718	286	3.5249	322	2.2223	359	2.2800
215	3.1041	251	3.3772	287	3.5282	323	2.2240	360	2.2800
						324	2.2257		

50-Year, 6-Hour Storm
1 Minute Time Increment

Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)
0	0.0031	36	0.1257	72	0.2750	108	0.4847	144	0.8583
1	0.0063	37	0.1295	73	0.2797	109	0.4919	145	0.8725
2	0.0094	38	0.1332	74	0.2844	110	0.4992	146	0.8870
3	0.0126	39	0.1370	75	0.2891	111	0.5066	147	0.9018
4	0.0158	40	0.1408	76	0.2939	112	0.5141	148	0.9170
5	0.0189	41	0.1446	77	0.2987	113	0.5217	149	0.9324
6	0.0221	42	0.1484	78	0.3036	114	0.5293	150	0.9526
7	0.0254	43	0.1523	79	0.3084	115	0.5370	151	0.9733
8	0.0286	44	0.1562	80	0.3134	116	0.5449	152	0.9944
9	0.0319	45	0.1601	81	0.3183	117	0.5528	153	1.0159
10	0.0351	46	0.1640	82	0.3233	118	0.5608	154	1.0380
11	0.0384	47	0.1679	83	0.3283	119	0.5689	155	1.0606
12	0.0417	48	0.1719	84	0.3334	120	0.5787	156	1.0838
13	0.0450	49	0.1759	85	0.3385	121	0.5886	157	1.1076
14	0.0483	50	0.1799	86	0.3437	122	0.5986	158	1.1320
15	0.0517	51	0.1839	87	0.3489	123	0.6087	159	1.1571
16	0.0550	52	0.1880	88	0.3542	124	0.6190	160	1.1830
17	0.0584	53	0.1921	89	0.3594	125	0.6294	161	1.2098
18	0.0618	54	0.1962	90	0.3655	126	0.6399	162	1.2374
19	0.0652	55	0.2004	91	0.3717	127	0.6506	163	1.2660
20	0.0686	56	0.2045	92	0.3778	128	0.6614	164	1.2957
21	0.0720	57	0.2087	93	0.3841	129	0.6723	165	1.3363
22	0.0755	58	0.2129	94	0.3903	130	0.6834	166	1.3783
23	0.0790	59	0.2172	95	0.3967	131	0.6947	167	1.4219
24	0.0825	60	0.2215	96	0.4031	132	0.7061	168	1.4672
25	0.0860	61	0.2258	97	0.4095	133	0.7177	169	1.5145
26	0.0895	62	0.2301	98	0.4160	134	0.7294	170	1.5640
27	0.0930	63	0.2344	99	0.4226	135	0.7414	171	1.6161
28	0.0966	64	0.2388	100	0.4292	136	0.7535	172	1.6714
29	0.1002	65	0.2432	101	0.4359	137	0.7658	173	1.7400
30	0.1038	66	0.2477	102	0.4427	138	0.7783	174	1.8130
31	0.1074	67	0.2522	103	0.4495	139	0.7911	175	1.9086
32	0.1110	68	0.2567	104	0.4564	140	0.8040	176	2.0101
33	0.1147	69	0.2612	105	0.4634	141	0.8172	177	2.1198
34	0.1183	70	0.2658	106	0.4704	142	0.8307	178	2.2698
35	0.1220	71	0.2704	107	0.4775	143	0.8443	179	2.4198

Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)	Time (Min.)	Accumulated Rainfall (in.)
180	2.5698	216	4.0852	252	4.4480	288	4.6535	325	4.8051
181	2.7198	217	4.0988	253	4.4551	289	4.6581	326	4.8087
182	2.8698	218	4.1121	254	4.4621	290	4.6626	327	4.8124
183	2.9751	219	4.1252	255	4.4690	291	4.6671	328	4.8160
184	3.0734	220	4.1380	256	4.4758	292	4.6716	329	4.8196
185	3.1490	221	4.1507	257	4.4826	293	4.6761	330	4.8232
186	3.2197	222	4.1631	258	4.4893	294	4.6805	331	4.8268
187	3.2864	223	4.1753	259	4.4960	295	4.6849	332	4.8303
188	3.3401	224	4.1873	260	4.5026	296	4.6893	333	4.8338
189	3.3908	225	4.1992	261	4.5092	297	4.6936	334	4.8374
190	3.4392	226	4.2109	262	4.5156	298	4.6980	335	4.8409
191	3.4854	227	4.2224	263	4.5221	299	4.7022	336	4.8443
192	3.5298	228	4.2337	264	4.5284	300	4.7065	337	4.8478
193	3.5726	229	4.2449	265	4.5347	301	4.7107	338	4.8512
194	3.6139	230	4.2559	266	4.5410	302	4.7149	339	4.8547
195	3.6442	231	4.2668	267	4.5472	303	4.7191	340	4.8581
196	3.6733	232	4.2775	268	4.5533	304	4.7233	341	4.8615
197	3.7014	233	4.2881	269	4.5594	305	4.7274	342	4.8649
198	3.7286	234	4.2985	270	4.5648	306	4.7315	343	4.8682
199	3.7549	235	4.3089	271	4.5700	307	4.7356	344	4.8716
200	3.7804	236	4.3191	272	4.5753	308	4.7396	345	4.8749
201	3.8052	237	4.3291	273	4.5804	309	4.7437	346	4.8782
202	3.8293	238	4.3391	274	4.5856	310	4.7477	347	4.8815
203	3.8528	239	4.3489	275	4.5907	311	4.7516	348	4.8848
204	3.8757	240	4.3571	276	4.5957	312	4.7556	349	4.8881
205	3.8980	241	4.3651	277	4.6008	313	4.7595	350	4.8913
206	3.9198	242	4.3731	278	4.6057	314	4.7635	351	4.8946
207	3.9411	243	4.3810	279	4.6107	315	4.7673	352	4.8978
208	3.9620	244	4.3888	280	4.6156	316	4.7712	353	4.9010
209	3.9824	245	4.3965	281	4.6204	317	4.7751	354	4.9042
210	3.9981	246	4.4041	282	4.6253	318	4.7789	355	4.9074
211	4.0134	247	4.4116	283	4.6301	319	4.7827	356	4.9106
212	4.0283	248	4.4190	284	4.6348	320	4.7865	357	4.9137
213	4.0430	249	4.4264	285	4.6395	321	4.7902	358	4.9169
214	4.0574	250	4.4337	286	4.6442	322	4.7940	359	4.9200
215	4.0714	251	4.4409	287	4.6489	323	4.7977	360	4.9200
						324	4.8014		

CHAPTER 3

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3.1 OVERVIEW

3.1.1 Introduction

This chapter emphasizes procedures for performing uniform calculations that aid in the selection or evaluation of appropriate channel linings, depths, and grades for natural or man-made channels. Allowable velocities are provided, along with procedures for evaluating channel capacity using Manning's equation. For information on roadside ditch requirements see Chapter 4.

3.1.2 Channel Linings

The three main classifications of open channel linings are vegetative, flexible and rigid. Vegetative linings include grass with mulch, sod, and lapped sod. Rock riprap is a flexible lining, while rigid linings are generally concrete.

3.1.2.1 Vegetative

Vegetation is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, inhibits erosion on the channel surface, and controls the movement of soil particles along the channel bottom. Conditions under which vegetation may not be acceptable; however, include but are not limited to:

1. Flow conditions in excess of the maximum shear stress for bare soils;
2. Standing or continuous flowing water;
3. Lack of regular maintenance necessary to prevent domination by taller vegetation;
4. Lack of nutrients and inadequate topsoil;
5. Excessive shade; or
6. Excessive velocities

Proper seeding, mulching, and soil preparation are required during construction to ensure establishment of a healthy stand of grass. Soil testing may be performed and the results evaluated by an agronomist to determine soil treatment requirements for pH, nitrogen, phosphorus, potassium, and other factors. In many cases, temporary erosion control measures are required to provide time for the seeding to establish a viable vegetative lining.

Sodding should be staggered, to avoid continuous seams in the direction of flow. Lapped or shingle sod should be staggered and overlapped by approximately 25 percent. Staked sod is usually only necessary for use on steeper slopes to prevent sliding.

3.1.2.2 Flexible

Rock riprap including rubble is the most common type of flexible lining. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. They typically require use of filter fabric and allow the infiltration and exfiltration of water. The growth of grass and weeds through the lining may present maintenance problems. The use of flexible lining may be restricted where space is limited, since the introduction of linings with higher roughness values generally results in the need for greater channel capacity.

3.1.2.3 Rigid

Rigid linings are generally constructed of concrete and used where smoothness offers a higher capacity for a given cross-sectional area. Higher velocities, however, create the potential for scour

at channel lining transitions. A rigid lining can be damaged by flow undercutting the lining, channel headcutting, or the buildup of hydrostatic pressure behind the rigid surfaces. When properly designed, rigid linings may be appropriate where the channel width is restricted. Filter fabric may be required to prevent soil loss through pavement cracks.

Under continuous base flow conditions when a vegetative lining alone would be appropriate, a small concrete pilot channel could be used to convey the continuous low flows. Vegetation could then be maintained for conveying larger flows.

3.2 DESIGN CRITERIA

3.2.1 General Criteria

The following criteria shall be used for open channel design:

1. Channel side slopes shall be stable throughout the entire length and slope shall be a maximum of 2:1.
2. Superelevation of the water surface at horizontal curves shall be accounted for by increased freeboard.
3. A minimum freeboard of 6” must be provided in the 10-year design storm.
4. Transition from closed systems to channel sections (or between transitioning channel sections) shall be smooth and gradual, with a minimum of 5:1 taper.
5. Low flow sections shall be considered in the design of channels with large cross-sections (Q > 100 cfs). Some channel designs will be required to have increased freeboard (see North Carolina Erosion and Sediment Control Planning and Design Manual, Section 8.05.21).

3.2.2 Return Period

Open channel drainage systems shall be designed to convey a 10-year design storm. The peak flow rate for the 100-year storm shall be computed at appropriate points within the drainage system to determine if a 100 + 1 flood study is required as described in section 3.2.3.

Sediment transport requirements must be considered for conditions or flow below the design frequency. A low flow channel component within a larger channel can reduce maintenance by improving sediment transport in the channel.

3.2.3 100+1 Flood Analysis

All streams in the City of Charlotte and Mecklenburg County which drain more than one square mile (640 acres) are regulated by Floodplain Ordinances, which restrict development in those flood plains. However, storm water conveyances that drain less than one square mile will also flood and require regulation as well. This regulation is known as the 100+1 flood analysis.

The following criteria will be used to determine how and when the 100+1 flood analysis will be used.

1. The 100+1 analysis will be required for all portions of the drainage system which are expected to carry 50 cfs or more for the 100-year storm.

2. The 100-year storm water surface elevations should be calculated using a method acceptable to the City/County Engineering Department, as further described in Section 3.6.
3. The peak flow rate used in the 100+1 analysis shall be based on an assumption of full build out of the tributary drainage area. The assumption of full build out shall be defined as either full development per the current zoning of the property, the existing land use, or adopted area land use plans, whichever generates the higher runoff rate.
4. For drainage systems within development projects subject to the Subdivision Ordinance, the 100+1 elevation and flood limits shall be shown on the recorded maps associated with the subdivision as further described in the Subdivision Ordinance.
5. For drainage systems within development projects not subject to the Subdivision Ordinance, the City/County Engineering Department may require that the 100+1 elevation be shown on a recorded map if the engineering analysis indicates that one of the following conditions is present:
    - The 100+1 line would exceed the set-back limits.
    - The estimated runoff or proposed modifications to a storm water conveyance would create a hazard for the adjacent properties or residents.
    - The flood limits would be of such magnitude that adjacent property owners should be informed of these limits.

3.2.4 Velocity Limitations

The design of open channels should be consistent with the velocity limitations for the selected channel lining. For design information see section 3.4 Open Channel Design.

3.3 MANNING'S N VALUES

3.3.1 General Considerations

The Manning's *n* value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth and velocity estimates. Since Manning's *n* values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

3.3.2 Selection

The following general factors should be considered when selecting the value of Manning's *n*:

1. The physical roughness of the bottom and sides of the channel. Fine particle soils on smooth, uniform surfaces result in relatively low values of *n*. Coarse materials such as gravel or boulders, and pronounced surface irregularity cause higher values of *n*.
2. The value of *n* depends on the height, density, and type of vegetation and how the vegetation affects the flow through the channel reach.
3. Channel dimension variations, such as abrupt changes in channel cross sections or alternating small and large sections, will require somewhat larger *n* values than normal. These variations in channel cross section become particularly important if they cause the flow to meander from side to side within the channel.
4. A significant increase in the value of *n* is possible if severe meandering occurs in the alignment of a channel. Meandering becomes particularly important when frequent changes in the direction of curvature occur with relatively small radii of curvature.

5. Active channel erosion or sedimentation (degradation and aggradation) will tend to increase the value of *n*, since these processes may cause variations in the shape of a channel. The potential for future erosion or sedimentation in the channel must also be considered.
6. Obstructions such as log jams or deposits of debris will increase the value of *n*. The level of this increase will depend on the number, type and size of obstructions.
7. To be conservative, it is better to use a higher resistance for capacity calculations and a lower resistance for stability calculations.
8. Proper assessment of natural channel *n* values requires field observations and experience. Special attention is required in the field to identify flood plain vegetation and evaluate possible variations in roughness with depth of flow.

All of these factors should be evaluated with respect to type of channel, degree of maintenance, seasonal requirements, and other considerations as a basis for determining appropriate design *n* values. The probable condition of the channel when the design event is anticipated should be considered. Values representative of a newly constructed channel are rarely appropriate as a basis for design capacity calculations.

3.3.3 Manning's *n* Values

Recommended Manning's *n* values for natural channels are given in Table 3-1 and recommended Manning's *n* values for artificial channels are given in Table 3-2.

Table 3-1
Recommended Manning's *n* Values for Natural Channels

Type of channel and description	Minimum	Normal	Maximum
NATURAL LININGS			
Minor Streams (top width at flood stage < 100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
Flood Plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crop	0.025	0.035	0.045
3. Mature field crop	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160

Abridged from Chow, V.T., ed. 1959, Open-Channel Hydraulics

Table 3-2
Recommended Manning's *n* Values for Artificial Channels

RIP RAP	*n* (depth of flow >=2')
Class B stone	0.037
Class 1 rip rap	0.040
Class 2 rip rap	0.045

Note: See Table 3-7 for temporary lining materials such as straw matting. If using values other than ones listed above or in Table 3-7, please provide documentation.

3.4 OPEN CHANNEL DESIGN

3.4.1 Overview

This section addresses the design of stable conveyance channels using vegetative and flexible linings. A stable channel is defined as a channel which is non-silting and non-scouring. To minimize silting in the channel, flow velocities should remain constant or increase slightly throughout the channel length.

Vegetative and flexible channel linings are generally preferred to rigid linings from an erosion control perspective because they conform to changes in channel shape without failure and are less susceptible to damage from frost heaving, soil swelling and shrinking, and excessive soil pore water pressure from lack of drainage. Vegetative and flexible linings also are generally less expensive to construct, and generally are more natural in appearance. On the other hand, vegetative and flexible linings generally have higher roughness coefficients and may require a larger cross section for the same discharge.

3.4.2 Design of Stable Channels

The minimum design criteria for conveyance channels require that two primary conditions be satisfied: the channel system must have capacity for the peak flow expected from the 10-year storm and the channel lining must be resistant to erosion for the design velocity. In some cases for intermittent and perennial systems, out-of-bank flow may be considered a functional part of the channel system. In those cases, flow capacities and design velocities should be considered separately for out-of-bank flows and channel flows. Design methodology for these larger more natural stream systems is not included in this manual.

Both the capacity of the channel and the velocity of flow are functions of the channel lining, cross-sectional area and slope. The channel system must carry the design flow with adequate freeboard, fit site conditions and be stable.

The following procedures are needed for designing stable channels: (1) the permissible velocity procedure (for permanent vegetative linings); and (2) the tractive force procedure (for temporary lining materials or riprap linings). Under the permissible velocity procedure, the channel is considered stable if the design mean velocity is lower than the maximum permissible velocity. Under the tractive force procedure, erosive stress evaluated at the boundary between flowing water and lining materials must be less than the minimum unit tractive force that will cause serious erosion of material from a level channel bed.

3.4.3 Permissible Velocity

The permissible velocity procedure uses two equations to calculate flow:

Manning's equation,

$$\mathbf{V = \frac{1.49\, R^{2/3}\, S^{1/2}}{n}} \qquad \textbf{(3.1)}$$

Where: V = average velocity in the channel (ft/sec)

n = Manning's roughness coefficient, based upon the lining of the channel

R = hydraulic radius, wetted cross-sectional area/wetted perimeter (ft)

S = slope of the channel (ft/ft)

And the continuity equation,

$$\mathbf{Q = AV} \qquad \textbf{(3.2)}$$

Where: Q = flow in the channel (cfs)

A = cross-sectional area of flow within the channel (square feet - ft^2)

V = average velocity in the channel (ft/sec)

Manning's equation and the continuity equation are used together to determine channel capacity and flow velocity.

3.4.4 Selecting Permanent Channel Lining

The design of concrete and similar rigid linings is generally not restricted by flow velocities. However, vegetative and flexible channel linings do have maximum permissible flow velocities beyond which they are susceptible to erosion. The designer should select the type of liner that best fits site conditions.

Table 3-3 lists maximum permissible velocities for established grass linings and soil conditions. Before grass is established, permissible velocity is determined by the choice of temporary liner. Permissible velocities for riprap linings are higher than for grass and depend on the stone size selected.

Table 3-3
Maximum Allowable Design Velocities[1]
for Vegetated Channels

Typical Channel Slope Application	Soil Characteristics[2]	Grass Lining	Permissible Velocity[3] for Established Grass Lining (ft/sec)
0-5%	Easily Erodible	Bermudagrass	5.0
	Non-plastic	Tall fescue	4.5
	(Sands & Silts)	Bahiagrass	4.5
		Kentucky bluegrass	4.5
		Grass-legume mixture	3.5
	Erosion Resistant	Bermudagrass	6.0
	Plastic	Tall fescue	5.5
	(Clay mixes)	Bahiagrass	5.5
		Kentucky bluegrass	5.5
		Grass-legume mixture	4.5
5-10%	Easily Erodible	Bermudagrass	4.5
	Non-plastic	Tall fescue	4.0
	(Sands & Silts)	Bahiagrass	4.0
		Kentucky bluegrass	4.0
		Grass-legume mixture	3.0
	Erosion Resistant	Bermudagrass	5.5
	Plastic	Tall fescue	5.0
	(Clay mixes)	Bahiagrass	5.0
		Kentucky bluegrass	5.0
		Grass-legume mixture	3.5
>10%	Easily Erodible	Bermudagrass	3.5
	Non-plastic	Tall fescue	2.5
	(Sands & Silts)	Bahiagrass	2.5
		Kentucky bluegrass	2.5
	Erosion Resistant	Bermudagrass	4.5
	Plastic	Tall fescue	3.5
	(Clay mixes)	Bahiagrass	3.5
		Kentucky bluegrass	3.5

Source: USDA-SCS Modified

NOTE: 1 Permissible Velocity based on 10-year storm peak runoff
2 Soil erodibility based on resistance to soil movement from concentrated flowing water (soil types are based on NRCS soils data).
3 Before grass is established, permissible velocity is determined by the type of temporary liner used.

3.4.5 Tractive Force

This procedure is for uniform flow in channels and is ***not*** to be used for design of deenergizing devices and may not be valid for larger channels.

To calculate the required size of an open channel, assume the design flow is uniform and does not vary with time. Since actual flow conditions change through the length of a channel, subdivide the channel into design reaches as appropriate.

The permissible shear stress, T_d, is the force required to initiate movement of the lining material. Permissible shear stress for the liner is not related to the erodibility of the underlying soil. However, if the lining is eroded or broken, the bed material will be exposed to the erosive force of the flow.

Shear stress, T, at normal depth is computed for the lining by the following equation:

$$\mathbf{T = \omega ds} \qquad \textbf{(3.3)}$$

$$T_d > T$$

Where: T = computated shear stress (in lb/ft^2)

T_d = Permissible shear stress (Table 3-8)

ω = unit weight of water, 62.4 lb/ft^3

d = flow depth (ft)

s = channel gradient (ft/ft)

If the permissible shear stress, T_d, is greater than the computed shear stress, the riprap or temporary lining is considered acceptable. If a lining is unacceptable, select a lining with a higher permissible shear stress and repeat the calculations for normal depth and shear stress. In some cases it may be necessary to alter channel dimensions to reduce the shear stress.

Computing tractive force around a channel bend requires special considerations because the change in flow direction imposes higher shear stress on the channel bottom and banks. The maximum shear stress in a bend, T_b, is given by the following equation:

$$\mathbf{T_b = KbT} \qquad \textbf{(3.4)}$$

Where: T_b = bend shear stress (lb/ft^2)

K_b = bend factor

T = computed stress for straight channel (lb/ft^2)

The value of K_b is related to the radius of curvature of the channel at its centerline, R_c, and the bottom width of the channel, B, Figure 3- 2. The length of channel requiring protection downstream from a bend, L_p, is a function of the roughness of the lining material and the hydraulic radius as shown in Figure 3-3.

3.4.6 Design Procedure Using Permissible Velocity and Tractive Force Procedures

The following is a step-by-step procedure for designing a runoff conveyance channel using Manning's equation and the continuity equation:

Step 1. Determine the required flow capacity, Q, by estimating peak runoff rate for the design storm.

Step 2. Determine the slope and select channel geometry and lining.

Step 3. Determine the permissible velocity for the lining selected, or the desired velocity, if paved (see Table 3-3).

Step 4. Make an initial estimate of channel size - divide the required Q by the permissible velocity to reach a "first try" estimate of channel flow area. Then select a geometry, depth, and top width to fit site conditions.

Step 5. Calculate the hydraulic radius, R, from channel geometry (Figure D-1, Appendix 3D).

Step 6. Determine roughness coefficient *n*.

Structural Linings - see Table 3-4.

Grass Lining:

a. Determine retardance class for vegetation from Table 3-5. To meet stability requirement, use retardance for newly mowed condition (generally C or D). To determine channel capacity, use at least one retardance class higher.

b. Determine *n* from Figure 3-1.

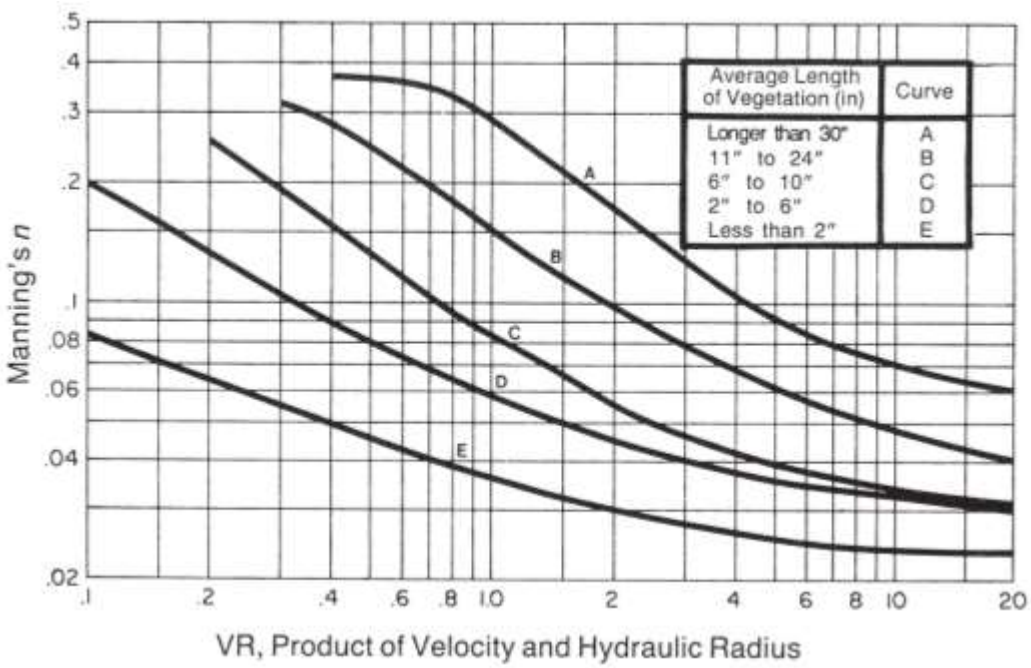


Figure 3-1 Manning's *n* Related to Velocity, Hydraulic Radius, and Vegetal Retardance
Source: www.dlr.enr.state.nc.us/images/Sediment_design_manual_June2006/ChapterEight_20070419.PDF, page 599

Step 7. Calculate the actual channel velocity, V, using Manning's equation (3.1) and calculate channel capacity, Q, using the continuity equation (3.2).

Step 8. Check results against permissible velocity and required design capacity to determine if design is acceptable.

Step 9. If design is not acceptable, alter channel dimensions as appropriate. For trapezoidal channels, this adjustment is usually made by changing the bottom width.

Step 10. For grass-lined channels once the appropriate channel dimensions have been selected for low retardance conditions, repeat steps 6 through 8 using a higher retardance class, corresponding to tall grass. This step is to verify that once the channel is fully established it is still designed appropriately. Adjust capacity of the channel by varying depth where site conditions permit.

> NOTE 1: If design velocity is greater than 2.0 ft/sec., a temporary lining may be required to stabilize the channel until vegetation is established. The temporary liner may be designed for peak flow from the 2-year storm. If a channel requires a temporary lining, the designer should analyze shear stresses in the channel to select the liner that provides protection and promotes establishment of vegetation. For the design of temporary liners, use the tractive force procedure.

Step 11. Check outlet for carrying capacity and stability. If discharge velocities exceed allowable velocities (Table 3-6) for the receiving stream, an outlet protection structure will be required. See Chapter 7 Energy Dissipation for design requirements.

Step 12. Add required 6-inch freeboard to design depth of channel cross section.

Step 13. Check to see if a temporary liner is necessary, as referenced in Note -1 of Step-10 above.

Step 14. Select a temporary liner material suitable for site conditions and application. Determine roughness coefficient from manufacturer's specifications or Table 3-7. Calculate the normal flow depth using Manning's equation (3.1). Check to see that calculated depth is consistent with that assumed for selection of Manning's roughness coefficient. The coefficient of roughness generally decreases with increasing flow depth.

Step 15. Calculate shear stress at normal depth.

Step 16. Compare computed shear stress with the permissible shear stress for the liner.

Step 17. If computed shear is greater than permissible shear, adjust channel dimensions to reduce shear, or select a more resistant lining and repeat steps 13 through 17.

Table 3-4
Manning's *n* Values for Structural Channel Linings

Channel Lining	Recommended *n* values
Asphaltic concrete, machine placed	0.014
Asphalt, exposed prefabricated	0.015
Concrete	0.015
Metal, corrugated	0.024
Plastic	0.013
Shotcrete	0.017
Gabion	0.030
Earth	0.020

Source: American Society of Civil Engineers (modified)

Table 3-5
Retardance Classification for Vegetal Covers

Retardance	Cover	Condition
A	Reed canarygrass	Excellent stand, tall (average 36”)
	Weeping lovegrass	Excellent stand, tall (average 30”)
B	Tall fescue	Good stand, uncut, (average 18”)
	Bermudagrass	Good stand, tall (average 12”)
	Grass-legume mixture (tall fescue,redfescue, sericealespedeza)	Good stand, uncut
	Grass mixture (timothy, smooth bromegrass ororchardgrass)	Good stand, uncut (average 20”)
	Sericea lespedeza	Good stand, not woody, tall (average 19”)
	Reed canarygrass	Good stand, cut, (average 12-15”)
	Alfalfa	Good stand, uncut (average 11”)
C	Tall fescue	Good stand (8-12”)
	Bermudagrass	Good stand, cut (average 6”)
	Bahiagrass	Good stand, uncut (6-8”)
	Grass-legume mixture--summer (orchardgrass,redtop and annuallespedeza)	Good stand, uncut (6-8”)
	Centipedegrass	Very dense cover (average 6”)
	Kentucky bluegrass	Good stand, headed (6-12”)
	Redtop	Good stand, uncut (15-20”)
D	Tall fescue	Good stand, cut (3-4”)
	Bermudagrass	Good stand, cut (2.5”)
	Bahiagrass	Good stand, cut (3-4”)
	Grass-legume mixture--fall-spring (orchardgrass,redtop, andannual lespedeza)	Good stand, uncut (4-5”)
	Red fescue	Good stand, uncut (12-18”)
	Centipedegrass	Good stand, cut (3-4”)
	Kentucky bluegrass	Good stand, cut (3-4”)
E	Bermudagrass	Good stand, cut (1.5”)
	Bermudagrass	Burned stubble

Modified from: USDA-SCS, 1969. Engineering Field Manual

Table 3-6
Maximum Velocities for Unprotected Soils in Existing Channels

Materials	Maximum Permissible Velocities (ft/s)
Fine Sand (noncolloidal)	2.5
Sand Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (colloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5

3.4.7 Example Problem

Step 1: Design Q_{10} = 16.6 cfs
Q_2 = 11.9 cfs

Step 2: Proposed channel grade = 2%
Proposed vegetation: Tall fescue
Soil: Creedmoor (easily erodible)
Trapezoidal channel dimensions:
designing for low retardance (class D) condition
designing to meet V_p

Step 3: Permissible velocity, V_p = 4.5 ft/s (Table 3-3)

Step 4: Make an initial estimate of channel size
A = Q/V, 16.6 cfs/4.5 ft/sec = 3.7 ft^2
Try bottom width (b) = 3.0 ft w/side slopes of 3:1 (M= 3)

Step 5: $R = A / P$
$A = by + My^2$
$P = b + 2y (M^2 + 1)^{1/2}$

Steps 6 - 8: An iterative solution using Figure 3-1 to relate flow depth to Manning's *n* proceeds as follows. Retardance class: "D" cut (Table 3-5). Manning's equation (3.1) is used to check velocities and Continuity equation (3.2) is used to calculate flow. Check results against permissible velocity and required design capacity to determine if design is acceptable. Iterate as necessary.

y (ft)	A (ft^2)	R (ft)	*n	V (ft/s)	Q (cfs)	Comments
0.8	4.32	0.54	0.043	3.25	14.0	$V < V_p$ ok $Q < Q_{10}$ too small
0.9	5.13	0.59	0.042	3.53	18.1	$V < V_p$ ok $Q > Q_{10}$ ok

Note: From Fig. 3-1 use Retardance Class D line and permissible velocity, V_p x calculated R (4.5x0.54 = 2.43) to get corresponding *n = 0.043

Step 9: Not needed in this example since original bottom width guess worked.

Step 10: Now design for high retardance (Class B): Repeat steps 6 - 8.
For the ease of construction and maintenance, try y = 1.5 feet and trial velocity V_t = 3 ft/s

y (ft)	A (ft^2)	R (ft)	V_t (ft/s)	*n	V(ft/s)	Q (cfs)	Comments
1.5	11.25	0.9	3	0.08	2.5	28	reduce V_t ,try 2
			2	0.11	1.8	20	reduce V_t ,try 1.6
			1.6	0.12	1.6	18	check V_t = 1.5
			1.5	0.13	1.5	17	$Q > Q_{10}$ ok

Step 11: Energy dissipation not needed in this example.

Step 12: After adding 6 inches of freeboard – required channel depth is 2 feet.
Channel Summary: Trapezoidal shape, M = 3, b = 3 ft, y = 2 ft, grade = 2%

Note: In the example problem, the Manning's *n* is first chosen based on a permissible velocity and not a design velocity criteria. Therefore, the use of Table 3-5 may not be as accurate as individual retardance class charts when a design velocity is the determining factor.

Step 13: Check to see if temporary liner is needed for the vegetated channel, if so select lining to try and use *n* value from manufacturer or Table 3-7.

Q_2 = 11.8 cfs, Bottom width (b) = 3.0 ft, M= 3
n = 0.02 (Use basic *n* value for channels cut in earth (Table 3-4)
Vp = 2.0 ft/sec maximum allowable velocity for bare soil
Channel gradient = 2%

Using Manning's equation:

B (ft)	y (ft)	A (ft^2)	R (ft)	V (ft/s)	Q (cfs)	Comments
3	0.5	2.23	0.36	5.35	11.9	V>Vp protection/liner is required

Step 14: Calculate channel design using <u>straw matting (double net)</u> as temporary liner.
n = 0.028 (Table 3-7); Td = 1.75 (Table 3-8)

b (ft)	y (ft)	A (ft^2)	R (ft)	V (ft/s)	Q (cfs)
3	0.59	2.83	0.42	4.21	11.9

Step 15: Calculate shear stress for Q_2 conditions:
$T = \omega ds$ (3.3)
T = (62.4)(0.59)(0.02) = 0.74

Step 16: Compare calculated T with permissible shear T_d:
If $T < T_d$ then OK**
T= 0.74 (Step 15)
T_d = 1.75 (Table 3-7)
0.74 < 1.75, OK**
Temporary liner: straw with net.

(**In some cases the solution is not as clearly defined; the use of a more conservative material is recommended.)

Channel Summary:

Channel is trapezoidal with a 3 foot bottom, 3:1 side slopes, 2 foot total depth, and 2% channel slope. Channel is lined with straw matting (double net).

Table 3-7
Manning's Roughness Coefficient for Temporary Lining Materials

	***n* value for Depth Ranges**		
Lining Type	**0-0.5 ft**	**0.5-2.0 ft**	**>2.0 ft**
Straw matting (double net)	0.055	0.028	0.021
70% straw/30% coconut matting (double net)	0.050	0.025	0.018
Coconut matting (double net)	0.022	0.014	0.014
Coconut matting (triple, permanent net)	0.041	0.018	0.012

Application Note: The designer is responsible for providing documentation for design values of roughness (n). The n values listed in Table 3-6 may not conform to the values of any specific manufacturer.

Table 3-8
Permissible Shear Stresses for Riprap and Temporary Liners

Lining Category	Lining Type	Permissible Unit Shear Stress T_d (lb/ft^2)
Temporary	Straw matting (double net)	1.75
	70%Straw/30% Coconut matting (double net)	2.00
	Coconut matting (double net)	2.25
	Coconut matting (triple, permanent net)	3.00

Application Note: The designer is responsible for providing documentation of design values for allowable stress and velocity of proprietary linings. The permissible shear stress values listed in Table 3-8 may not conform to the values of any specific manufacturer.

Lining Category	d_{50} Stone Size (inches)	Permissible Unit Shear Stress T_d (lb/ft^2)
Gravel Riprap	1	0.33
	2	0.67
Rock Riprap	6	2.00
	9	3.00
	12	4.00
	15	5.00
	18	6.00
	21	7.80
	24	8.00

Application Note: The designer is responsible for specifying the proper depth and size for riprap used either as a channel lining or as an energy dissipator.

3.5 RIPRAP DESIGN

3.5.1 Assumptions

This procedure applies to riprap placement in both natural and prismatic channels and has the following assumptions and limitations:

1. Maximum side slope is 2:1

2. Maximum allowable velocity is 14 feet per second

If significant turbulence is caused by boundary irregularities, such as installations near obstructions or structures, this procedure is not applicable. Where riprap is used in ditches/channels that are 2% or less, the design should take into account the potential for sedimentation.

3.5.2 Procedure for Riprap Channel Design

Following are the steps in the procedure for riprap design.

1. Determine the average velocity in the main channel for the design condition. Use the higher value of velocity calculated both with and without riprap in place (this may require iteration using procedures in Appendix 3C.4). Manning's *n* values for riprap can be calculated from the equation:

$$\mathbf{n = 0.0395\ (d_{50})^{1/6}} \qquad \textbf{(3.5)}$$

   Where: n = Manning's roughness coefficient for stone riprap

   d_{50} = diameter of stone for which 50 percent, by weight, of the gradation is finer (ft)

2. If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 3-4 for either a natural or prismatic channel. This requires determining the channel top width, W, just upstream from the bend and the centerline bend radius, R_b.

3. If the specific weight of the stone varies from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 3-5.

4. Determine the required minimum d_{30} value from Figure 3-6 which is based on the equation:

$$\mathbf{d_{30}/D = 0.193\ Fr^{2.5}} \qquad \textbf{(3.6)}$$

   Where: d_{30} = diameter of stone for which 30 percent, by weight, of the gradation is finer (ft)

   D = depth of flow above stone (ft)

   Fr = Froude number (see equation D.3), dimensionless

   v = mean velocity above the stone (ft/s)

   g = acceleration due to gravity (32.3 ft/sec^2)

5. Determine available riprap gradations. A well graded riprap is preferable to uniform size or gap graded. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$\mathbf{w = 0.5236\ SW_s\ d^3} \qquad \textbf{(3.7)}$$

   Where: w = stone weight (lbs)

   d = selected stone diameter (ft)

   SW_s = specific weight of stone (lbs/ft^3)

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased by 50 percent for underwater placement.

6. If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 3-7 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

7. Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

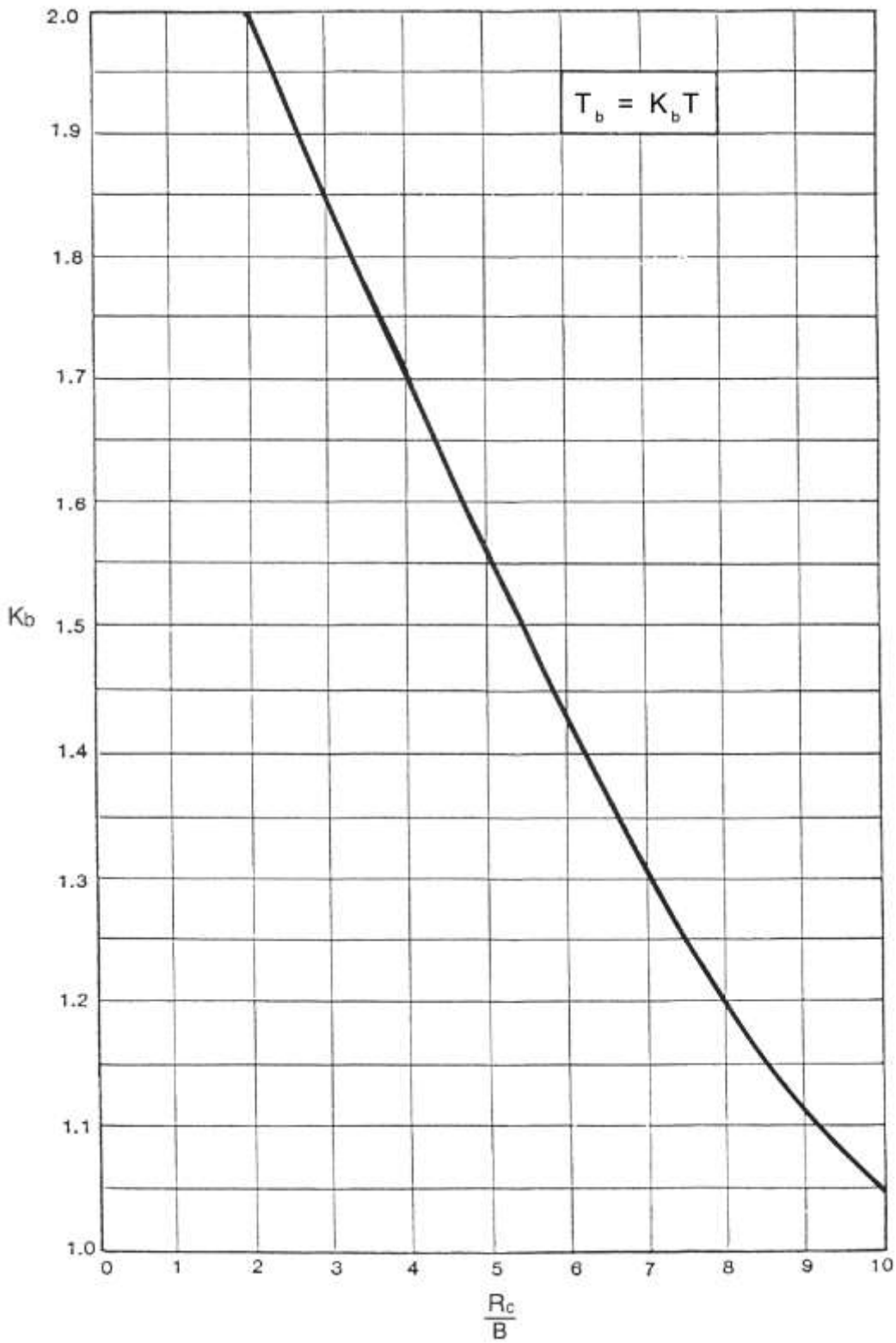


Figure 3-2 K_b Factor for Maximum Shear Stress on Channel Bends
Source: www.dlr.enr.state.nc.us/images/Sediment_design_manual_June2006/ChapterEight_20070419.PDF, page 607

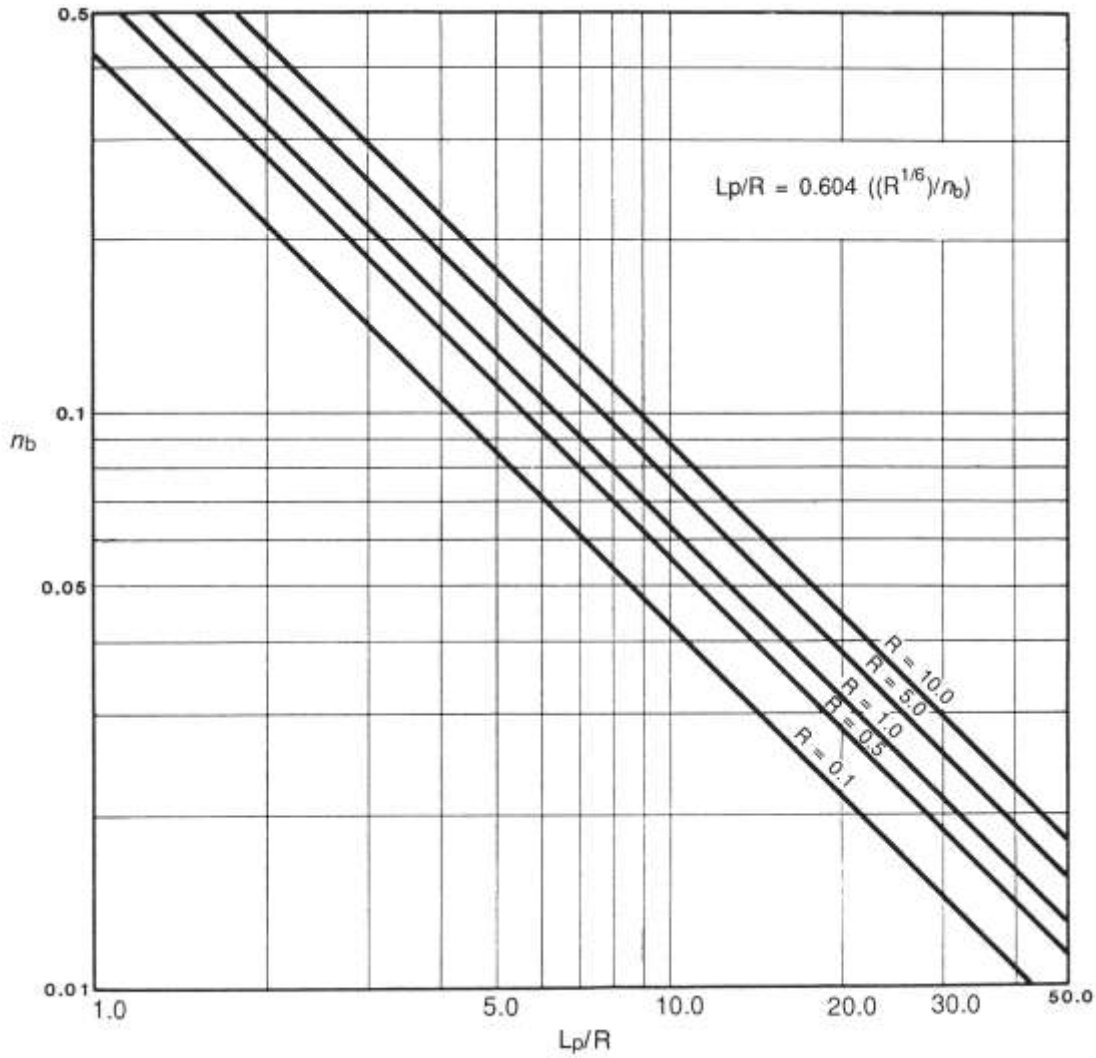


Figure 3-3 Protection Length, L_p, Downstream from Channel Bend.

n_b = Manning's Roughness of the lining material in the bend and the depth of flow
R = Hydraulic Radius = Area/wetted perimeter

Source: www.dlr.enr.state.nc.us/images/Sediment_design_manual_June2006/ChapterEight_20070419.PDF, page 608

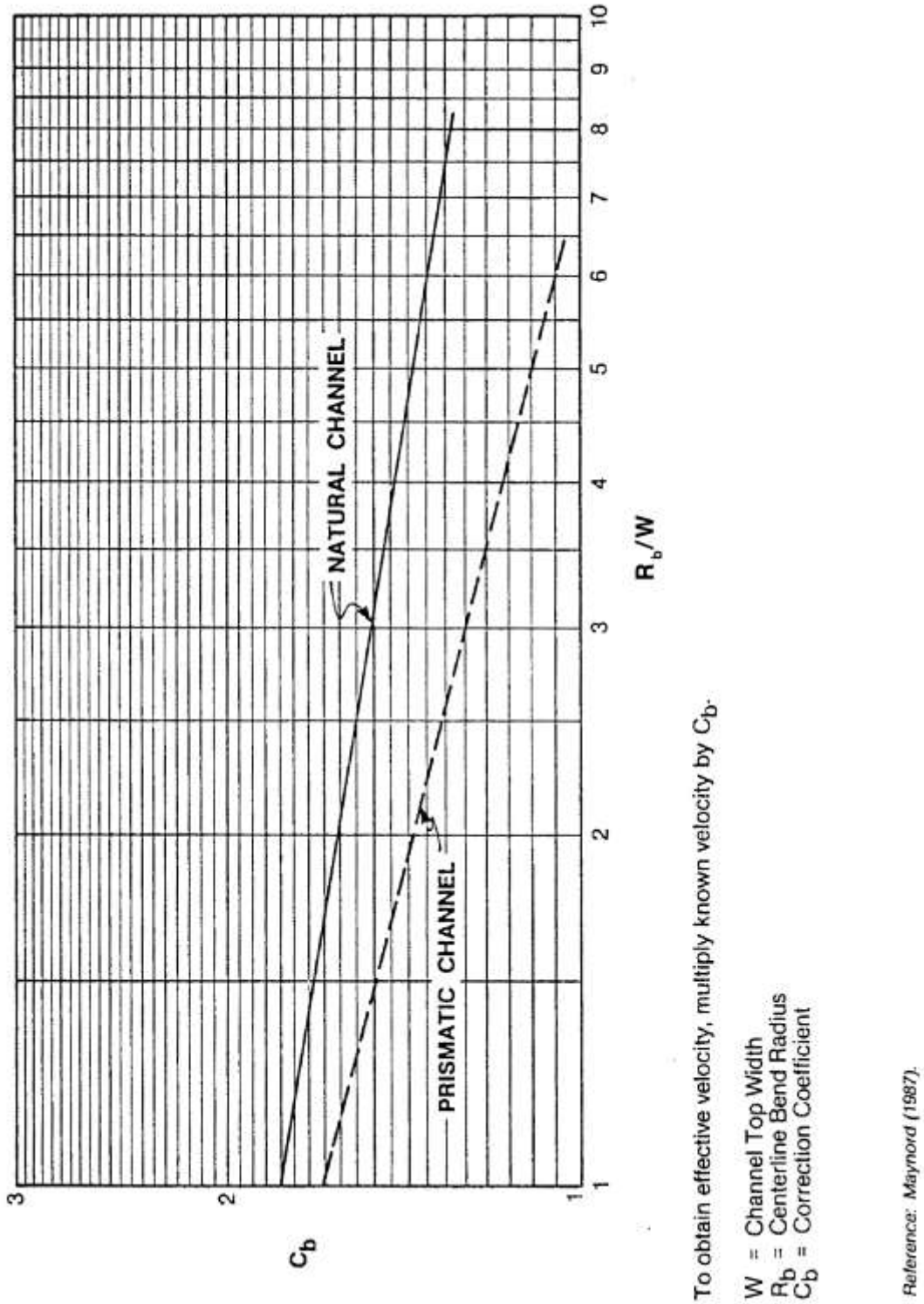
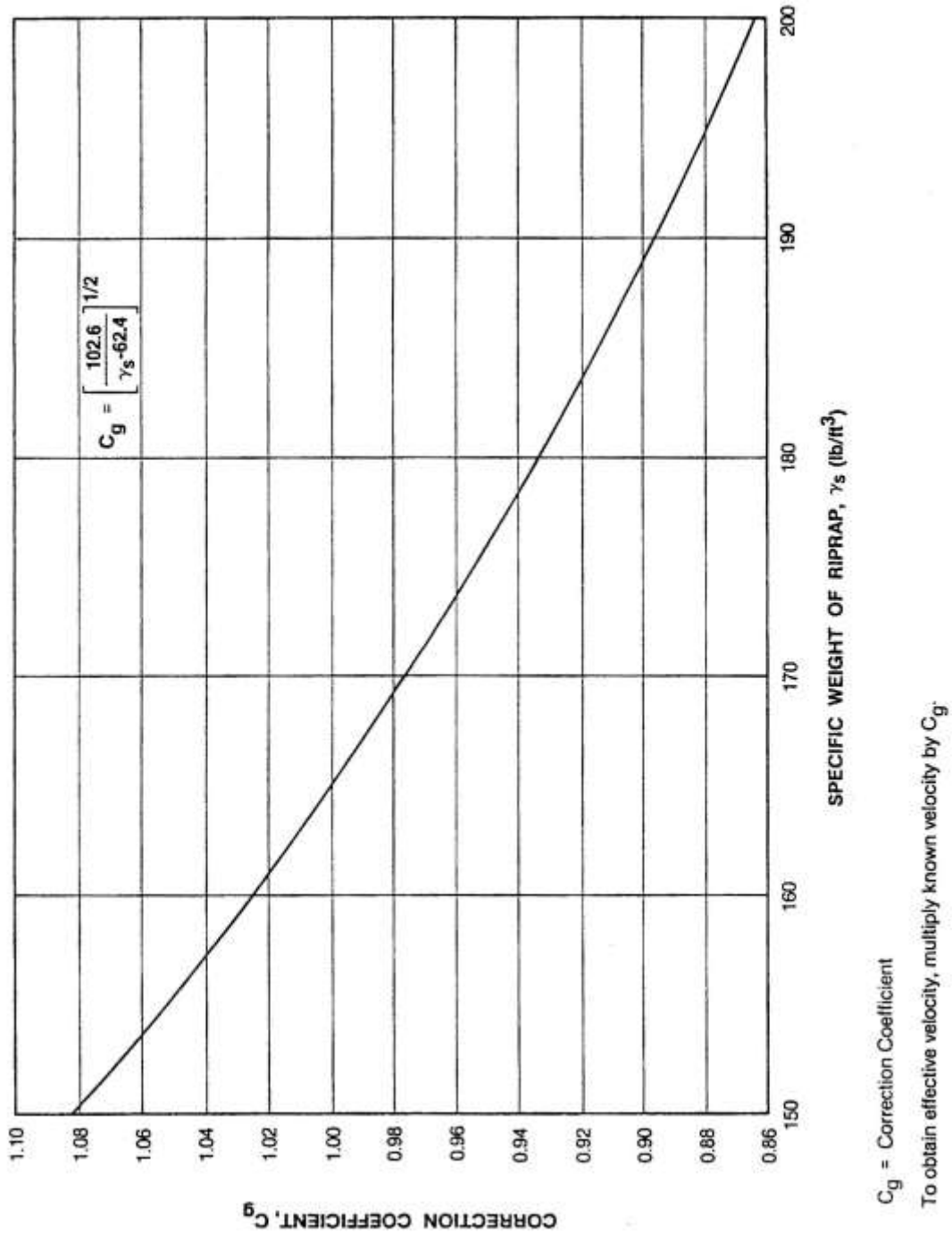


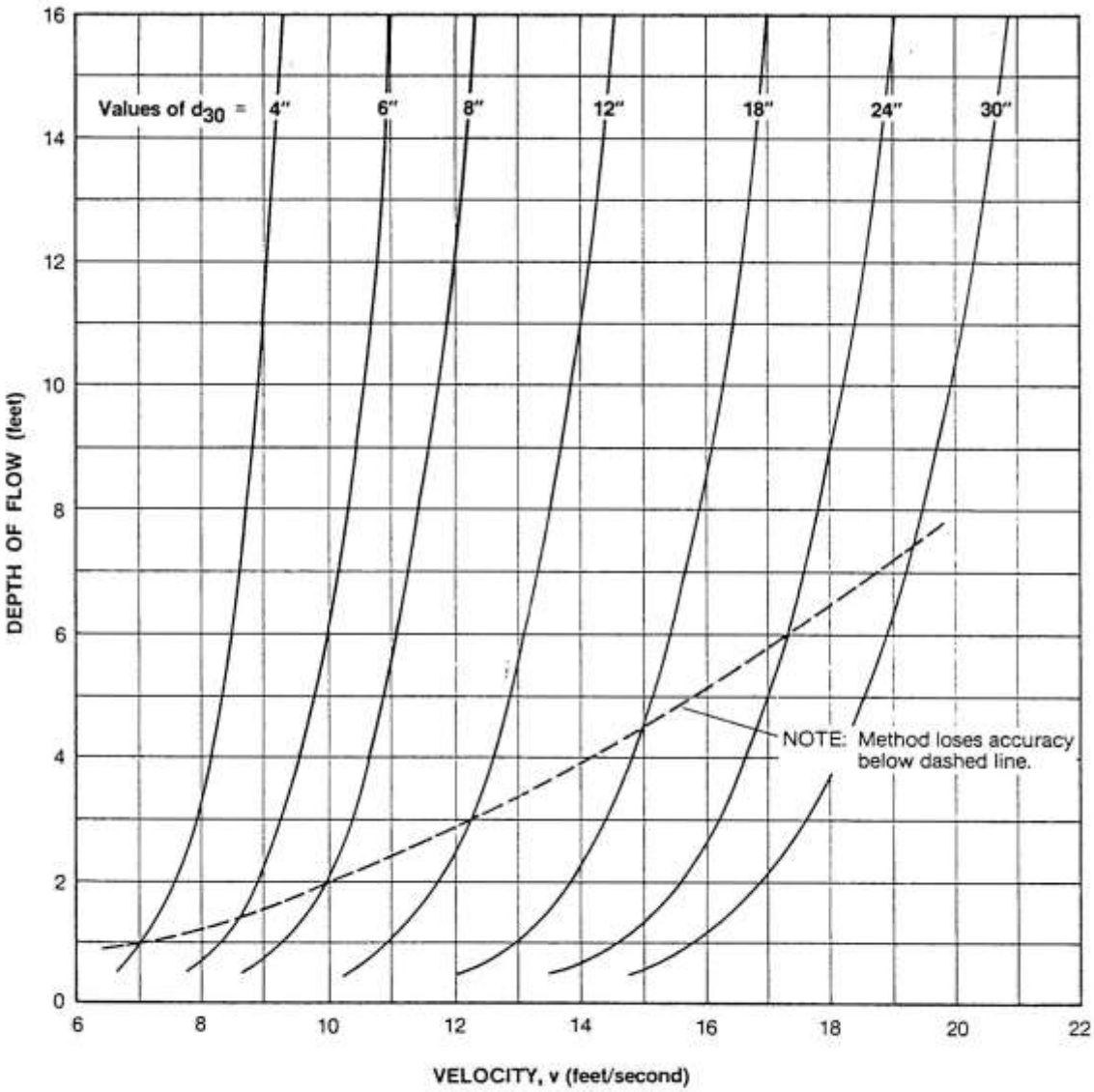
Figure 3-4 Riprap Lining Bend Correction Coefficient
Source: Georgia Stormwater Management Manual, Volume 2, 4.4-23



C_g = Correction Coefficient

To obtain effective velocity, multiply known velocity by C_g.

Figure 3-5 Riprap Lining Specific Weight Correction Coefficient
Source: Nashville Storm Water Management Manual, 1988



Reference: Reese (1988).

Figure 3-6 Riprap Lining d_{30} Stone Size - Function of Mean Velocity and Depth
Source: Georgia Stormwater Management Manual, Volume 2, 4.4-25

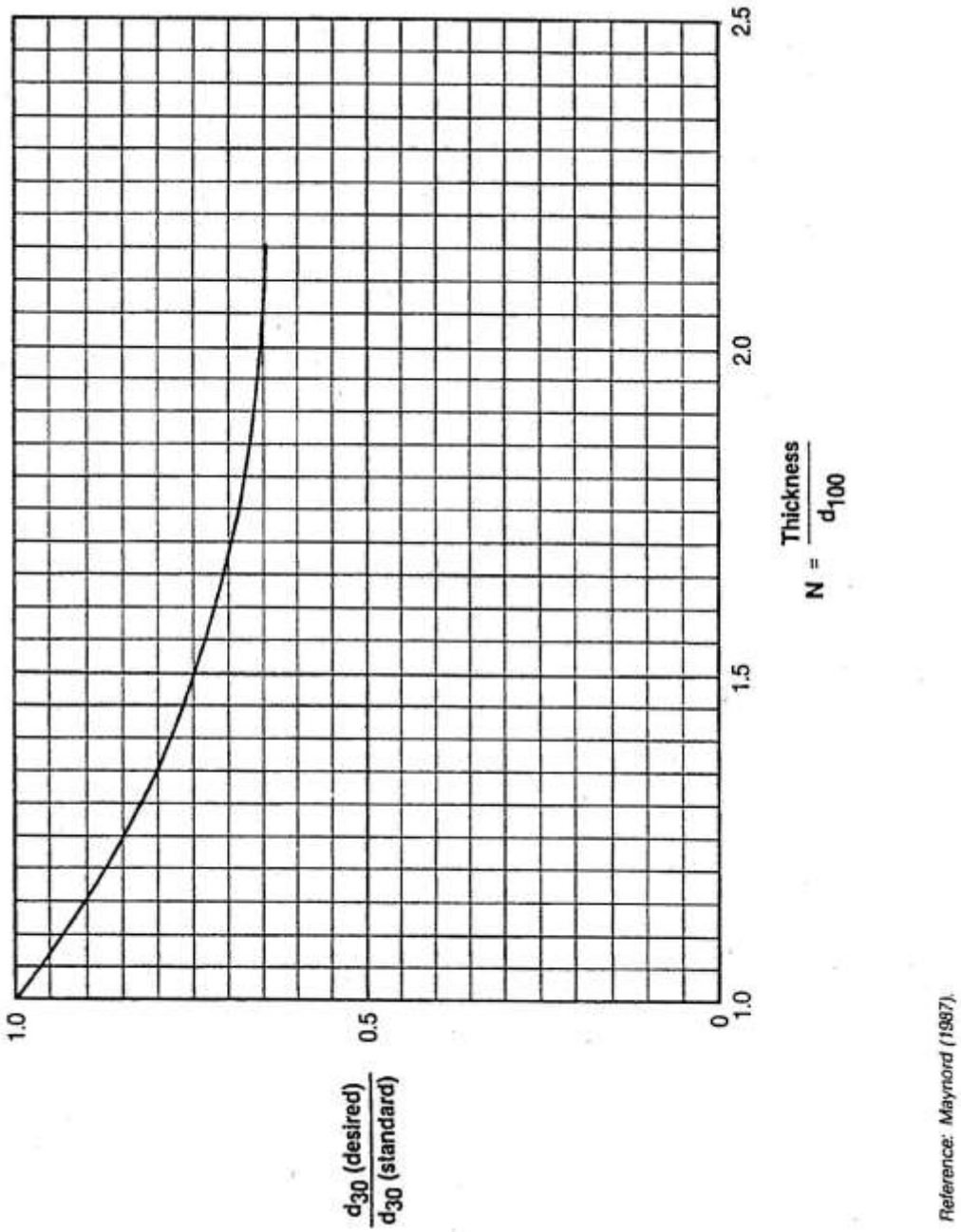


Figure 3-7 Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 2.0 to 2.3
Source: Georgia Stormwater Management Manual, Volume 2, 4.4-27

3.6 APPROXIMATE FLOOD LIMITS

3.6.1 Introduction

For streams and tributaries with drainage areas smaller than one square mile, analysis may be required to identify the 100-year flood elevation and building restriction floodline. This requires a backwater analysis to determine the stream flow depth. The HEC-RAS software package is an acceptable method.

3.6.2 Floodline Restrictions

For such cases when the design engineer can demonstrate that a complete backwater analysis is unwarranted, approximate methods may be used.

A generally accepted method for approximating the 100-year flood elevation is outlined as follows:

1. Divide the stream or tributary into reaches that may be approximated using average slopes, cross sections, and roughness coefficients for each reach. The maximum allowable distance between cross sections is 100 feet.

2. Estimate the 100-year peak discharge for each reach using an appropriate hydrologic method from Hydrology Chapter 2.

3. Compute normal depth for uniform flow in each reach using Manning's equation for the reach characteristics from Step 1 and peak discharge from Step 2.

4. Use the normal depths computed in Step 3 to approximate the 100-year flood elevation in each reach. The 100-year flood elevation is then used to delineate the flood plain.

This approximate method is based on several assumptions, including, but not limited to, the following:

1. A channel reach is accurately approximated by average characteristics throughout its length.

2. The cross-sectional geometry, including area, wetted perimeter, and hydraulic radius, of a reach may be approximated using typical geometric properties that can be used in Manning's equation to solve for normal depth.

3. Uniform flow can be established and backwater effects are negligible between reaches.

4. Expansion and contraction effects are negligible.

As indicated, the approximate method is based on a number of restrictive assumptions that may limit the accuracy of the approximation and applicability of the method. The engineer is responsible for appropriate application of this method.

APPENDIX 3A
HYDRAULIC TERMS

3A.1 Energy of Flow

Flowing water contains energy in two forms—potential and kinetic. The potential energy at a particular point is represented by the depth of the water plus that elevation of the channel bottom above a convenient datum plane. The kinetic energy (in feet) is represented by the velocity head, $V^2/2g$. Figure 3A-1 illustrates open channel energy concepts and equation 3A.1 is the energy equation.

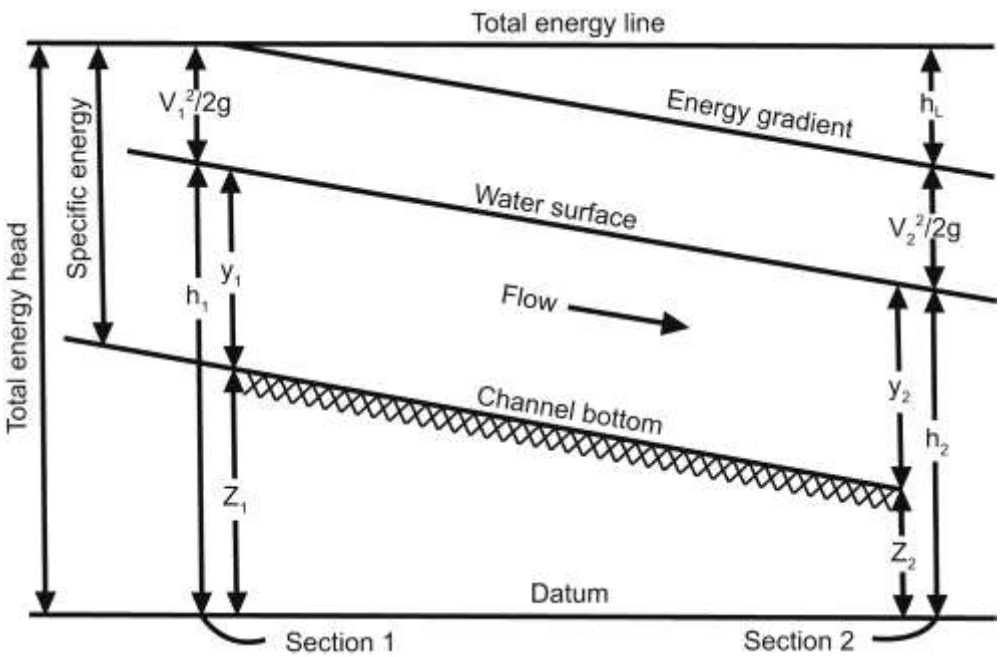


Figure 3A-1 Energy in Open Channel Flow

$$\mathbf{y_1 + V_1^2/2g + Z_1 = y_2 + V_2^2/2g + Z_2 + h_L} \qquad \textbf{(3A.1)}$$

Where: y = depth of flow above streambed (ft)

V = mean velocity of flow (ft/s)

Z = vertical distance from datum (ft)

g = acceleration due to gravity (32.2 ft/s^2)

h_L = head loss (ft)

The slope (gradient) of the total energy grade line is a measure of the friction slope or rate of energy head loss due to friction. The total head loss for a length of channel is the product of the length and friction slope (h_L = S x L). Under uniform flow, the energy line is parallel to the water surface and streambed.

3A.2 Steady and Unsteady

Flow in open channels is classified either as steady or unsteady flow. Steady flow occurs when discharge or rate of flow at any cross section is constant with time. In unsteady flow, the discharge or rate of flow varies from one cross-section to another with time.

3A.3 Uniform and Non-Uniform Flow

Uniform flow exists when the channel cross-section, roughness, and slope are constant; and non-uniform or varied flow exists when the channel properties vary from section to section.

3A.4 Froude Number

The Froude Number is the ratio of the inertial force to that of gravitational force, expressed by the following equation:

$$\mathbf{Fr = v / (gD)^{1/2}} \qquad \textbf{(3A.2)}$$

Where: v = mean velocity of flow (ft/s)

g = acceleration due to gravity (32.2 ft/s^2)

D = hydraulic depth (ft) – defined as the cross sectional area of water normal to the direction of channel flow divided by free surface width.

3A.5 Critical Flow

Critical flow is defined as the condition for which the Froude Number is equal to one. At that state of flow, the specific energy is a minimum for a constant discharge. By plotting specific energy head against depth of flow for a constant discharge, a specific energy diagram can be drawn as illustrated in Figure 3A-2. Also, by plotting discharge against specific energy head we can illustrate not only minimum specific energy for a given discharge per unit width, but also maximum discharge per unit for a given specific energy.

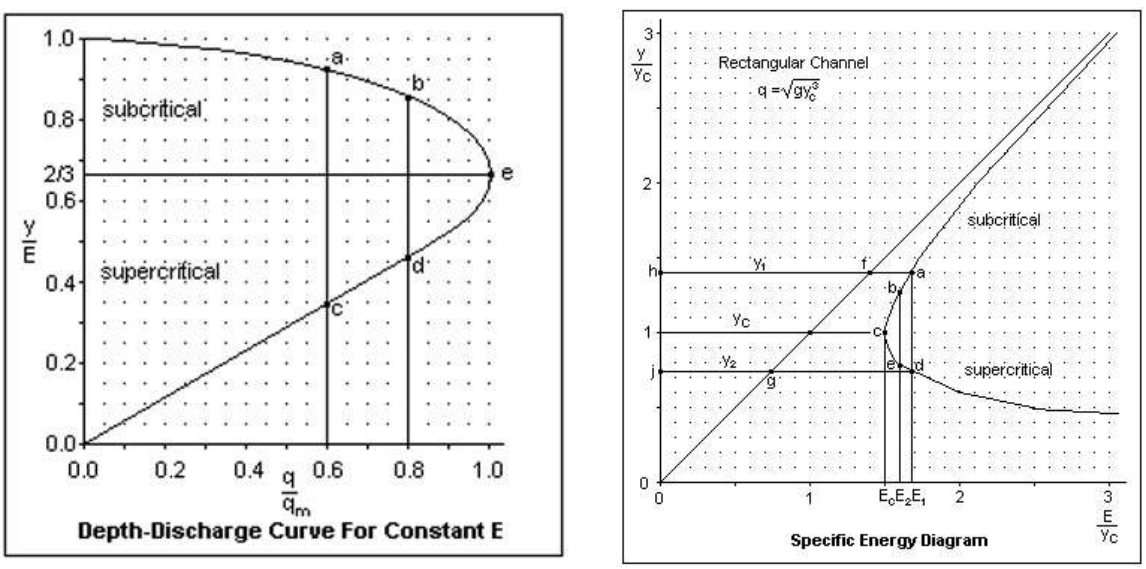


Figure 3A-2 Definition Sketch of Specific Energy

3A.6 Subcritical Flow

When the Froude Number is smaller than 1, the state of flow is defined as subcritical or tranquil flow, and surface waves propagate upstream as well as downstream. Control of subcritical flow depth is always downstream.

3A.7 Supercritical Flow

When the Froude Number is larger than 1, the state of flow is defined as supercritical or rapid flow, and surface disturbance can propagate only in the downstream direction. Control of supercritical flow depth is always at the upstream end of the critical flow region.

APPENDIX 3B
BEST HYDRAULIC SECTION

3B.1 Introduction

For a given discharge, slope and channel roughness, maximum velocity implies minimum cross sectional area. From Manning's equation, if velocity is maximized and area is minimized, wetted perimeter will also be minimized. The best hydraulic section therefore, simultaneously minimizes area and wetted perimeter.

For ease of construction, most channels are built with trapezoidal cross-sections. Therefore, this chapter deals with computing the best hydraulic section for trapezoidal section channels.

3B.2 Equations

Given that the desired side slope, M to one, has been selected for a given channel, the minimum wetted perimeter (P) exists when:

$$\mathbf{P = 4y\,(1 + M^2)^{1/2} - 2My} \qquad \textbf{(3B.1)}$$

Where: P = wetted perimeter (ft)

y = depth of flow (ft)

M= side slope, M to 1

(Figure D-1 below shows a definition of variables.)

From the geometry of the channel cross-section and the Manning's equation, design equations can be developed for determining the dimensions of the best hydraulic section for a trapezoidal channel.

The depth of the best hydraulic section is defined by:

$$\mathbf{y = C_M\,(Qn/(S^{1/2}))^{3/8}} \qquad \textbf{(3B.2)}$$

Where:

$$\mathbf{C_M = \left[\frac{\{k + 2(M^2 + 1)^{1/2}\}^{2/3}}{1.49\,(k + M)^{5/3}}\right]^{3/8}} \qquad \textbf{(3B.3)}$$

The associated bottom width is:

$$\mathbf{b = ky} \qquad \textbf{(3B.4)}$$

The cross-sectional area of the resulting channel is:

$$\mathbf{A = by + My^2} \qquad \textbf{(3B.5)}$$

where: y = depth of flow (ft)

C_M = coefficient

n = Manning's roughness coefficient

Q = discharge rate (cfs)

S = slope (ft/ft)

M = sideslope, M to 1

k = coefficient

A = cross-sectional area (ft^2)

Table 3B -1 lists values of C_M and k for various values of M.

Table 3B-1

Best Hydraulic Section Coefficients

Values of C_M and k for determining bottom width and depth of best hydraulic section for a trapezoidal channel.

M	C_M	k
0/1	0.790	2.00
0.5/1	0.833	1.236
0.577/1	0.833	1.155
1.0/1	0.817	0.828
1.5/1	0.775	0.606
2.0/1	0.729	0.472
2.5/1	0.688	0.385
3.0/1	0.653	0.325
3.5/1	0.622	0.280
4.0/1	0.595	0.246
5.0/1	0.522	0.198
6.0/1	0.518	0.166
8.0/1	0.467	0.125
10.0/1	0.430	0.100
12.0/1	0.402	0.083

APPENDIX 3C
OPEN CHANNEL FLOW CALCULATIONS

3C.1 Design Charts

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Association has prepared numerous design charts to aid in the design of rectangular, triangular and trapezoidal open channel cross sections. In addition, design charts for grass-lined channels have been developed. For a complete discussion of these charts and their use in open channel design refer to the publication *Design Charts for Open Channel Flow*, Federal Highway Administration, Hydraulic Design Series No. 3, 1973.

3C.2 Manning's Evaluation

Manning's equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$\mathbf{v = (1.49/n)R^{2/3}S^{1/2}} \qquad \textbf{(3C.1)}$$

$$\mathbf{Q = (1.49/n)A\,R^{2/3}\,S^{1/2}} \qquad \textbf{(3C.2)}$$

$$\mathbf{S = [Qn/(1.49\,A\,R^{2/3})]^2} \qquad \textbf{(3C.3)}$$

Where: v = average channel velocity (ft/s)

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

A = cross-sectional area (ft^2)

R = hydraulic radius (ft)

S = slope of the energy grade line (ft/ft)

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line and channel bottom are equal.

3C.3 Geometric Relationships

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross-sections can be calculated from their geometric dimensions. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

3C.4 Normal Depth Solutions

A trial and error procedure for solving Manning's equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness and design discharge are known. For purposes of the trial and error process, Manning's equation can be arranged as:

$$\mathbf{AR^{2/3} = (Qn)/(1.49\ S^{1/2})} \qquad \textbf{(3C.4)}$$

Where: A = cross-sectional area (ft)

R = Hydraulic radius (ft)

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

S = slope of the energy grade line (ft/ft)

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of equation 3C.4 is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figure 3C-1 for trapezoidal channels, which is described below.

1. Determine design discharge, Q, Manning's *n* value, channel bottom width (b), channel slope (S). and channel side slope (M).

2. Calculate the trapezoidal conveyance factor using the equation:

$$\mathbf{K_T = (Qn)/(b^{8/3}S^{1/2})} \qquad \textbf{(3C.5)}$$

Where: K_T = Trapezoidal open channel conveyance factor

Q = Discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

b = bottom width (ft)

S = slope of the energy grade line (ft/ft)

3. Enter the x-axis of Figure 3C-1 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate M value from Step 1.

4. From the point of intersections obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, y/b.

5. Multiply the y/b value from Step 4 by b to obtain the normal depth of flow.

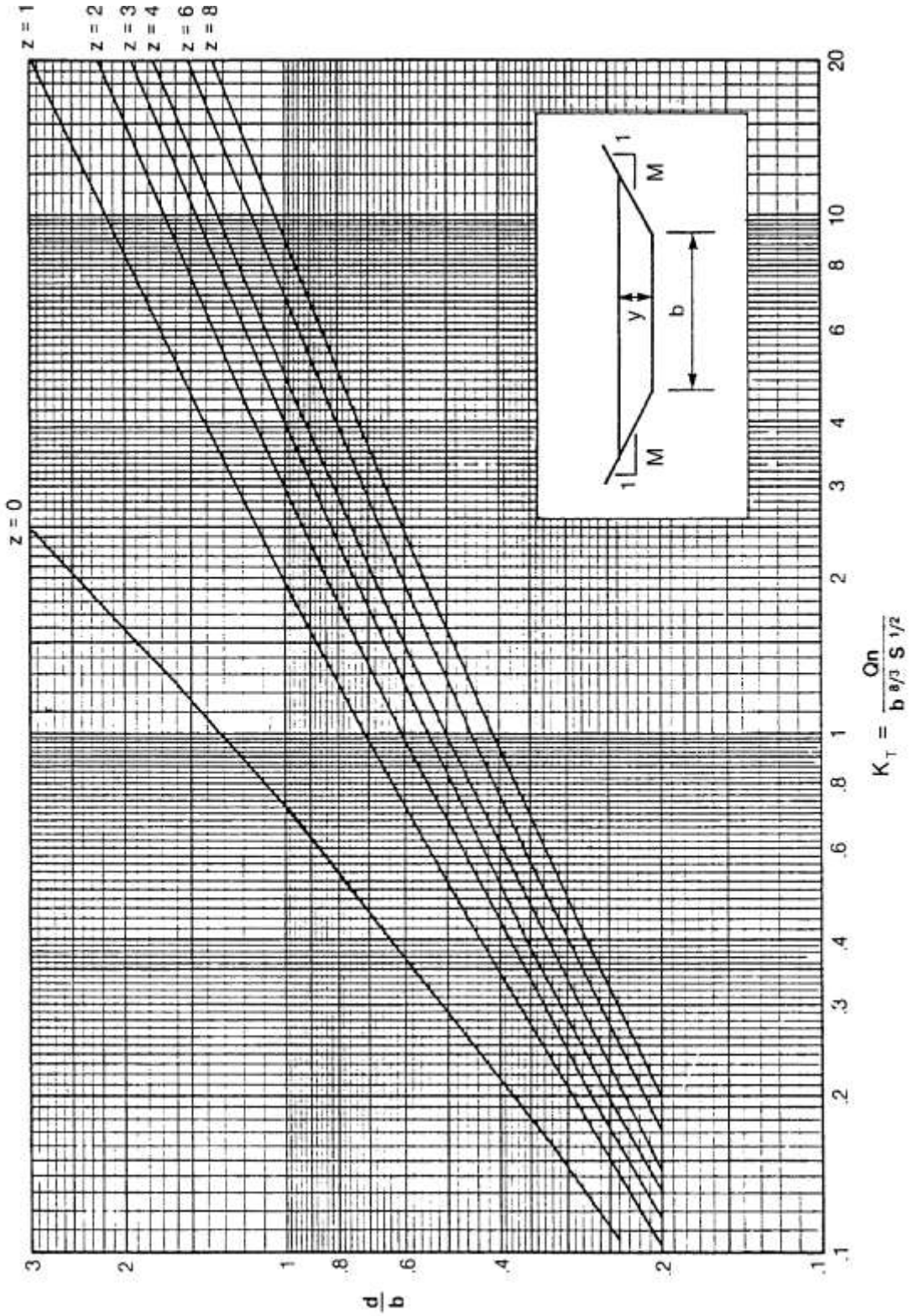


Figure 3C-1 Trapezoidal Channel Capacity Chart
Source: Georgia Stormwater Management Manual, Volume 2, 4.4-15

APPENDIX 3D
CRITICAL FLOW CALCULATIONS

3D.1 Background

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$\mathbf{Q^2/g = A^3/W} \qquad \textbf{(3D.1)}$$

Where: Q = discharge rate for conditions (cfs)

g = acceleration due to gravity (32.2 ft/sec^2)

A = cross-sectional area (ft^2)

W = top width of water surface (ft)

A trial and error procedure is needed to solve equation D.1.

3D.2 Semi-Empirical Equations

Semi-empirical equations (as presented in Table 3D-1) or section factors (as presented in Figure 3D-1) can be used to simplify trial and error critical depth calculations. The following equation from Chow (1959) is used to determine critical depth with the critical flow section factor, Z:

$$\mathbf{Z = Q/(g^{1/2})} \qquad \textbf{(3D.2)}$$

Where: Z = critical flow section factor

Q = discharge rate for design conditions (cfs)

g = acceleration due to gravity (32.2 ft/sec^2)

The following guidelines are presented for evaluating critical flow conditions of open channel flow:

1. A normal depth of uniform flow within 10 percent of critical depth is unstable and should be avoided in design, if possible.

2. If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.

3. If the velocity head is equal to one-half the mean depth of flow, the flow is critical.

4. If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

5. If an unstable critical depth cannot be avoided in design, the least type of flow should be assumed for the design.

3D.3 Froude Number

The Froude Number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$\mathbf{Fr = v / (gA/W)^{1/2}} \qquad \textbf{(3D.3)}$$

Where: Fr = Froude number (dimensionless)

v = velocity of flow (ft/s)

g = acceleration due to gravity (32.2 ft/sec^2)

A = cross-sectional area of flow (ft^2)

W = top width of flow (ft)

If Fr is greater than 1.0, flow is supercritical; if is under 1.0 flow is subcritical. Fr equals 1.0 for critical flow conditions.

Table 3D-1
Critical Depth Equations for Uniform Flow in Channel Cross Sections

Channel Type[a]	Semi-Empirical Equation[b] for Estimating Critical Depth	Range of Applicability
1. Rectangular[c]	$y_c = (Q^2/gb^2)^{1/3}$	N/A
2. Trapezoidal	$y_c = 0.81(Q^2/gM^{0.75}b^{1.25})^{0.27}-(b/30M)$	$0.1<0.5522(Q/b^{2.5})<0.4$, For $0.5522\ (Q/b^{2.5})< 0.1$, use rectangular channel equation
3. Triangular[c]	$y_c = [(2Q^2)/(gM^2)]^{1/5}$	N/A
4. Circular[d]	$y_c = 0.325(Q/D)^{2/3} + 0.083D$	$0.3 < y_c/D < 0.9$
5. General[e]	$A^3/W = (Q^2/g)$	N/A

Where:
y_c = Critical depth, in feet
Q = Design discharge, in cfs
g = Acceleration due to gravity, 32.2 feet/second^2
b = Bottom width of channel, in feet
M = Side slopes of a channel (horizontal to vertical)
D = Diameter of circular conduit, in feet
A = Cross-sectional area of flow, in square feet
W = Top width of water surface, in feet

[a] See Figure D-1 for channel sketches
[b] Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[c] Reference: French (1985)
[d] Reference: USDOT, FHWA, HDS-4 (1965)
[e] Reference: Brater and King (1976)

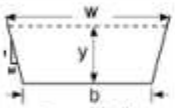
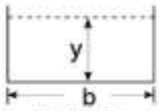
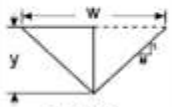
Section	Area A	Wetted Perimeter, P	Hydraulic Radius, R	Top Width, W	Critical Depth Factor, Z
 Trapezoid	$by + My^2$	$b + 2y(M^2+1)^{1/2}$	$\frac{by + My^2}{b + 2y\sqrt{M^2+1}}$	$b + 2My$	$\frac{[(b + My)y]^{1.5}}{\sqrt{b + 2My}}$
 Rectangle	by	$b + 2y$	$\frac{by}{b + 2y}$	b	$by^{1.5}$
 Triangle	My^2	$2y\sqrt{M^2+1}$	$\frac{My}{2\sqrt{M^2+1}}$	$2My$	$\frac{\sqrt{2}}{2} My^{2.5}$

Figure 3D-1 Open Channel Geometric Relationships for Various Cross Sections

CHAPTER 4

STORM DRAINAGE SYSTEMS

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4.1 OVERVIEW

4.1.1 Introduction

In this chapter, guidelines are given for calculating gutter and inlet hydraulics and storm drainage design. Procedures for performing gutter flow calculations are based on a modification of Manning's equation. Inlet capacity calculations for grate and combination inlets are based on information contained in HEC- 22 (USDOT, FHWA, 1996). Storm drain design is based on the use of the rational formula.

4.1.2 Inlet Definition

There are four storm water inlet categories:

- curb opening inlets
- grated inlets
- combination inlets
- multiple inlets

In addition, inlets may be classified as being on a continuous grade or in a sag. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sag" condition exists when the inlet is located at a low point and water enters from both directions.

4.1.3 Criteria

The following criteria shall be used for drainage system design.

- Maximum spread of 6 feet in a travel lane based on a rainfall intensity of 4 inches/hour.
- For a street with a valley gutter, another foot for the gutter is allowed with a total maximum spread of 7 feet based on a rainfall intensity of 4 inches per hour.
- For a street with a standard 2 feet 6 inch curb and gutter, an additional 2 feet is allowed with a total maximum spread of 8 feet from the face of the curb based on a rainfall intensity of 4 inches per hour.
- See Section 2.3.1 for additional criteria on design storm frequencies.

Other

- In sag areas where relief by curb overflow is not provided the system standard design level ($Q_{25} - Q_{50}$) is to be used for analysis to ensure traffic flow is not interrupted (NC Division of Highways, Guidelines for Drainage Studies and Hydraulic Design, 1999). Therefore, in a sag condition where relief by overflow for a typical roadway cross section is not provided, inlet capacity and the storm drainage system must be designed for:
    - one dry 8 foot travel lane in the 25-year event for local streets; and
    - one dry 8 foot travel lane in each direction in the 50-year event for thoroughfares.

  All upstream bypass flow must be considered in the design of the inlets and the storm drainage system at the sag. Available overflow at sag locations can be considered when making the determination for available travel lanes in the relevant storm events.

- For local roads the pipe system hydraulic grade line shall not surcharge sag inlets in the 25-year storm. For thoroughfare roads the pipe system hydraulic grade line shall not surcharge in the 50-year storm.
- When checking spread requirements at sag points (0% slope) check spread upstream of sags (at the 0.5% slope point) to verify spread is not exceeded. Additional flanking inlets upstream may need to be added to keep spread criteria from being exceeded at these points.
- Ponding at yard inlets outside the roadway shall be limited to a maximum of one foot above a grated inlet for the 10-year storm.
- No concentrated runoff flowing over City sidewalks except at driveways.
- Roadside ditches, when allowed, shall be a minimum of 18 inches deep and shall provide the capacity designed for a 10-year storm with 6-inches of freeboard. For subdivision streets ditch flow for the 25-year storm shall not encroach onto the pavement. For thoroughfare streets ditch flow for the 50-year storm shall not encroach onto the pavement.
- For subdivision streets the driveway and culvert shall be designed such that the flow from a 25-year storm shall not encroach onto the roadway pavement. For thoroughfare streets the driveway and culvert shall be designed such that the flow from a 50-year storm shall not encroach onto the roadway pavement.

4.2 PAVEMENT DRAINAGE

4.2.1 Introduction

Design factors to be considered during gutter, inlet and pavement drainage calculations include:

- Return period
- Spread
- Storm drain location
- Inlet types and spacing
- Longitudinal slope
- Shoulder gutter
- Cross slope
- Curb and gutter sections
- Roadside and median ditches
- Bridge decks
- Median barriers

4.2.2 Storm Drain Location

For standards related to storm drain location refer to the Charlotte Land Development Standards Manual.

4.2.3 Inlet Types and Spacing

Inlet types shall be selected from the Charlotte Land Development Standards Manual or equivalent North Carolina State Department of Transportation standards. Inlets shall be located or spaced in such a manner that the design curb flow does not exceed the spread limitations. Flow across intersecting streets will be reviewed and approved on a case by case basis.

4.2.4 Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement, since it is susceptible to stormwater spread. Flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Curb and gutter grades that are equal to pavement slopes shall not fall below 0.5 percent. Minimum grades can be maintained in very flat terrain by use of a sawtooth profile. For long vertical curves, cross slope may be varied slightly to achieve 0.5 percent minimum gutter grade.

4.2.5 Cross Slope

Refer to the design standards for pavement cross slopes as shown in the Charlotte Land Development Standards Manual.

4.2.6 Curb and Gutter

Curb and gutter installation shall be designed in accordance with the relevant standards for the jurisdiction.

4.2.7 Median Ditches

Large median areas and inside shoulders should be sloped to a center swale, preventing drainage from the median area from running across the pavement. This is particularly important for high-speed facilities, and for facilities with more than two lanes of traffic in each direction.

4.2.8 Roadside Ditches

Roadside ditches (when allowed) will be required behind the shoulder of roadways without curb and gutter to convey storm drainage away from the pavement to a discharge point. The steepest side slope allowed is 3:1 (horizontal to vertical) on the roadside of the ditch and 2:1 on the side closest to the right-of-way line. The ditch shall be graded to a minimum longitudinal slope of 1 percent and a maximum velocity of 4 ft/sec. For grass lined channels with velocities up to 7 ft/sec, permanent matting may be approved on a case by case basis. For velocities greater than 7 ft/sec, a concrete lined ditch may be required. Riprap will not be allowed for stabilization within the street right-of-way (except as outlet protection on culverts).

In addition to the design of roadside ditches, a design shall be provided for driveway culverts for each individual lot on the plan. The use of a small driveway culvert, 15 inches minimum diameter, in conjunction with overtopping of the driveway itself will be allowed as further described in Section 4.1.3. Sizes for all driveway culverts shall be shown in tabular form on the plans, and each culvert shall be designed for the highest ditch flow applicable for the lot.

4.2.9 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal. Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 1 percent. When bridges are placed at a vertical curve and the longitudinal slope is less than 1 percent, the gutter spread should be checked to ensure a safe, reasonable design.

Scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope protection, navigation channels, driving lanes, or railroad

tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains.

For situations where traffic under the bridge or environmental concerns prevent the use of scuppers, grated bridge drains should be used.

4.2.10 Median Barriers

Weep holes are often used to prevent ponding of water against median barriers (especially on superelevated curves). In order to minimize flow across traveled lanes, it is preferable to collect the water into a subsurface system connected to the main storm drain system.

4.3 GUTTER FLOW CALCULATIONS

4.3.1 Formula

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

$$\mathbf{Q = [0.56/n]\ S_T^{5/3}\ S_L^{1/2}\ T^{8/3}} \qquad \textbf{(4.1)}$$

Where: Q = Gutter flow rate (ft^3/sec)

n = Manning's roughness coefficient

S_T = Pavement cross slope (ft/ft)

S_L = Longitudinal slope (ft/ft)

T = Width of flow or spread (ft)

Note: Manning's *n* value for concrete curb and gutter is 0.016.

4.3.2 Procedure

Using Table 4-1, identify the following:

1. Inlet # – Assigned number (or label) of drainage structure.
2. Drainage Area – Area contributing runoff to the inlet (acres).
3. Surface 'Q' Sub. – Flow (in cfs) to the inlet.

   Q = CiA

   Where: C = runoff coefficient for the sub-drainage area

   i = 4.0 inches/hour

   A = Area determined in #2 (acres)
4. Surface 'Q' Total = $Q_{sub} + \sum Q_{bypass}$

5. Long. Slope – Longitudinal gutter slope at "Inlet #" (in feet per foot). This is equivalent to the roadway centerline profile.

6. Trans. Slope – Transverse slope at "Inlet #" (in feet per foot). This is equivalent to the roadway cross-slope.

7. K – This coefficient is used to determine the inlet capacity of a catch basin grate on grade. Refer to Figure 4-1.

8. Inlet Capacity (cfs)

   $$Q_{cap} = KD^{5/3}$$

   Where: $D = S_T \times T$, depth of flow at curb (ft)

   For a "normal crown" street, S_T = 3/8" per 1' = 0.0313 ft/ft.
   The maximum spread, T, is 8 feet. Therefore D = 0.0313 x 8 = 0.25ft.

9. Spread = Width of flow (feet)
   The maximum width of spread in a travel lane is 6 feet. Total allowable spread with standard curb and gutter is 8 feet, and with valley gutter is 7 feet.

10. Bypass Q = Surface Q_{total} – Inlet Q_{cap}

11. Bypass to Inlet# = List Inlet# directly downstream (bypass destination).
    Remember to add:
    Bypass Q + Surface Q_{sub} =Surface Q_{total}

    Note: Computer software for gutter flow analysis is acceptable. The computer printout should contain the same information that is shown in Table 4-1.

INLET CAPACITY CHART

PROJECT _____ COMPUTED BY _____ DATE _____

LOCATION _____ CHECKED BY _____

RAINFALL INTENSITY = 4.0 IN/HR

INLET #	DRAINAGE AREA	SURFACE 'Q'		SLOPE		K	INLET	SPREAD	BYPASS		REMARKS
		SUB.	TOTAL	LONG.	TRANS.		Qcap		Q	TO INLET #	

Table 4-1 Inlet Capacity Chart

4.4 GRATE INLET DESIGN

4.4.1 Grate Inlets on Grade

Following is a discussion of the procedure for the design of grate inlets on grade. Figure 4-1 is used for the design of grate inlets on grade using type 'F' and 'G' grates.

4.4.1.1 Design Steps

1. Determine the following input data: Q, S_L, S_T, and n

2. D is the depth of water (or head) in the gutter immediately upstream of the grate (in feet). However, before this depth can be calculated, certain parameters must be set. In the case of street design, it is undesirable to have the street inundated and impassable due to the amount of runoff drainage down a given street. Therefore, the maximum allowable top width of water flow, or spread, T, in the gutter and street must be regulated such that flooding does not occur.

3. With the discharge rate, Q, known, T can be solved by applying the modified Manning's equation:

$$\mathbf{Q = (0.56/n)ZD^{8/3}S_L^{1/2}} \qquad \textbf{(4.2)}$$

Since: $$\mathbf{D = T(S_T)} \qquad \textbf{(4.3)}$$

And: $$\mathbf{S_T = 1/Z} \qquad \textbf{(4.4)}$$

T can be derived:

$$\mathbf{T = [Qn(Z^{5/3})/0.56(S_L^{1/2})]^{3/8}} \qquad \textbf{(4.5)}$$

Where: T = top width, or spread, of water flow (ft)

Q = discharge to the inlet structure (cfs)

Z = reciprocal of the transverse slope (ft/ft)

n = roughness coefficient

S_L = longitudinal street slope (ft/ft)

S_T = transverse street slope (ft/ft)

D=depth of water in gutter (ft)

4. To determine Q_{cap} of inlet:

$$\mathbf{Q_{cap} = KD^{5/3}}$$

Where: D = depth of water in gutter, upstream from the grate (ft)

Q_{cap} = discharge intercepted by grate (cfs)

K= grate inlet coefficient

Note: For S_L> 6%, use the curve for S_L= 6%

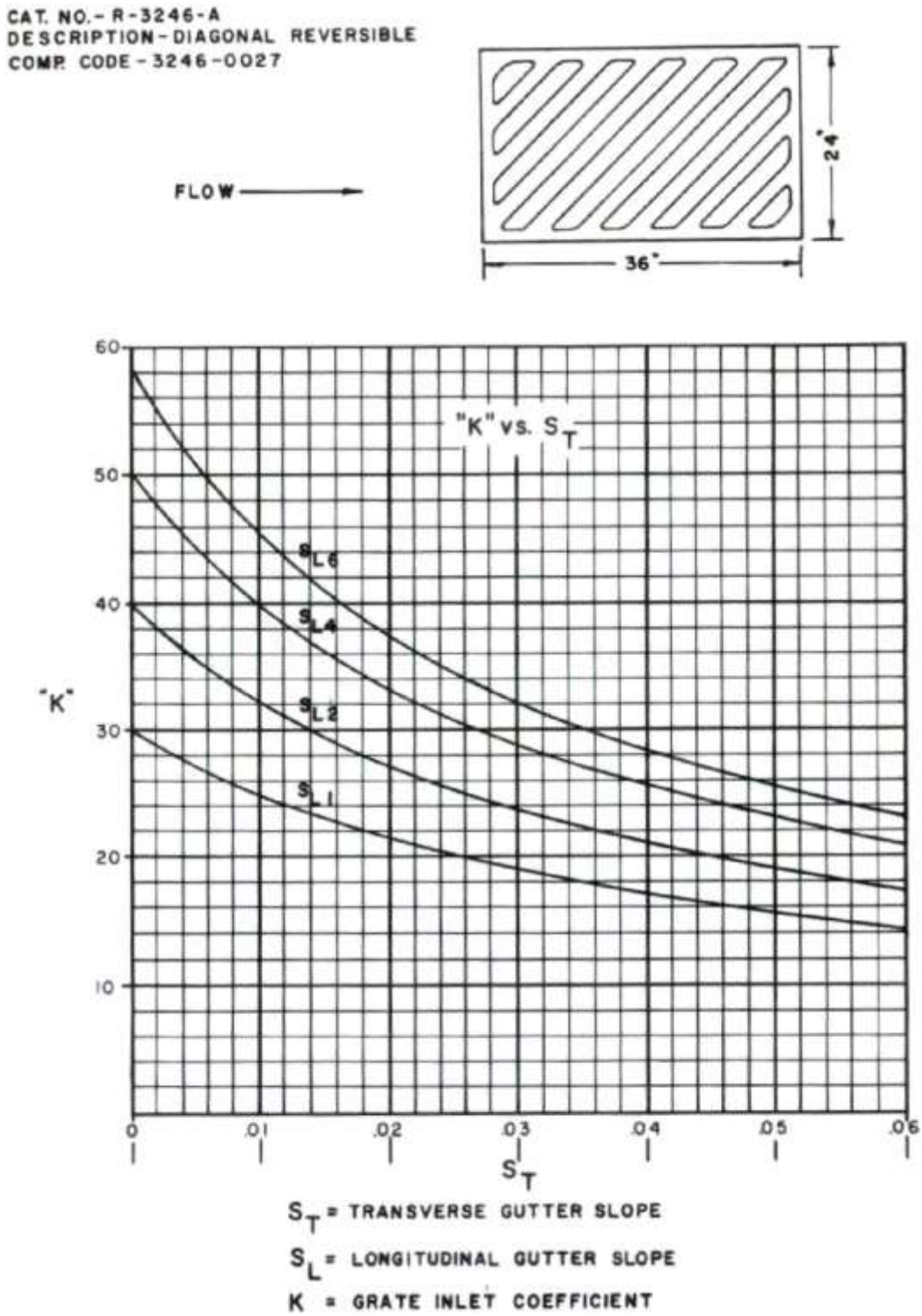


Figure 4-1 Grate Inlet Coefficient – On Grade

5. Once T is calculated and determined to be within its imposed limits, D can be calculated as follows:

$$\mathbf{D = T/Z} \qquad \textbf{(4.6)}$$

Where: D = depth of water in the gutter, upstream from the grate (ft)

6. The inlet capacity of the grate, Q_{cap}, can then be determined by using Figure 4-1. If 100% interception is not achieved, the overflow must be included in the next downstream inlet.

4.4.1.2 Example 1

A 10-year discharge of 4 cfs (3 cfs off-site and 1 cfs additional roadway drainage) drains to a residential street, with standard curb and gutter, in sheet flow and is to be intercepted in a catch basin midway down the street.

Given: S_L = 4% = 0.04 ft/ft

S_T = 3.125% = 0.03125 ft/ft

T_{max} allowable = 8 ft

Type G grate

Find: Determine whether this runoff is excessive and whether it can be handled by a single grate, type 'G', catch basin.

Solution: 1. $Z = 1/S_T = 1/0.03125 = 32$

$$\left(T = \frac{4\,(0.016)\,(32)^{5/3}}{0.56(0.04)^{1/2}} \right)^{3/8}$$

T = 7.1 ft (which is less than the 8 ft available)

Since T is lower than the maximum allowable top width of 8 ft, the runoff in the street is acceptable.

2. D = T/Z = 7.1/32 = 0.22 ft

3. From Figure 4-1, K = 28.5

Therefore: $\mathbf{Q_{cap} = KD^{5/3} = 28.5(0.22)^{5/3} = 2.3\ cfs}$ **(4.7)**

4. Q_{bypass} = 4.0 – 2.3 = 1.7 cfs

Thus, the grate will intercept 2.3 cfs and 1.7 cfs will continue downstream to another structure. If the next downstream structure can handle this 1.7 cfs in addition to any additional runoff that reaches this structure, then the design is adequate. If this additional 1.7 cfs will overload the next downstream structure, additional storm drainage structure can be added upstream.

4.4.2 Grate Inlets in Sag

Because a grate inlet in a sag condition is subject to clogging, a curb opening is required as a supplemental inlet. The capacity of a grate in a sag depends upon the area of the openings and the depth of water at the grate. Figure 4-2 can be used to calculate the head or flow for standard drop inlet grates, and Figure 4-3 for Type 'E' grates in sag or depressed conditions, .

4.4.2.1 Type 'E' Grate

For a type 'E' grate, the weir equation will control to a depth (D) of 0.69 feet. Refer to Figure 4-3. Because a depth of 0.69 feet could never be reached without flooding the street, only the weir equation will be used for the analysis of a street sag condition.

1. Knowing the Surface Q_{total}, solve for the required depth:

$$\mathbf{D} = \left(\frac{\textbf{Surface } Q_{total}}{3.3P} \right)^{2/3} \qquad \textbf{(4.8)}$$

P = 6.94 ft. (single type 'E' grate)
P = 9.92 ft. (double type 'E' grate)

2. Check the spread:

$$T = \frac{D}{S_T}$$

D = depth solved in #1 (ft)

S_T = transverse slope (ft/ft)

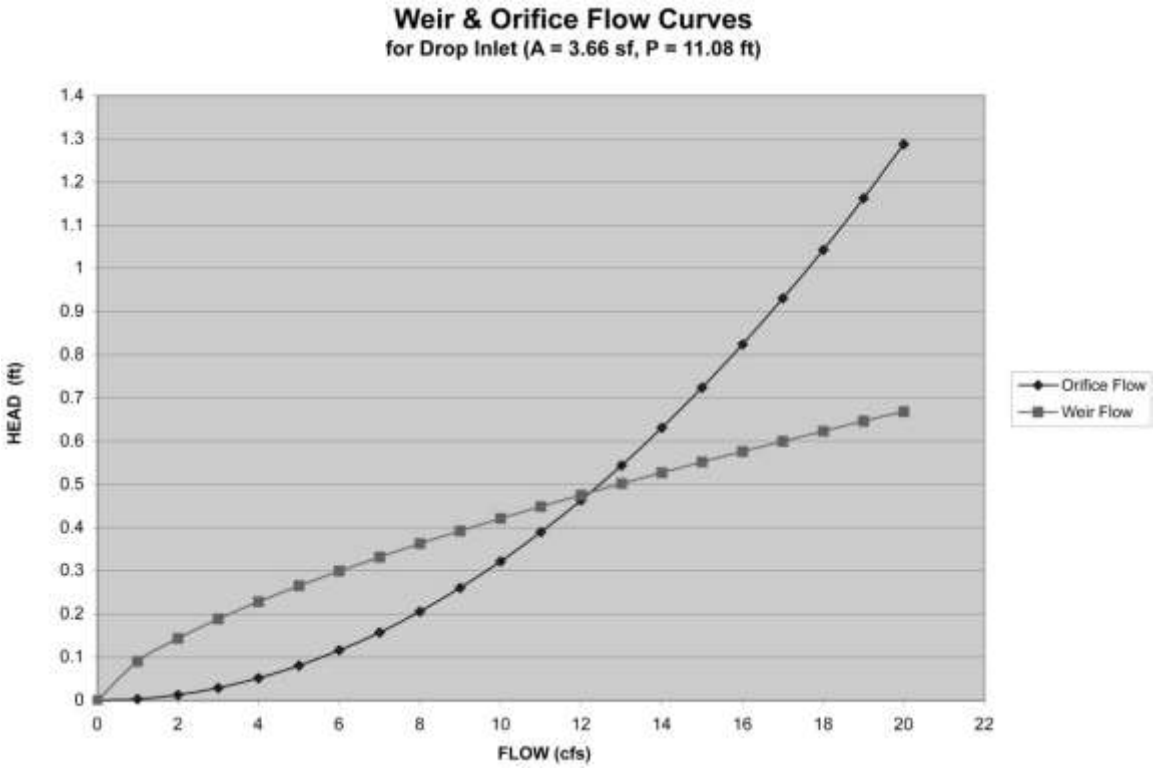


Figure 4-2 Standard Drop Inlet Grate NCDOT Std. 840.16 (2012)

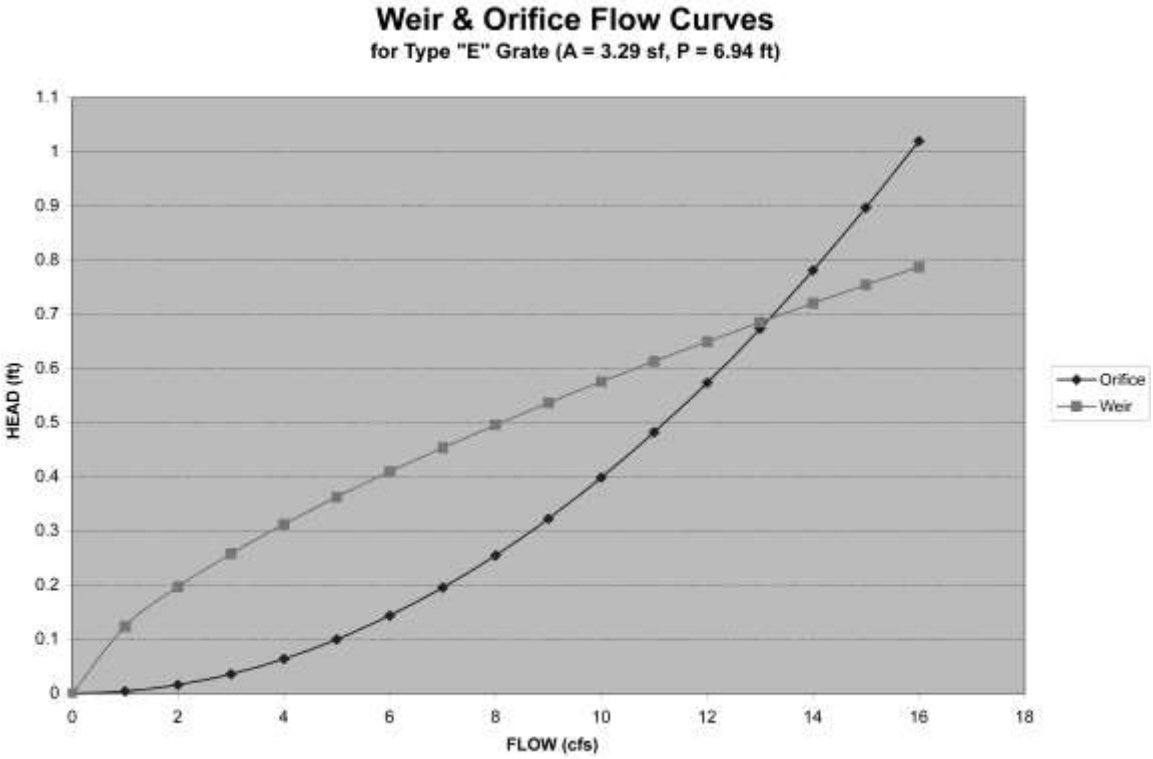


Figure 4-3 Type 'E' Grate NCDOT Std. 840.03 (2012)

4.4.2.2 Grated Drop Inlet

For a grated drop inlet, both the weir and orifice equation should be analyzed:

(orifice) $$D = \left(\frac{\text{Surface } Q_{total}}{(0.6)\ A\ (64.4)^{1/2}} \right)^2 \qquad (4.9)$$

A = 3.66 sq. ft. (open area of standard drop inlet grate-NCDOT Std. 840.16 (2006))

(weir) $$D = \left(\frac{\text{Surface } Q_{total}}{3.3P} \right)^{2/3} \qquad (4.8)$$

P = 11.08 ft. (perimeter of standard drop inlet grate–NCDOT Std. 840.16 (2006))

Solve both equations 4.8 and 4.9 for D, or refer to Figure 4-3. The larger D controls.

4.4.2.2.1 Example 2

Using the data given for Example 1 in 4.4.1.2, calculate the flow intercepted by type 'E' grate in a sag location.

Given: Same data given for Example 1 in 4.4.1.2

Find: Determine the depth of flow that can be intercepted by a type 'E' grade in a sag location.

Solution:

1. From Example 4.4.1.2, Surface Q_{total} = 4 cfs

2. Solve for D using Figure 4-3 or Equation 4-8

$$D = \left(\frac{4\ \text{cfs}}{3.3\ (6.94\ \text{ft})} \right)^{2/3} = 0.31\ \text{ft}$$

3. Check the spread

$$T = \frac{D}{S_t} = \frac{0.31\ \text{ft}}{0.03125\ \text{ft/ft}} = 9.9\ \text{ft}$$

4. This exceeds the maximum allowable spread of 8 ft. Install additional catch basins above the sag to intercept the additional flow, or try a double catch basin at the sag.

$$D = \left(\frac{4\ \text{cfs}}{3.3\ (9.92\ \text{ft})} \right)^{2/3} = 0.25\ \text{ft}$$

Check the spread

$$T = \frac{D}{S_t} = \left(\frac{0.25\ \text{ft}}{0.03125\ \text{ft/ft}} \right) = 7.9\ \text{ft}$$

This is less than the maximum allowable spread of 8 feet, so a double catch basin will work.

5. Finally, sag location would need to be evaluated in the appropriate storm (25-year for local roads and 50-year storm for thoroughfare roads) to ensure a minimum 8 foot passable lane for vehicles.

4.4.2.3 Open Throat (Slab Top) Catch Basin

Depth of flow for this type of inlet depends on how many sides will be open to flow and what the total length the openings are (L). Typical height for openings for this type of structure is 6 in (0.5 ft)[A]. Both weir and orifice equations should be analyzed to find depth at the inlet:

(weir) $$D = \left(\frac{\text{Surface } Q_{total}}{(3.3)\ (L)}\right)^{2/3}$$

If D> inlet opening height (.5 ft) then use the orifice equation to find the driving head (H) needed to pass the flow into the inlet instead

(orifice) $$H = \frac{[\text{Surface } Q_{total}]^2}{(0.6)\ (.5)\ (L)\ (64.4)^{1/2}}$$

since the driving head (H) is drawn to the centerline of the height of the inlet simply adding 0.25 ft to H will yield depth of flow (D)

$$D = H + (0.5/2)$$

4.4.2.3.1 Example

A four sided slab top open throated drop inlet is planned for a backyard. Surface flow is 25 cfs to this inlet. All four sides of the inlet have openings with each inlet side being 4 ft long (16 ft total). Since the height for inlet openings is 6 in the preceding equations can be used.

Analyzing weir flow for this inlet:

$$D = (25/(3.3*16))^{2/3} = 0.61 \text{ ft}$$

Since this is greater than the inlet opening height of 0.5 ft the orifice equation must be used to find depth of flow:

$$H = (25/(0.6*0.5*16*64.4^{1/2}))^2 = 0.42 \text{ ft}$$

Adding .25 ft to H to find depth of flow to the invert of the opening

$$D = 0.42 \text{ ft} + 0.25 \text{ ft} = 0.67 \text{ ft}$$

Depth of flow is 0.67 ft. Add required freeboard and grade accordingly to allow this ponding depth.

[A] *- For inlets with inlet opening heights different than 0.5 ft adjust these equations accordingly.*

4.5 COMBINATION INLETS

4.5.1 Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus, capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figure 4-1.

4.6 HYDRAULIC GRADIENT

4.6.1 Friction Losses

Energy losses from pipe friction may be determined by rewriting the Manning's equation.

$$\mathbf{S_f = [Qn/1.486\ A(R)^{2/3}]^2}$$

Then the head losses due to friction may be determined by the formula:

$$\mathbf{H_f = S_f L}$$

Where: H_f = friction head loss (ft)

S_f = friction slope (ft/ft)

L = length of outflow pipe (ft)

4.6.2 Velocity Head Losses

From the time storm water first enters the storm drainage system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as inlets, manholes, junctions, bends, contractions, enlargements and transitions, which will cause velocity head losses. Velocity losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisbach Equations.

$$\mathbf{H = KV^2/2g} \qquad \textbf{(4.10)}$$

Where: H = velocity head loss (ft)

K = loss coefficient for the particular structure

V = velocity of flow (ft/s)

g = acceleration due to gravity (32.2 ft/s)

4.6.3 Entrance Losses

Following are the equations used for entrance losses for beginning flows.

$$\mathbf{H_{tm} = V^2/2g} \qquad \textbf{(4.11)}$$

$$\mathbf{H_e = KV^2/2g} \qquad \textbf{(4.12)}$$

Where: H_{tm} = terminal (beginning of run) loss (ft)

H_e = entrance loss for outlet structure (ft)

K = 0.5 (assuming square-edge)

(Other terms defined above)

4.6.4 Junction Losses

Incoming Opposing Flows

The head loss at a junction, H_{j1} for two almost equal and opposing flows meeting "head on" with the outlet direction perpendicular to both incoming directions, head loss is considered as the total velocity head of outgoing flow.

$$\mathbf{H_{j1} = (V_3^2)\ (outflow)/2g} \qquad \textbf{(4.13)}$$

Where: H_{j1} = junction losses (ft)

(Other terms are defined above.)

Changes in Direction of Flow

When main storm drain pipes or lateral lines meet in a junction, velocity is reduced within the chamber and specific head increases to develop the velocity needed in the outlet pipe. The sharper the bend (approaching 90°) the more severe this energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber by the formula:

$$\mathbf{H_b = K(V^2)\ (outlet)/2g} \qquad \textbf{(4.14)}$$

Where: H_b = bend head loss (ft)

K = junction loss coefficient

Table 4-2 below lists the values of K for various junction angles.

Table 4-2

Values of K for Change in Direction of Flow in Lateral

K	Degrees of Turn (In Junction)
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater

K values for other degree of turns can be obtained by interpolating between values.

Several Entering Flows

The computation of losses in a junction with several entering flows utilizes the principle of conservation of energy. For a junction with several entering flows, the energy content of the inflows is equal to the energy content of outflows plus additional energy required by the collision and turbulence of flows passing through the junction.

The total junction losses at the sketched intersections are as follows:

The following equation can be used to calculate these losses:

$$\mathbf{H_{j2} = [(Q_4V_4^2) - (Q_1V_1^2) - (Q_2V_2^2) + (KQ_1V_1^2)]/(2gQ_4)} \qquad \mathbf{(4.15)}$$

Where: H_{j2} = junction losses (ft)

Q = Discharge (cfs)

V = horizontal velocity (ft/s)
(V_3 is assumed to be zero)

g = acceleration due to gravity (32.2 ft/s^2)

K = bend loss factor

Where subscript nomenclature is as follows:

Q_1 = 90° lateral (cfs)

Q_2 = straight through inflow (cfs)

Q_3 = vertical dropped-in flow from an inlet (cfs)

Q_4 = main outfall = total computed discharge (cfs)

V_1, V_2, V_3, V_4 = horizontal velocities of foregoing flows, respectively (ft/s)

Also assume: $H_b = K(V_1^2)/2g$ for change in direction.

No velocity head of an incoming line is greater than the velocity head of the outgoing line.

The water surface of inflow and outflow pipes in junction are level.

When losses are computed for any junction condition for the same or a lesser number of inflows, the above equation will be used with zero quantities for those conditions not present. If more directions or quantities are at the junction, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

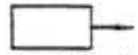
4.6.5 Summary

The final step in designing a storm drain system is to check the hydraulic grade line (HGL) as described in the next section of this chapter. Computing the HGL will determine the elevation, under design conditions, to which water will rise in various inlets, manholes, junctions, and etc.

Figure 4-4 on the following page is a summary of energy losses which should be considered. Following this in Figure 4-5 is a sketch showing the proper and improper use of energy losses in developing a storm drain system.

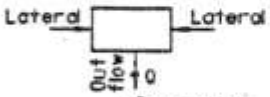
$$H_{tm} = \frac{v^2}{2g}$$

TERMINAL LOSSES
(at beginning of run)
Where g = gravitational constant. 32.2 feet per second per second.



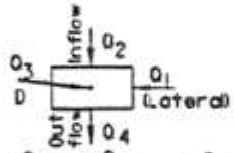
$$H_e = \frac{0.5\,v^2}{2g}$$

ENTRANCE LOSSES
(at end of run)
Assuming square - edge



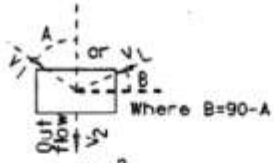
$$H_{j1} = \frac{v^2}{2g}\text{(Outflow)}$$

JUNCTION LOSSES
(Incoming-opposing Flow)
Use only where flows are identical to above, otherwise use H_{j2} Equation.



$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4}$$

JUNCTION LOSSES
(Several Entering Flows)
Total losses to include H_{j2} plus losses for changes in direction of less than 90° (h_b).
Where K=Bend loss coefficient
Q_3=Vertical dropped-in flow from an inlet
V_3=Assumed to be zero



$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(changes in direction of flow)

Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSSES $(H)_f$

$H_f = S_f \times L$

Where H_f=friction head loss
S_f=friction slope
L =length of conduit

$$\text{Where } S = \left(\frac{Qn}{1.486AR^{\frac{2}{3}}}\right)^2$$

Q=Discharge of conduit
n=Mannings coefficient of roughness
A=area of conduit
R=hydraulic radius of conduit

TOTAL ENERGY LOSSES AT EACH JUNCTION

$$H_T = H_{tm} + H_e + (H_{j1} \text{ or } H_{j2}) + H_b + H_f$$

Figure 4-4 Summary of Energy Losses
Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt32.pdf

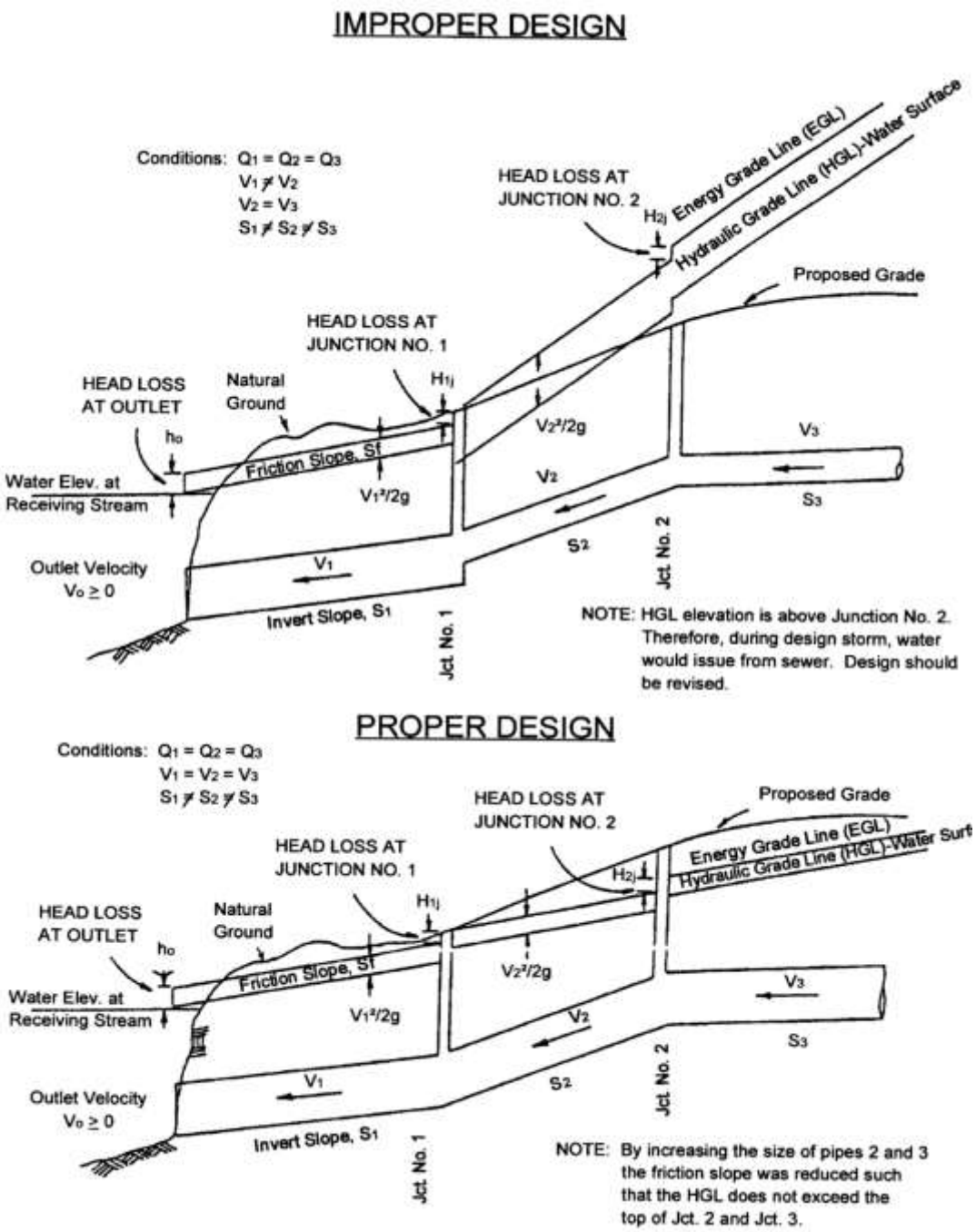


Figure 4-5 Energy and Hydraulic Grade Lines for Storm Drainage Under Constant Discharge
Source: www.ct.gov/dot/lib/dot/documents/ddrainage/11.12.pdf

4.7 STORM DRAIN

4.7.1 Introduction

After the tentative locations of inlets, drain pipes, and outfalls with tailwaters have been determined and the inlets have been sized, the next logical step is the computation for the rate of discharge to be carried by each drain pipe and the determination of the size and gradient of pipe required to convey this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the drain pipe serving that discharge is sized, and the process is repeated for the next run downstream. It should be recognized that the rate of discharge to be carried by any particular section of drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

For ordinary conditions, drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning's equation is recommended for capacity calculations.

4.7.2 Design Criteria

The standard maximum and minimum slopes for storm drains should conform to the following criteria:

1. The maximum pipe velocity shall not exceed 20 feet per second, or 10 feet per second in corrugated metal pipe.

2. The maximum discharge velocity at pipe outlets is 10 fps except for pipes greater than 48 inches in diameter unless velocity is further restricted for energy dissipation.

3. The minimum allowable slope is 0.5 percent or the slope which will produce a velocity of 2.5 feet per second when the storm drainage system is flowing full, whichever is greater.

Systems should generally be designed for non-pressure conditions by using the Manning's equation. When hydraulic calculations do not consider minor energy losses such as expansion, contraction, bend, junction, and manhole losses, the elevation of the hydraulic gradient for design flood conditions should be at least 1.0 foot below surface inlet elevation. As a general rule, minor losses should be considered when the pipe velocity exceeds 6 feet per second (lower if flooding could cause critical problems). If all minor energy losses are accounted for, it is usually acceptable for the hydraulic gradient to reach 6 inches below the grate elevation.

4.7.3 Capacity

Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's equation and it is expressed below:

$$\mathbf{V = [1.486\ R^{2/3}S^{1/2}]/n} \qquad \textbf{(4.16)}$$

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

S = the slope of hydraulic grade line (ft/ft)

n = Manning's roughness coefficient (see Table 4-3)

In terms of discharge, the above formula becomes:

$$\mathbf{Q = [1.486\ AR^{2/3}S^{1/2}]/n} \qquad \textbf{(4.17)}$$

Where: Q = rate of flow (cfs)

A = cross sectional area of flow (ft^2)

For pipes flowing full, the above equations become:

$$\mathbf{V = [0.590\ D^{2/3}S^{1/2}]/n} \qquad \textbf{(4.18)}$$

$$\mathbf{Q = [0.463\ D^{8/3}S^{1/2}]/n} \qquad \textbf{(4.19)}$$

Where: D = diameter of pipe (ft)

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$\mathbf{H_f = (2.88n^2V^2L)/D^{4/3}} \qquad \textbf{(4.20)}$$

$$\mathbf{H_f = (29\ n^2LV^2)/(R^{4/3})(2g)} \qquad \textbf{(4.21)}$$

Where: H_f = total head loss due to friction (ft)

L = length of pipe (ft)

g = acceleration due to gravity = 32.2 ft/sec^2

Table 4-3
Manning's *n* Values

Type of Conduit	Wall and Joint Description	Manning's *n* value
Concrete Pipe & Boxes	Good joints, smooth walls	0.012-0.013
Corrugated Metal Pipe & Boxes Annular Corrugations	68 by 13 mm 2- 2/3 by ½ inch corrugations	0.024
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	68 by 13 mm 2-2/3 by ½ inch corrugated, 24 inch plate width	0.021

Note: For futher information, consult Hydraulic Design of Highways Culverts, Federal Highway Administration, HDS No. 5, page 33.

4.7.4 Hydraulic Grade Lines

In calculating the hydraulic grade line within a closed storm drainage system, all head losses shall be computed to determine the water surface elevation within various structures.

The calculations are begun at the upstream or downstream opening, dependent upon whether the pipe is in inlet or outlet control. If it is inlet control the hydraulic grade line is the headwater elevation minus the entrance loss and the difference in velocity head. If the outlet controls, the tail water surface elevation or 0.8 times the diameter of the pipe, wherever is higher, is the outlet hydraulic grade line. Hydraulic grade lines will be required only as requested on a case by case basis.

If computer models are utilized then results should be consistent with the procedure outlined below. All input data should be supplied, including loss coefficients, and output should be in similar format to Figure 4-6.

4.7.4.1 Design Procedure

The head losses are calculated beginning from the downstream control point to the first junction upstream, and the procedure is repeated for the next junction. The computation for outlet control may be tabulated using Figure 4-6 and the following procedure:

1. Enter in Column 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.

2. Enter in Column 2 the outlet water surface elevation or 0.8 diameter plus invert out elevation of the outflow pipe whichever is greater.

3. Enter in Column 3 the diameter (D_o) of the outflow pipe.

4. Enter in Column 4 the design discharge (Q_o) for the outflow pipe.

5. Enter in Column 5 the length (L_o) of the outflow pipe.

6. Enter in Column 6 the friction slope (S_f) in ft/ft of the outflow pipe. This can be determined from the following formula:

$$\mathbf{S_f = \{Qn/1.486AR^{2/3}\}^2} \qquad \textbf{(4.22)}$$

7. Multiply the friction slope (S_f) in Column 6 by the length (L_o) in Column 5 and enter the friction loss (H_f) in Column 7.

8. Enter in Column 8 the velocity of the flow (V_o) of the outflow pipe.

9. Enter in Column 9 the contraction loss (H_o) by using the formula:

$$\mathbf{H_o = 0.25\ (V_o^2)/2g} \qquad \textbf{(4.23)}$$

Where: $g = 32.2\ ft/s^2$

10. Enter in Column 10 the design discharge (Q_j) for each pipe flowing into the junction. Neglect lateral pipes with inflows of less than ten percent of the mainline outflow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.

11. Enter in Column 11 the velocity of flow (V_i) for each pipe flowing into the junction (for exception see Step 10).

12. Enter in Column 12 the product of (Q_i x V_i) for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest (Q_i x V_i) product is the line which will produce the greatest expansion loss (H_i). (For exception, see step 10).

13. Enter in Column 13 the controlling expansion loss (H_i) using the formula $H_i = 0.35\ (V_i^2)/2g$.

14. Enter in Column 14 the angle of skew of each inflowing pipe to the outflow pipe (for exception, see Step 10).

15. Enter in Column 15 the greatest bend loss (H_Δ) calculated by using the formula:

$$\mathbf{H_\Delta = KV_i^2/2g} \qquad \textbf{(4.24)}$$

Where: K = the bend loss coefficient corresponding to the various angles of skew of the inflowing pipes.

16. Enter in Column 16 the total head loss (H_t) by summing the values in Column 9 (H_o), Column 13 (H_i) and Column 15 (H_Δ).

17. If the junction incorporates adjusted surface inflow of ten percent or more of the mainline outflow, i.e., drop inlet, increase H_t by 30 percent and enter the adjusted H_t in Column 17.

18. If the junction incorporates full diameter inlet shaping, such as standard manholes, reduce the value of H_t by 50 percent and enter the adjusted value in column 18.

19. Enter in Column 19 the FINAL H, the sum of H_f and H_t, where H_t is the final adjusted value of the H_t.

20. Enter in Column 20 the sum of the elevation in Column 2 and the final H in Column 19. This elevation is the potential water surface elevation for the junction under design conditions.

21. Enter in Column 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Column 20. If the potential water surface elevation exceeds the rim elevation or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the H.G.L.

22. Repeat the procedure starting with Step 1 for the next junction upstream.

4.7.5 Minimum Grade

The minimum allowable slope is 0.5 percent or the slope which will produce a velocity of 2.5 feet per second when the storm drainage system is flowing full, whichever is greater.

The minimum slopes are calculated by the modified Manning's equation:

$$\mathbf{S = (nV)^2/(2.208\ R^{4/3})} \qquad \mathbf{(4.25)}$$

4.7.6 Design Procedures

The design of storm drain systems is generally divided into the following operations:

1. The first step is the determination of inlet location and spacing as outlined earlier in this chapter.

2. The second step is the preparation of a plan layout of the storm drainage system establishing the following design area:

    a. Location of storm drains
    b. Direction of flow
    c. Location of manholes
    d. Location of existing facilities such as water, gas, or underground cables

3. The design of the storm drain system is then accomplished by determining drainage areas, computing runoff using the rational method, and computing the hydraulic capacity using Manning's equation. As a final step the hydraulic performance of the system can be checked by calculating the hydraulic grade line.

4. The storm drain design computation sheet (Figure 4-6) and hydraulic grade line computation form (Figure 4-7) can be used to summarize the design computations.

Structure		Length (feet)	Drainage Area "A" (acres)		Runoff Coefficient "c"	"A" x "C"		Flow Time Min		Rainfall Intensity (inches/hour)	Total Runoff (cfs)	Pipe Diameter (inches)	Full Capacity (cfs)	Velocity fps		Invert Elev		Manhole Invert Drop	Slope of Sewer (ft/ft)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

Figure 4-6 Storm Drainage Computation Form

Hydraulic Grade Line

Project _____

Station	Outlet Water Surface Elev.	D_o	Q_o	L_o	S_f	H_f	Junction Loss												Final H	Inlet Water Surface Elev	Rim Elev
							V_o	H_o	Q_1	V_1	$Q_1 V_1$	$\frac{V_1^2}{2g}$	H_1	Angle	H_Δ	H_t	$H_t^{1.3}$	$H_t^{0.6}$			
1	2	3	4	5	6	7	8	9	10	11	12		13	14	15	16	17	18	19	20	21

Final $H = H_f + H_t$

$H_1 = 0.35\ (V_1^2/2g)$ $H_o = 0.25\ (V_o^2/2g)$ $H_\Delta = K\ (V_1^2/2g)$ $H_t = H_1 + H_o + H_\Delta$

90° K = 0.70	45° K = 0.47
75° K = 0.64	30° K = 0.35
60° K = 0.56	15° K = 0.19

Figure 4-7 Hydraulic Grade Line Computation Form

4.7.7 Rational Method Examples

The following example will illustrate the hydrologic calculations needed for storm drain design using the rational formula (see Hydrology Chapter for Rational Method description and procedures). Figure 4-8 shows a hypothetical storm drain system that will be used in this example. Following is a tabulation of the data needed to use the rational equation to calculate inlet flow rate for the seven inlets shown in the system layout.

Table 4-4
Hydrologic Data

Inlet[a]	Drainage Area (acres)	Time of Concentration (minutes)	Rainfall Intensity (inches/hour)	Runoff Coefficient	Inlet Flow Rate[b] (cfs)
1	2.0	8	6.26	.9	11.3
2	3.0	10	5.84	.9	15.8
3	2.5	9	6.04	.9	13.6
4	2.5	9	6.04	.9	13.6
5	2.0	8	6.26	.9	11.3
6	2.5	9	6.04	.9	13.6
7	2.0	8	6.26	.9	11.3

[a] Inlet and storm drain system configuration are shown in Figure 4-8
[b] Calculated using the Rational Equation (see Chapter 2, Hydrology)

The following table shows the data and results of the calculation needed to determine the design flow rate in each segment of the hypothetical storm drain system.

Table 4-5
Storm Drain System Calculations

Storm Drain Segment	Tributary Area (acres)	Time of Concentration (minutes)	Rainfall Intensity (inches/hour)	Runoff Coefficient	Design Flow Rate (cfs)
I_1-M_1	2.0	8	6.26	.9	11.3
I_2-M_1	3.0	10	5.84	.9	15.8
M_1-M_2	5.0	10.5	5.76	.9	25.9
I_3-M_2	2.5	9	6.04	.9	13.6
I_4-M_2	2.5	9	6.04	.9	13.6
M_2-M_3	10.0	11.5	5.60	.9	50.4
I_5-M_3	2.0	8	6.26	.9	11.3
I_6-M_3	2.5	9	6.04	.9	13.6
M_3-M_4	14.5	13.5	5.27	.9	68.8
I_7-M_4	2.0	8	6.26	.9	11.3
M_4-O	16.5	14.7	5.08	.9	75.4

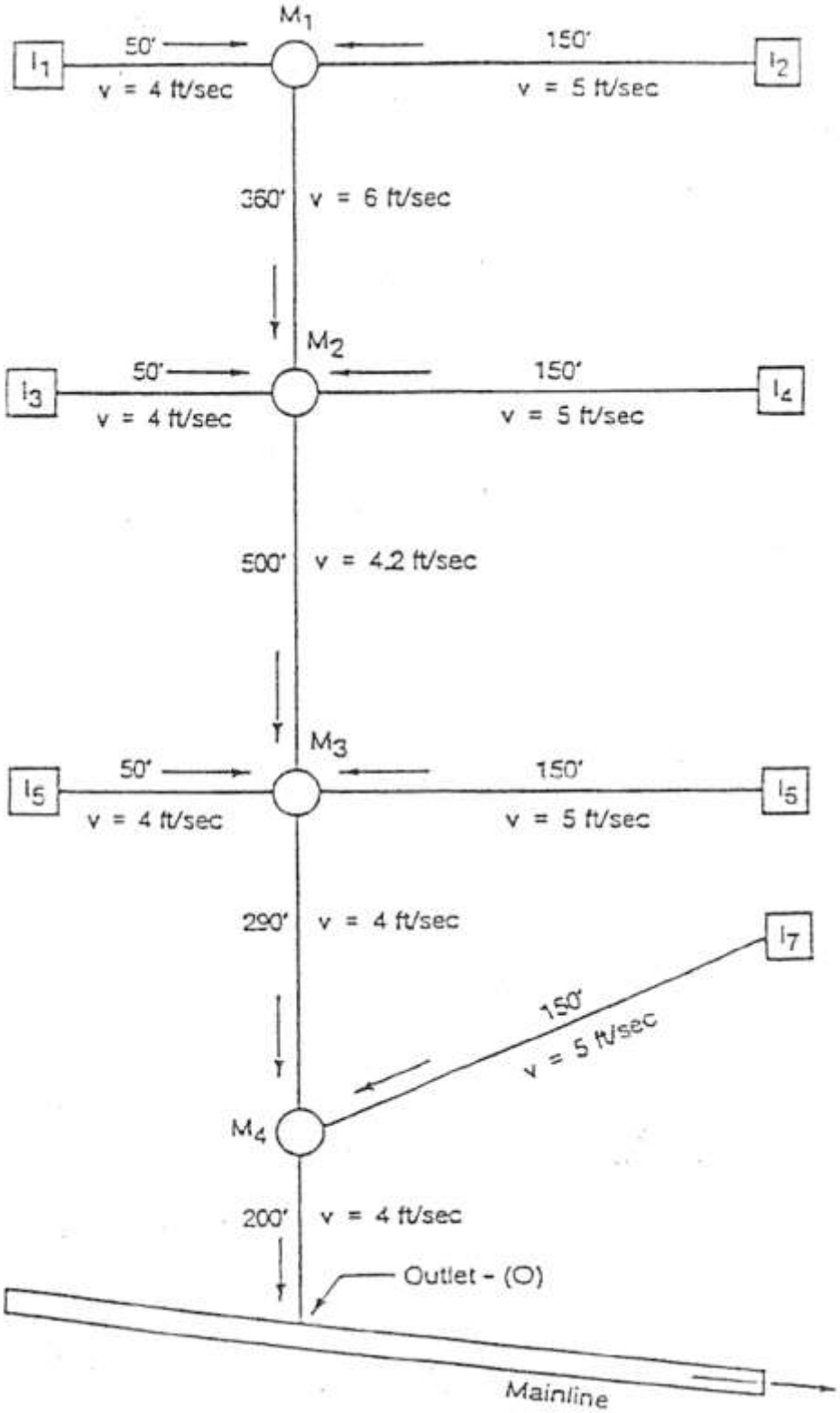


Figure 4-8 Hypothetical Storm Drain System Layout
Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt32.pdf

CHAPTER 5

DESIGN OF CULVERTS

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5.1 OVERVIEW

5.1.1 Definitions

Culverts are structures used to convey surface runoff from one side of the road to another and are usually covered with an embankment composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. For economy and hydraulic efficiency, culverts should be designed to operate with the inlet submerged during flood flows, if conditions permit. Cross-drains are those culverts and pipes that are used to convey runoff from one side of a roadway to another.

5.1.2 Performance Curves

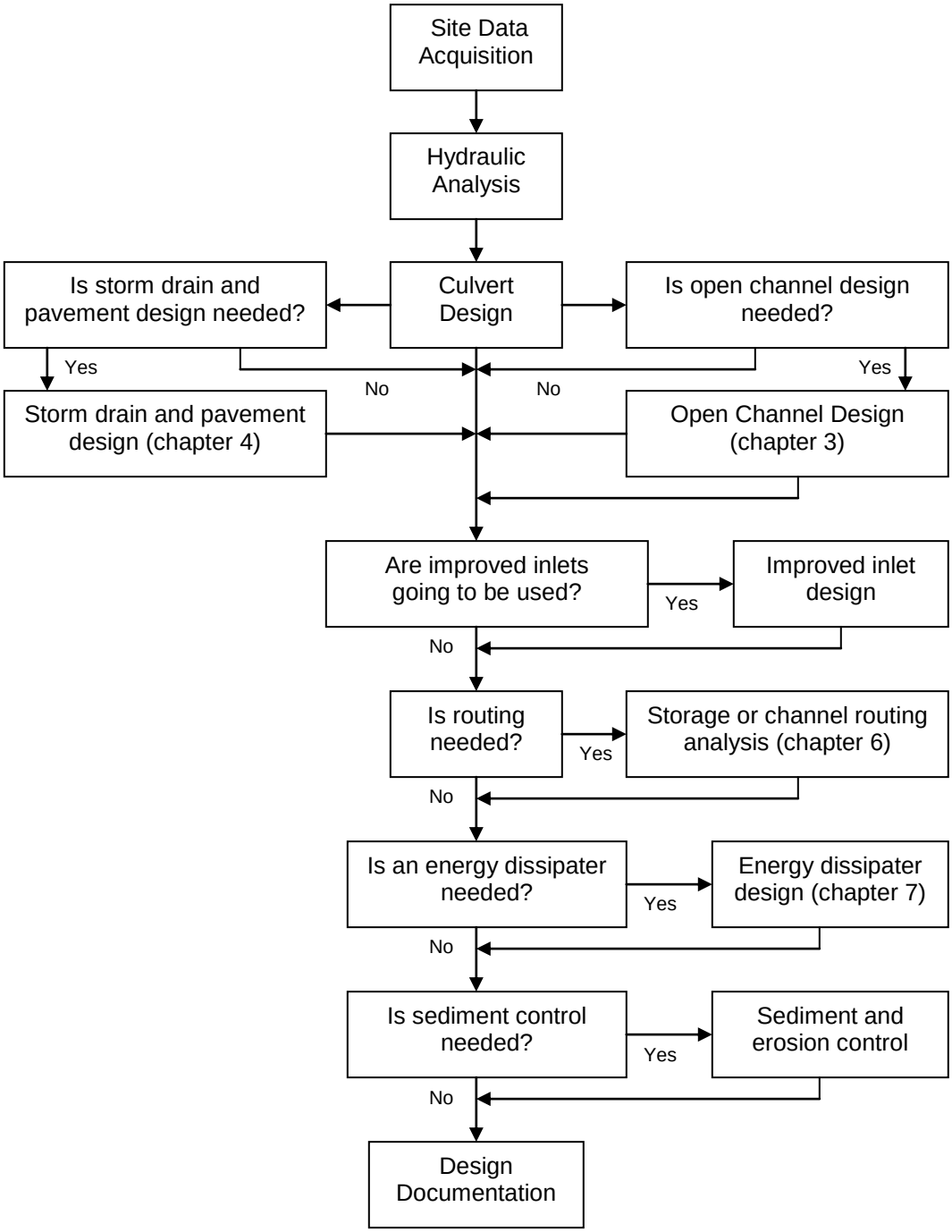
Performance curves should be developed for all culverts for evaluating the hydraulic capacity of a culvert for various headwaters. These curves will display the consequences of high flow rates at the site and any possible hazards. Sometimes a small increase in flow rate can affect a culvert design. If only the design peak discharge is used in the design, the engineer cannot assess what effect increases in the essential design discharge will have on the culvert design.

5.2 CULVERT DESIGN PROCEDURE FLOWCHART

5.2.1 Purpose and Use

The purpose of the culvert design procedure flow chart is to show the relationship between the different stages in culvert design and the alternatives that should be considered.

5.2.2 Design Flowchart



5.3 ENGINEERING DESIGN CRITERIA

5.3.1 Introduction

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs where applicable.

5.3.2 Criteria

Engineering Aspects

- Flood frequency
- Velocity limitations

Site Criteria

- Needed length and available slope
- Debris control

Design Limitations

- Headwater
- Tailwater conditions
- Ground cover
- Utility conflicts
- Regulated floodway requirements

Design Options

- Culvert inlets
- Inlets with headwalls
- Wingwalls and aprons
- Improved inlets

Material selection

- Culvert skews
- Culvert sizes

Other Design Considerations

- Weep holes
- Outlet protection
- Erosion and sediment control
- Environmental considerations

Some culvert designs are relatively simple involving a straight-forward determination of culvert size and length. Other designs are more complex where structural, hydraulic, environmental, or other considerations must be evaluated and provided for in the final design.

The following sections discuss each of the above criteria as it relates to culvert siting and design.

5.3.3 Flood Frequency

The appropriate flood frequency for determining the flood carrying-capacity of a culvert is dependent upon:

- the level of risk associated with failure of the culvert crossing; and
- the level of risk associated with increasing the flood hazard to upstream (backwater) or downstream (redirections of floodwaters or loss of attenuation) properties.

For specific design storm frequencies for culvert crossings, reference Section 2.3.1, Design Frequencies. Also, in compliance with the National Flood Insurance Program, it is necessary to consider the 100-year frequency flood at locations identified as being special flood hazard areas. The design engineer should review the City and County floodway regulations for more information related to floodplain regulations.

5.3.4 Velocity Limitations

Both minimum and maximum velocities should be considered when designing a culvert. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. As outlet velocities increase, the need for stabilization at the culvert outlet increases. If velocities exceed permissible velocities for the various types of nonstructural outlet lining material available, the installation of structural energy dissipaters is required.

5.3.5 Debris Control

In designing debris control structures it is recommended that the U.S. Army Corps of Engineers, Hydraulic Engineering Circular No. 9 entitled “Debris – Control Structures” be consulted.

5.3.6 Headwater Limitations

The allowable headwater elevation is determined from an elevation of land use upstream of the culvert and the proposed or existing roadway elevation. Headwater is the depth of water above the culvert invert at the entrance end of the culvert.

The following criteria related to headwater shall be used (based on the design storm event):

- The allowable headwater for design frequency conditions should allow for the following upstream controls:
  - 12-inch freeboard for culverts up to 3 feet in diameter
  - 18-inch freeboard for culverts larger than 3 feet in diameter
  - Upstream property damage
  - Elevations established to delineate flood plain zoning
  - Low point in the road grade that is not at the culvert location
  - Ditch elevation of the terrain that will permit flow to divert around culvert
  - $HW/D \leq 1.2$
- The headwater should be checked for the 100-year flood to ensure compliance with the locally adopted floodway ordinance and 100 + 1 criteria.
- The maximum acceptable outlet velocity should be identified. Either the headwater should be set to produce acceptable velocities, or stabilization or energy dissipation should be provided.

- Other site-specific design considerations should be addressed as required.
- In general, the constraint which gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.

If there is insufficient headwater elevation to convey the required discharge, it will be necessary to either use a larger culvert, lower the inlet invert, use an irregular cross-section, use an improved inlet if in inlet control, multiple barrels, or use a combination of these measures. If the inlet invert is lowered, special consideration must be given to scour.

5.3.7 Tailwater Considerations

The hydraulic conditions downstream of the culvert site shall be evaluated to determine a tailwater depth for a range of discharge. At times there may be a need for calculating backwater curves to establish the tailwater conditions.

If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line should be determined.

For culverts which discharge to an open channel, the tailwater depth is the normal depth for the design storm in an open channel. See Chapter 3, Open Channel Hydraulics.

If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.

If the culvert discharges to a lake, pond, or other major water body, the expected high water elevation for the culvert design frequency of the particular water body may establish the culvert tailwater.

5.3.8 Culvert Inlets

Selection of the inlet type is an important part of culvert design, particularly with inlet control. Hydraulic efficiency and cost can be significantly affected by inlet conditions.

The inlet coefficient K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. All methods described in this chapter, directly or indirectly, use inlet coefficients. Inlet coefficients are given in the table shown on the next page.

Table 5-1
Inlet Coefficients

Type of Structure and Design of Entrance	Coefficients K_e
PIPE, CONCRETE	
Projecting from fill, socket end (groove-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” or 45” bevels	0.2
Side- or slope-tapered inlet	0.2
PIPE, OR PIPE-ARCH, CORRUGATED METAL	
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” or 45” 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° to 75° 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° or 25° 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

*Note: Concrete end sections conforming to fill slope 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. Some end sections, incorporate a closed taper in their design to yield more efficient hydraulic performance.

5.3.9 Inlets with Headwalls

Headwalls may be used for a variety of reasons:

1) Increasing the efficiency of the inlet
2) Providing embankment stability
3) Providing embankment protection against erosion
4) Providing protection from buoyancy
5) Shorting the length of the required structure

The relative efficiency of the inlet depends on the conduit. Structures or flared end sections are required at all inlets and outlets of all pipe systems. Headwalls are required for all metal and HDPE culverts. Concrete flared end sections may be utilized in lieu of headwalls upon approval of the City Engineer. The figure below illustrates the use of headwalls and wingwalls.

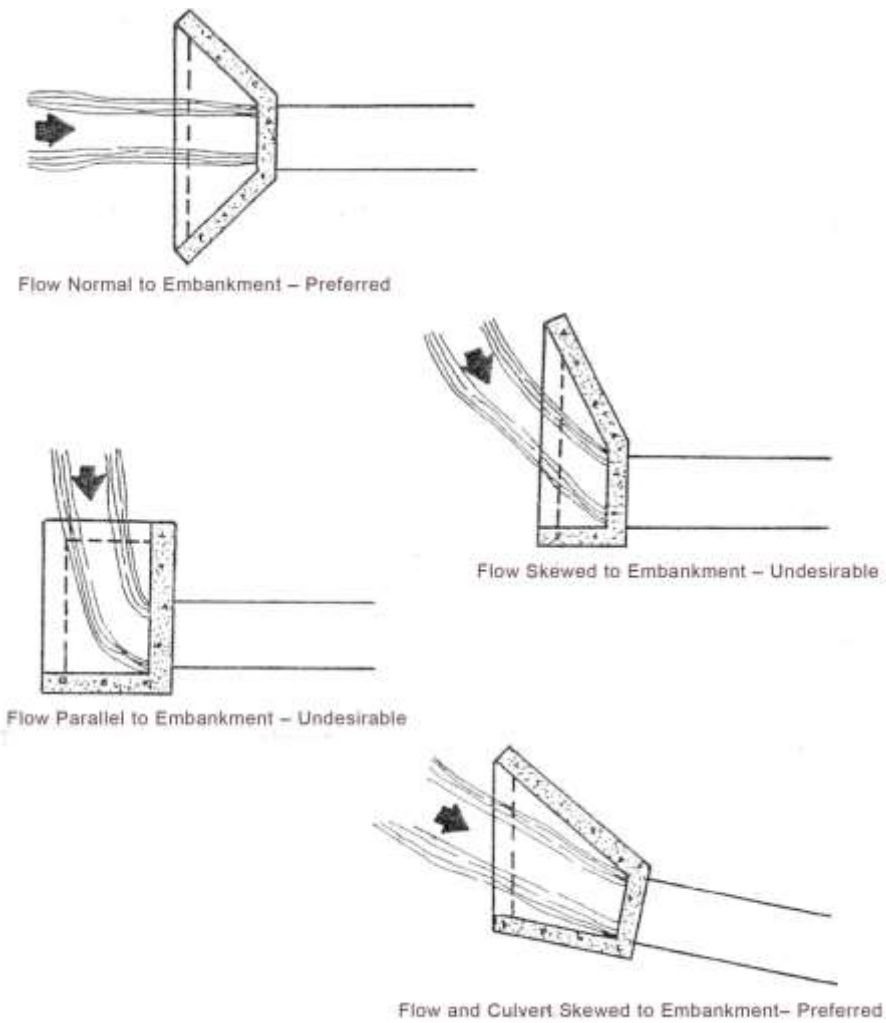
Corrugated metal pipe in a headwall is essentially square-edged with an inlet coefficient of about 0.5.

The primary reasons for using headwalls are for embankment protection, buoyancy control, and ease of maintenance.

5.3.10 Wingwalls and Aprons

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

Little increase in hydraulic efficiency is realized with the use of wingwalls, regardless of the pipe material used, and, therefore, the use should be justified for other reasons. Wingwalls can be used to increase hydraulic efficiency if designed as a side-tapered inlet (See Appendix 5B for more information on the design of side-tapered inlets). The figure shown below illustrates several uses of wingwalls.



Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt4.pdf

If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall. This apron should extend at least the barrel depth upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

5.3.11 Improved Inlets

Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance at the culvert. For these designs refer to Appendix 5B which describes the design of improved inlets.

5.3.12 Material Selection

For culvert selection, only reinforced concrete pipe is allowed within the street right-of-way except for culverts equal to or greater than 60 inches. For culverts equal to or greater than 60 inches in diameter, aluminum or aluminized steel pipe is allowed.

5.3.13 Outlet Protection

See Chapter 7, Energy Dissipation, for information on the design of outlet protection.

5.3.14 Environmental Considerations

In addition to controlling erosion, sedimentation and debris at the culvert site, care must be exercised in selecting the location of the culvert site. Environmental considerations are a very important aspect of culvert selection and design.

This selection must consider the entire site and include provisions for maintaining existing stream cross section at the inlet and outlet while providing passage of aquatic life.

5.4 CULVERT FLOW AND CONTROLS AND EQUATIONS

5.4.1 Introduction

Generally, the hydraulic control in a culvert will be at the culvert outlet if the culvert is operating on a mild slope. Entrance control usually occurs if the culvert is operating on a steep slope.
For outlet control, the head losses due to tailwater and barrel friction are predominant in controlling the headwater of the culvert. The entrance will allow the water to enter the culvert faster than the backwater effects of the tailwater, and barrel friction will allow it to flow through the culvert.

For inlet control, the entrance characteristics of the culvert are such that the entrance head losses are predominant in determining the headwater of the culvert. The barrel will carry water through the culvert more efficiently than the water can enter the culvert.

Each culvert flow, however classified, is dependent upon one or both of these controls; due to the importance of these controls, further discussion follows.

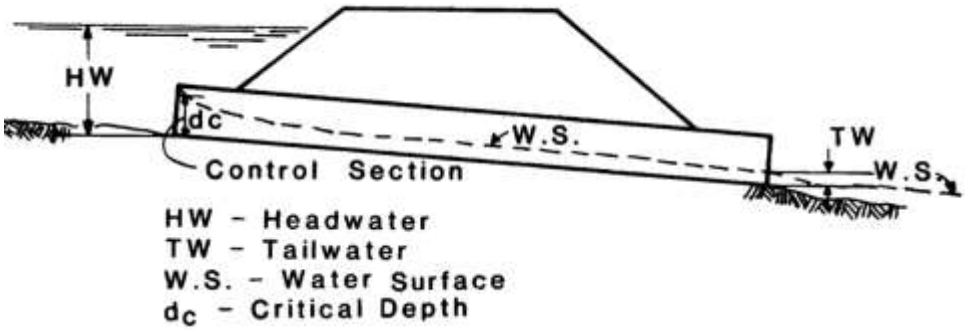
5.4.2 Inlet and Outlet Control

Inlet Control - If the culvert is operating on a steep slope, it is likely that the entrance geometry will control the headwater and the culvert will be on inlet control.

Outlet Control - If the culvert is operating on a mild slope, the outlet will probably control the flow and the culvert will be on outlet control.

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control see the Federal Highway Administration publication entitled Hydraulic Design of Highway Culverts, HDS-5, 2005.

The following diagram illustrates the terms and dimensions used in the culvert headwater equations.



Source: http://isddc.dot.gov/OLPFiles/FHWA/012545.pdf

5.4.3 Equations

There are many combinations of conditions which may classify a particular culvert's hydraulic operation. By consideration of a succession of parameters, the engineer may arrive at the appropriate calculation procedure. The most common types of culvert operations for any barrel type are classified as follows.

5.4.3.1 Mild Slope

Critical Depth – Outlet Control – The entrance is unsubmerged (HW ≤ 1.5D), the critical depth is less than uniform depth at the design discharge (dc < du), and the tailwater is less than or equal to critical depth (TW ≤ dc). This condition is a common occurrence where the natural channels are on flat grades and have wide, flat flood plains. The control is critical depth at the outlet.

$$\mathbf{HW = d_c + V_c^2/(2g) + H_e + H_f - SL} \qquad \textbf{(5.1)}$$

Where: HW = Headwater depth (ft)

d_c = critical depth (ft)

V_c = critical velocity (ft/sec)

g = 32.2 ft/sec^2

H_e = entrance headloss (ft)

H_f = friction headloss (ft)

S = slope of culvert (ft/ft)

L = length of culvert (ft)

Tailwater Depth – Outlet Control – The entrance is submerged (HW ≤ 1.5D), the critical depth is less than uniform depth at design discharge ($d_c < d_u$), TW is greater than critical depth (TW > d_c) and TW is less than D (TW < D). This condition is a common occurrence where the channel is deep, narrow and well defined. The control is tailwater at the culvert outlet. The outlet velocity is the discharge divided by the area of flow in the culvert at tailwater depth.

$$\mathbf{HW = TW + V^2/(2g) + H_e + H_f - SL} \qquad \textbf{(5.2)}$$

Where: HW = Headwater depth (ft)

TW = tailwater at the outlet (ft)

V= velocity based on tailwater depth (ft/sec)

g = 32.2 ft/sec^2

H_e = entrance headloss (ft)

H_f = friction headloss (ft)

S = slope of culvert (ft/ft)

L = length of culvert (ft)

Tailwater Depth > Barrel Depth - Outlet Control – This condition will exist if the critical depth is less than uniform depth at the design discharge ($d_c < d_u$) and TW depth is greater than D (TW > D), or; the critical depth is greater than the uniform depth at the design discharge ($d_c > d_u$) and TW is greater than SL + D, [TW > (SL + D)]. The HW may not be greater than 1.5D, though often it is greater. If the critical depth of flow is determined to be greater than the barrel depth (only possible for rectangular culvert barrels), then this operation will govern. Outlet velocity is based on full flow at the outlet.

$$\mathbf{HW = H + TW – SL} \qquad \textbf{(5.3)}$$

Where: HW = Headwater depth (ft)

H = total head loss of discharge through culvert (ft)

TW = tailwater at the outlet (ft)

S = slope of culvert (ft/ft)

L = length of culvert (ft)

Tailwater < Barrel – Outlet Control – The entrance is submerged (HW > 1.5D) and the tailwater depth is less than D (TW < D). Normally, the engineer should arrive at this type of operation only after previous consideration of the operations depth covered when the critical depth, tailwater depth, or “slug” flow controls the flow in outlet control conditions. On occasion, it may be found that (HW ≥ 1.5D) for the three previously outlined conditions but (HW < 1.5D) for equation 5.4. If so, the higher HW should be used. Outlet velocity is based on critical depth if TW depth is less than critical depth. If TW depth is greater than critical depth, outlet velocity is based on TW depth.

$$HW = H + P - SL \quad (5.4)$$

Where: HW = Headwater depth (ft)

H = total head loss of discharge through culvert (ft)

P = empirical approximation of equivalent hydraulic grade line. $P = (d_c + D)/2$ if TW depth is less than critical depth at design discharge. If TW is greater than critical depth, then P = TW

S = slope of culvert (ft/ft)

L = length of culvert (ft)

5.4.3.2 Steep Slope

Tailwater Insignificant – Inlet Control – The entrance may be submerged or unsubmerged, the critical depth is greater than uniform depth at the design discharge ($d_c < d_u$), TW depth is less than SL (tailwater elevation is lower than the upstream flowline). Tailwater depth with respect to the diameter of the culvert is inconsequential as long as the above conditions are met. This condition is a common occurrence for culverts in rolling or hilly country. The control is critical depth at the entrance for HW values up to about 1.5D. Control is the entrance geometry for HW values over about 1.5D. HW is determined from empirical curves in the form of nomographs that are discussed later in this chapter. If TW is greater than D, outlet velocity is based on full flow at the outlet. If TW is less than D, outlet velocity is based on uniform depth for the culvert.

5.4.3.3 "Slug" Flow

Inlet or Outlet Control – For "slug" flow operation, the entrance may be submerged or unsubmerged, critical depth is greater than uniform depth at the design discharge ($d_c > d_u$), TW depth is greater than ($SL + d_c$) (TW elevation is above the critical depth at the entrance), and TW depth is less than SL + D (TW elevation is below the upstream crown). TW depth with respect to D alone is inconsequential as long as the above conditions are met. This condition is a common occurrence for culverts in rolling or hilly country. The control for this type of operation may be at the entrance or the outlet or control may transfer itself back and forth between the two (commonly called "slug" flow). For this reason, it is recommended that HW be determined for both entrance control and outlet control and that the higher of the two determinations be used. Entrance control HW is determined from the inlet control nomographs and outlet control HW is determined by equations 5.3 or 5.4 or the outlet control nomographs.

If TW depth is less than D, outlet velocity should be based on TW depth. It TW depth is greater than D, outlet velocity should be based on full flow at the outlet.

5.5 DESIGN PROCEDURES

5.5.1 Procedures

There are two procedures for designing culverts: (1) the manual use of inlet and outlet control nomographs; and (2) the use of a computer model.

The following will outline the design procedures for use of the nomographs. The use of computer models will follow the discussion on improved inlets. Other computer programs can be used if approved by the City or County Engineering Department.

5.5.2 Tailwater Elevations

In some cases culverts fail to perform as intended because of tailwater elevations high enough to create backwater. The problem is more severe in areas where gradients are very flat, and in some cases in areas with moderate slopes. Thus, as part of the design process, the normal depth of flow in the downstream channel at discharges equal to those being considered should be computed.

If the tailwater computation leads to water surface elevations below the invert of the culvert exit, there are obviously no problems; if elevations above the culvert invert are computed, the culvert capacity will be somewhat less than assumed. The tailwater computation can be simple, and on steep slopes requires little more that the determination of a cross section downstream where normal flow can be assumed, and a Manning's equation calculation. (See Chapter 3, Open Channel Hydraulics, for more information on open channel analysis). Conversely, with sensitive flood hazard sites, if the slopes are flat, or natural and man-made obstructions exist downstream, a water surface profile analysis reaching beyond these obstructions may be required.

5.5.3 Nomographs

The use of nomographs requires a trial and error solution. The solution is quite easy and provides reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs.

Following is an example of an inlet control and outlet control nomograph that can be used to design concrete pipe culverts. For culvert designs not covered by these nomographs, refer to the complete set of nomographs given in Appendix 5C at the end of this chapter.

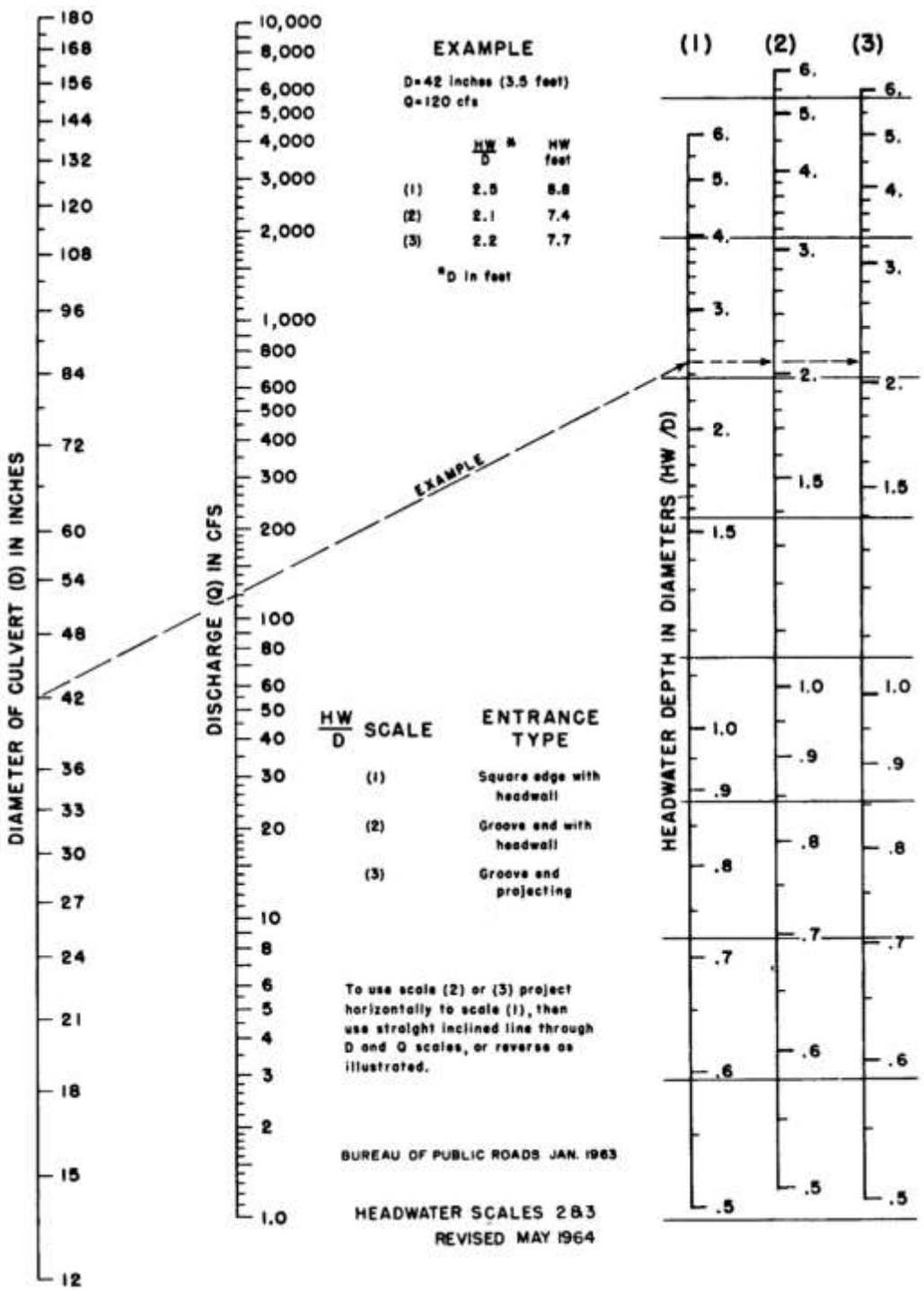


Figure 5-1 Headwater Water Depth for Concrete Pipe Culverts with Inlet Control – English Units

Source: www.deldot.gov/information/pubs_forms/manuals/road_design/pdf/supp_figures_chap_6.pdf, page 23

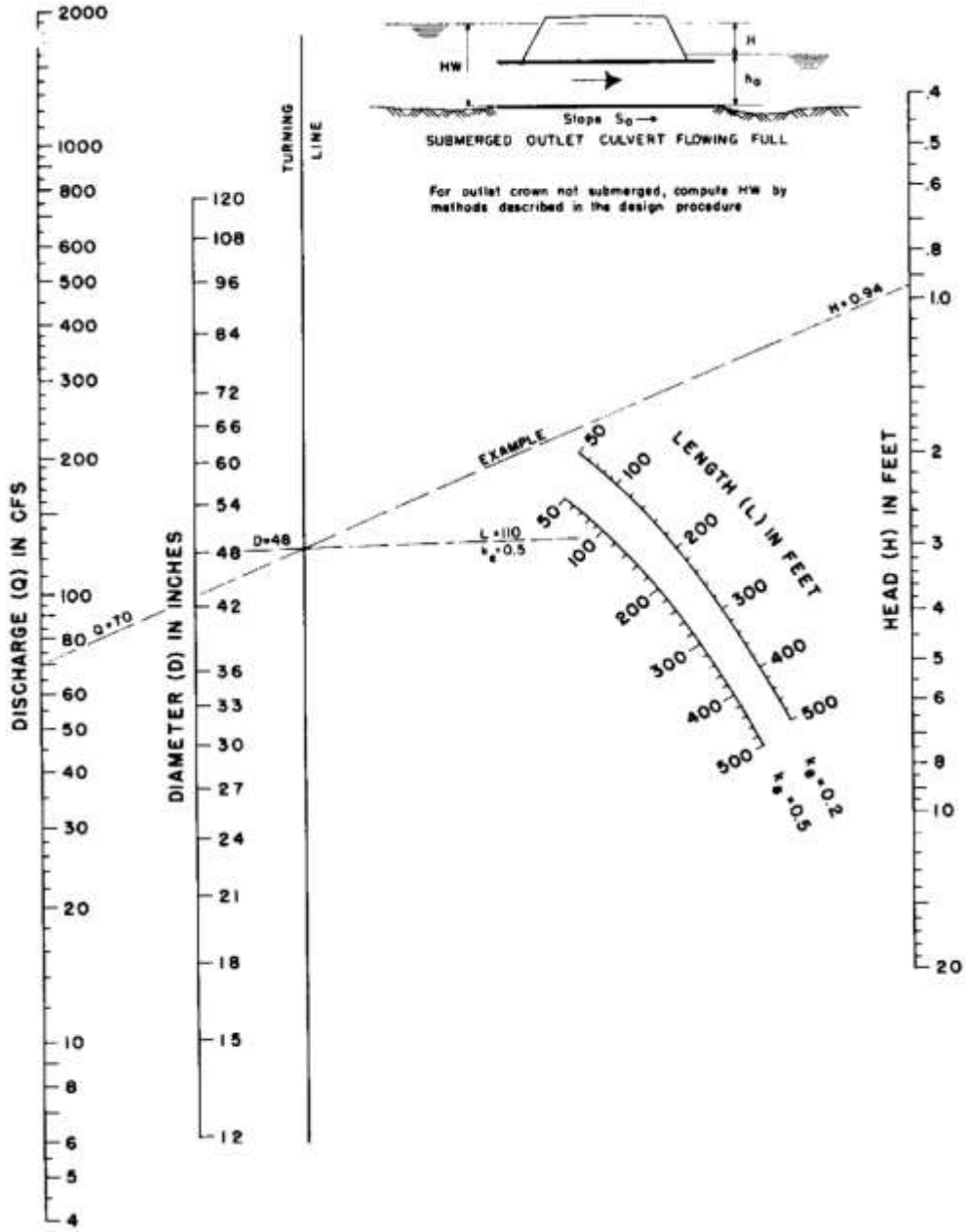


Figure 5- 2 Head for Concrete Pipe Culverts Flowing Full; n = 0.012
Source: www.deldot.gov/information/pubs_forms/manuals/road_design/pdf/supp_figures_chap_6.pdf, page 24

5.5.4 Steps in Design Procedure

The design procedure requires the use of both inlet and outlet nomographs

1) List design data:
   Q = discharge (cfs)
   L = culvert length (ft)
   S = culvert slope (ft/ft)
   HW = allowable headwater depth for the design storm (ft)
   V = velocity for trial diameter (ft/s)

K_e = inlet loss coefficient
TW = tailwater depth (ft)

2) Determine trial culvert diameter by assuming a trial velocity 3 - 5 ft/s and computing the culvert area, A = Q/V

3) Find the actual HW for the trial size culvert for both inlet and outlet control.

   - For *inlet control*, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.

   - Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.

   - For *outlet control* enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.

   - To compute HW, connect the length scale for the type entrance condition and culvert diameter scale with a straight line, pivot on the turning line and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation.

   $$\mathbf{HW = H + h_o - L} \qquad \mathbf{(5.5)}$$

4) Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control.

   If outlet control governs and the HW is unacceptable, select a larger size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

5) Calculate exit velocity and expected streambed scour to determine if an energy dissipater is needed.

5.5.5 Performance Curves

A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater versus discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the computer program discussed in the next section of this manual.

5.5.6 Roadway Overtopping

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, and overall performance curve can be developed. The performance curve depicts the sum of the flow through the culvert and across the roadway.

The overall performance curve can be determined as follows:

1) Select a range of flow rates and determine the corresponding headwater elevations for the culvert for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

2) Combined the inlet and outlet control performance curves to define a single performance curve for the culvert.

3) When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and equation 5.6 to calculate flow rates across the roadway.

$$\mathbf{Q = C_d L H W_r^{1.5}} \qquad \textbf{(5.6)}$$

Where: Q = overtopping flow rate (ft^2/s)

C_d = overtopping discharge coefficient

L = length of roadway (ft)

HW_r = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: For more information on calculating overtopping flow rates see the Hydraulic Design of Highway Culverts, HDS No. 5, Federal Highway Administration.

4) Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

5.6 CULVERT DESIGN EXAMPLE

5.6.1 Introduction

The following example problem illustrates the procedure to be used in designing culverts using the nomographs.

5.6.2 Problem

Size a culvert given the following design conditions which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this handbook.

5.6.3 Input Data

Discharge for 10-year flood = 70 cfs
Discharge for 100-year flood = 176 cfs
Allowable HW for 10-year discharge = 4.5 ft
Allowable HW for 100-year discharge = 7.0 ft
Length of culvert = 100 ft
Natural channel invert elevations

Inlet = 15.50 ft
Outlet = 15.35 ft
Culvert slope = 0.0015 ft/ft
Tailwater depth for 10-year discharge = 3.0 ft
Tailwater depth for 100-year discharge = 4.0 ft
Tailwater depth is the normal depth in downstream channel
Entrance type = Groove end with headwall

5.6.4 Computations

1) Assume a culvert velocity (3 - 5 ft/s is usually a good place to start).

   Required flow area = (70 cfs)/(5 ft/s) = 14 ft^2 (for the 10-year recurrence flood).

2) The corresponding culvert diameter is about 48 in.

   This can be calculated by using the formula for area of a circle:

   Area = $(3.14D^2)/4$ or D = $(\text{Area times } 4/3.14)^{0.5}$

   Therefore: D = $[(14\ ft^2 \times 4/3.14)^{0.5} \times 12\ in/ft]$

   D = 50.7 in

3) A grooved end culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 5-1), with a pipe diameter of 48 in and a discharge of 70 cfs; read a HW/D value of 0.93.

4) The depth of headwater (HW) is (0.93) x (4) = 3.72 ft which is less than the allowable headwater of 4.5 ft.

5) The culvert is checked for outlet control by using Figure 5-2.

   With an entrance loss coefficient K_e of 0.20, a culvert length of 100 ft, and a pipe diameter of 48 in, an H value of 0.77 ft is determined. The headwater for outlet control is computed by the equation:

   $HW = H + h_o - LS$

   For the tailwater depth lower than the top of culvert,

   h_o = TW or ½ (critical depth in culvert + D) whichever is greater.

   h_o = 3.0 ft or h_o = ½ (2.55 + 4.0) = 3.28 ft

   The headwater depth for outlet control is:

   $HW = H + h_o - LS$
   HW = 0.77 + 3.28 – (100) x (0.0015) = 3.90 ft

6) Since HW for outlet control (3.90 ft) is greater than the HW for inlet control (3.72 ft), outlet control governs the culvert design.

Thus, the maximum headwater expected for a 10-year recurrence flood is 3.90 ft, which is less than allowable headwater of 4.5 ft.

7) The performance of the culvert is checked for the 100-year discharge.

   The allowable headwater for a 100-year discharge is 7 ft; critical depth in the 48 in. diameter culvert for the 100-year discharge is 3.96 ft.

   For outlet control, an H value of 5.2 is read from the outlet control nomograph. The maximum headwater is:

   $HW = H + h_o – LS$
   $HW = 5.2 + 4.0 - (100) \times (0.0015) = 9.05$ ft

   This depth is greater than the allowable depth of 7 ft, thus a *larger size culvert must be selected*.

8) A 54 in diameter culvert is tried and found to have a maximum headwater depth of 3.74 ft for the 10-year discharge and of 6.97 ft for the 100-year discharge. These values are acceptable for the design conditions.

9) Estimate outlet exit velocity. Since this culvert is in outlet control and discharges into an open channel downstream, the culvert will be flowing full at the flow depth in the channel.

   Using the 100-year design peak discharge of 176 cfs and the area of a 54 inch or 4.5 ft diameter culvert the exit velocity will be:

   $Q = VA$

   Therefore: $V = 176 / (63.62/4) = 11.1$ ft/s

   With this high velocity, an energy dissipater may be needed downstream from the culvert for streambank protection. It will first be necessary to compute a scour hole depth and then decide if protection is needed. See Chapter 7, Energy Dissipation, for design procedures related to energy dissipaters.

10) The Design engineer should check minimum velocities for low frequency flow if the larger storm event (100-year) controls culvert design.

    Figure 5-3 on the next page provides a convenient form to organize culvert design calculations.

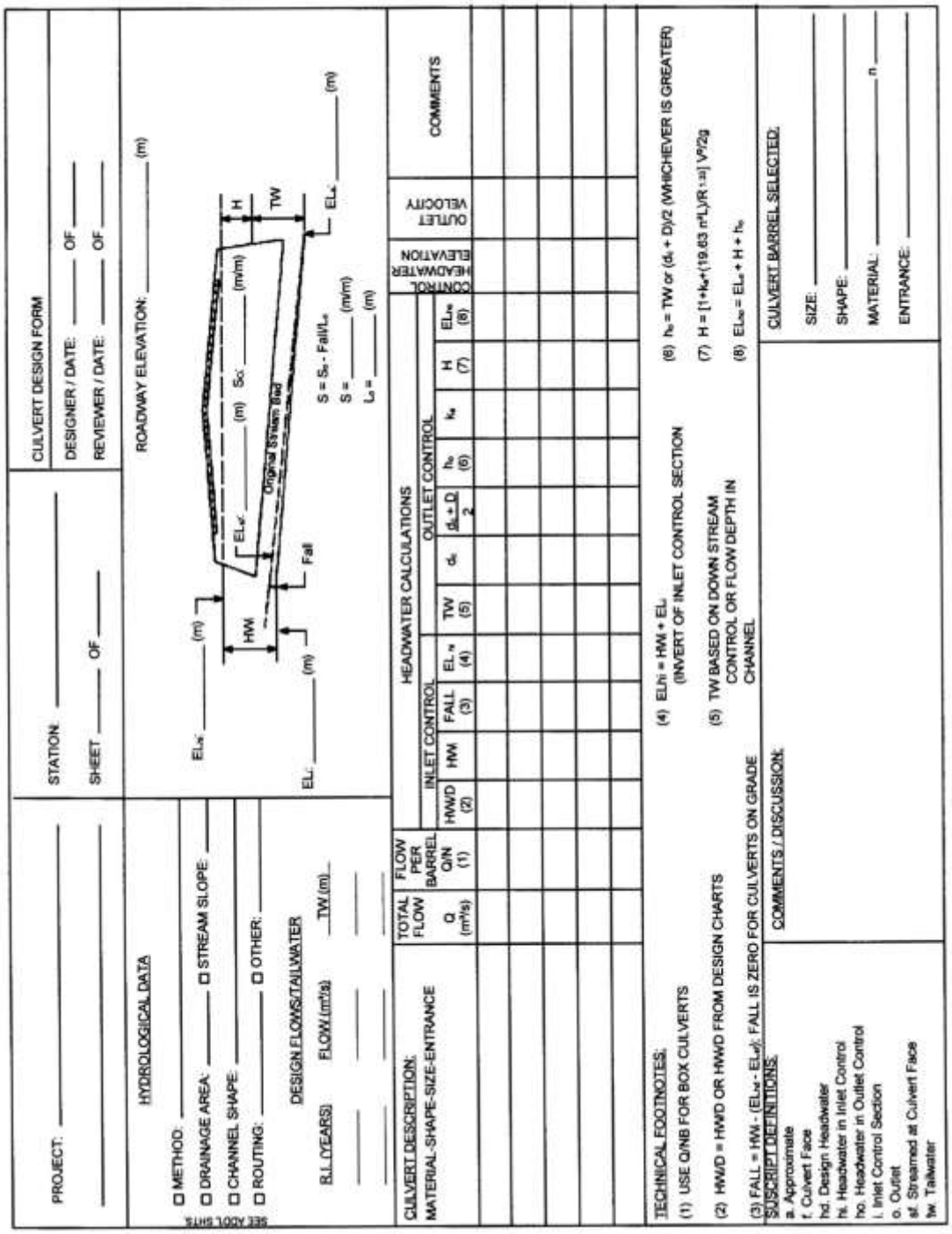


Figure 5-3 Culvert Design Form – Metric Version
Source: www.ct.gov/dot/lib/dot/documents/ddrainage/8.C.pdf, Appendix C

5.7 LONG SPAN CULVERTS

5.7.1 Introduction

Long span culverts are better defined on the basis of structural design aspects than on the basis of hydraulic considerations. According to the AASHTO Specifications for Highway Bridges, long span structural plate structure: 1) exceed certain defined maximum sizes for pipes, pipes-arches, and arches, or 2) may be special shapes of any size that involve a long radius of curvature in the crown or side plates. Special shapes include vertical and horizontal ellipses, underpasses, low and high profile arches, and inverted pear shapes. Generally, the spans of long span culverts range from 20 feet to 40 feet.

5.7.2 Structural Aspects

Long span culverts depend on interaction with the earth embankment for structural stability. Therefore, proper bedding and selection and compaction of backfill are of utmost importance. For multiple barrel structures, care must be taken to avoid unbalanced loads during backfilling.

Anchorage of the ends of long span culverts is required to prevent flotation or damage due to high velocities at the inlet. This is especially true for mitered inlets. Severe miters and skews are not recommended.

5.7.3 Hydraulic Considerations

Long span culverts generally are hydraulically short (low length to equivalent diameter ratio) and flow partly full at the design discharge. The same hydraulic principles apply to the design of long span culverts as to other culverts. However, due to their large size and variety of shapes, it is very possible that design nomographs are not available for the barrel shape of interest. For these cases, dimensionless inlet control design curves have been prepared. For the nomographs and design curves consult the publication, Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5.

For outlet control, backwater calculations are usually appropriate, since design headwaters exceeding the crowns of these conduits are rare. The bridge design techniques of HDS No. 1, Hydraulics of Bridge Waterways, are appropriate for the hydraulic design of most long span culverts.

5.8 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

An important step in the design process involves identifying whether special provisions are warranted to properly construct or maintain proposed facilities. Maintenance concerns of storm system design centers on adequate physical access for cleaning and repair.

Culverts must be kept free of obstructions. Sand and sediment deposits should be removed as soon as possible. During major storms, critical areas should be patrolled and the inlets kept free of debris. Inlet and outlet channels should be kept in alignment and vegetation should be controlled in order to prevent any significant restriction of flow. Preventative maintenance should be used to inspect for structural problems, replacement needs, and scheduling of needed repairs.

APPENDIX 5A
CRITICAL DEPTH CHARTS

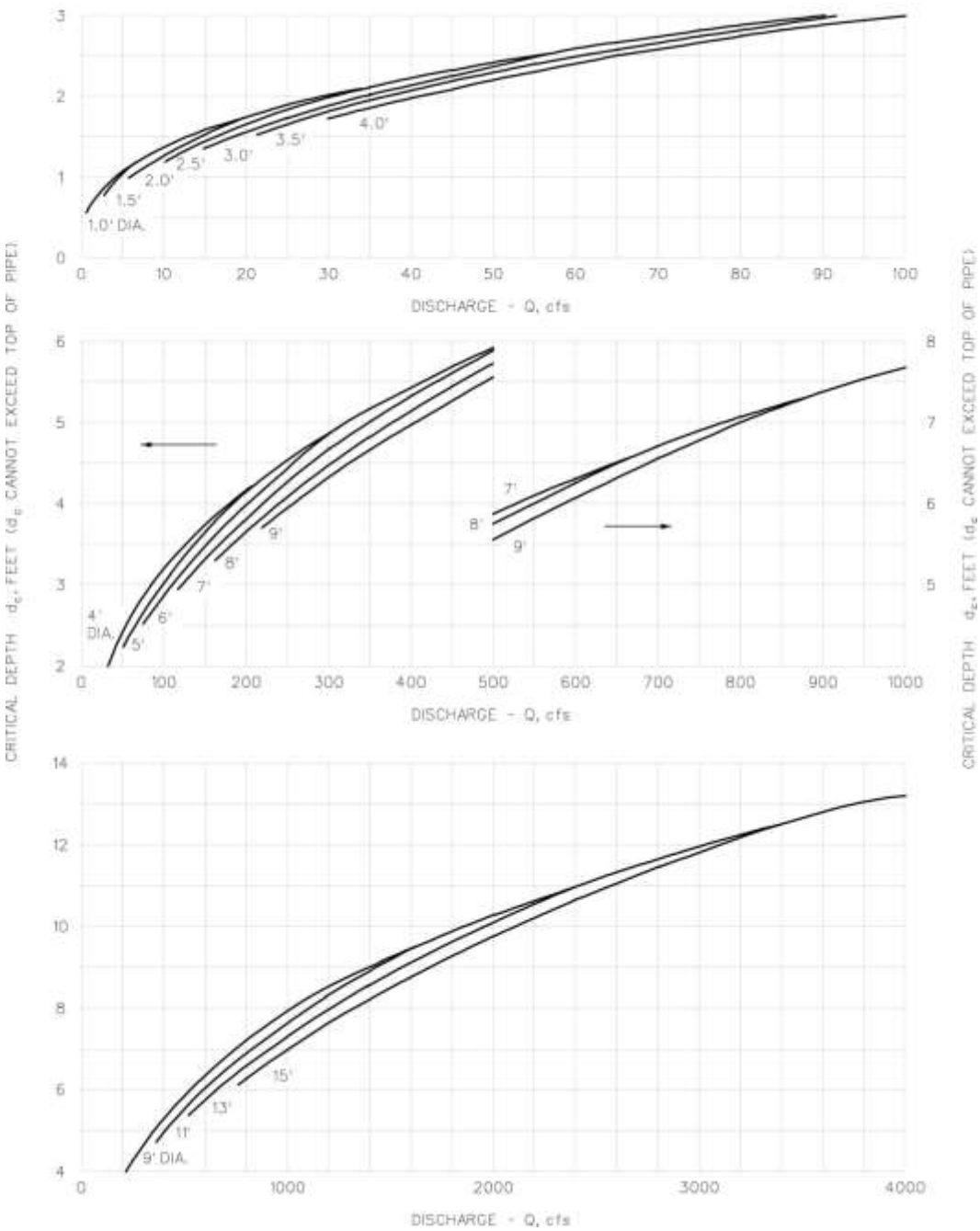


Figure 5A-1 Critical Depth for Circular Pipe
Source: http://epg.modot.org/files/9/96/750.2_Critical_Depth_for_Circular_Pipe.pdf

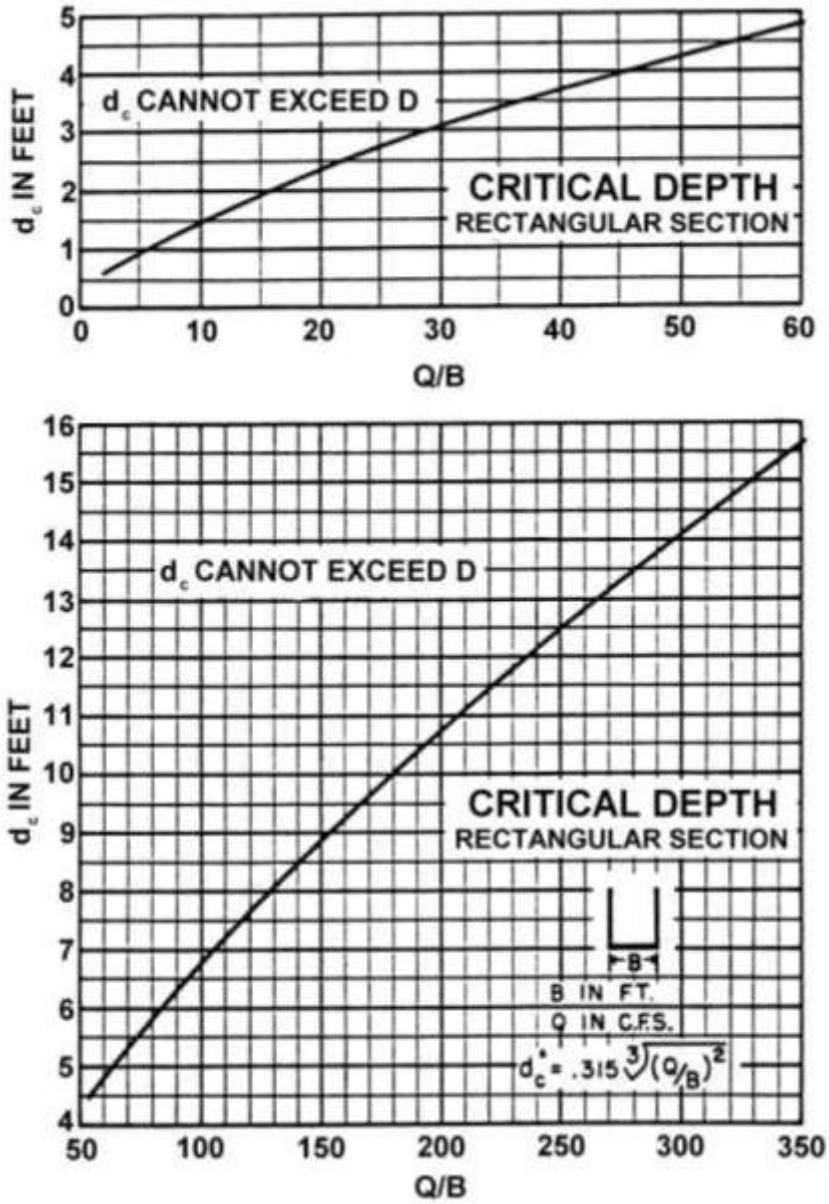


Figure 5A-2 Critical Depth for Rectangular Section
Source: http://Michigan.gov/documents/deq/lwm-smg-e_202862_7.pdf, page 2

APPENDIX 5B
DESIGN OF IMPROVED INLETS

5B.1 Introduction

A culvert operates in either inlet or outlet control. For a culvert operating under outlet control, the following characteristics influence the capacity of the culvert: headwater depth, tailwater depth, entrance configuration, and barrel characteristics.

The entrance configuration is defined by the barrel cross sectional area, shape, and edge condition, while the barrel characteristics are area, shape, slope, length and roughness.

5B.2 Outlet Control

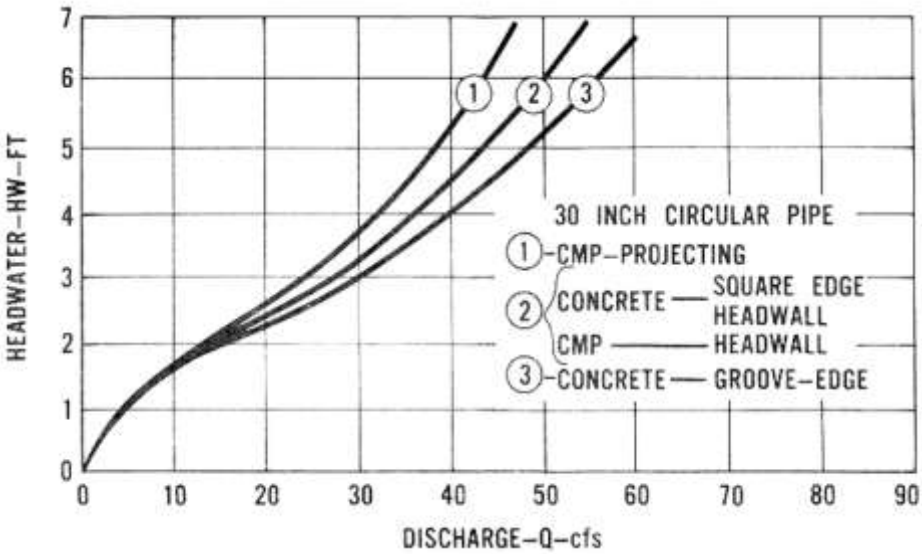
The flow condition for outlet control may be full or partly full for all or part of the culvert length. The design discharge usually results in full flow. Inlet improvements in these culverts reduce the entrance losses, which are only a small portion of the total headwater requirements. Therefore, only minor modifications of the inlet geometry (which result in little additional cost) are justified.

5B.3 Inlet Control

In inlet control, only entrance configuration and headwater depth determine the culvert's hydraulic capacity. Barrel characteristics and tailwater depth are of no consequence. These culverts usually lie on relatively steep slopes and flow only partly full. Entrance improvements can result in full or nearly full flow, thereby increasing culvert capacity significantly.

5B.4 Common Entrances

The figure below illustrates the performance of a 30-inch circular culvert in inlet control with three commonly used entrances: thin-edged projecting, square-edged, and groove-edged.

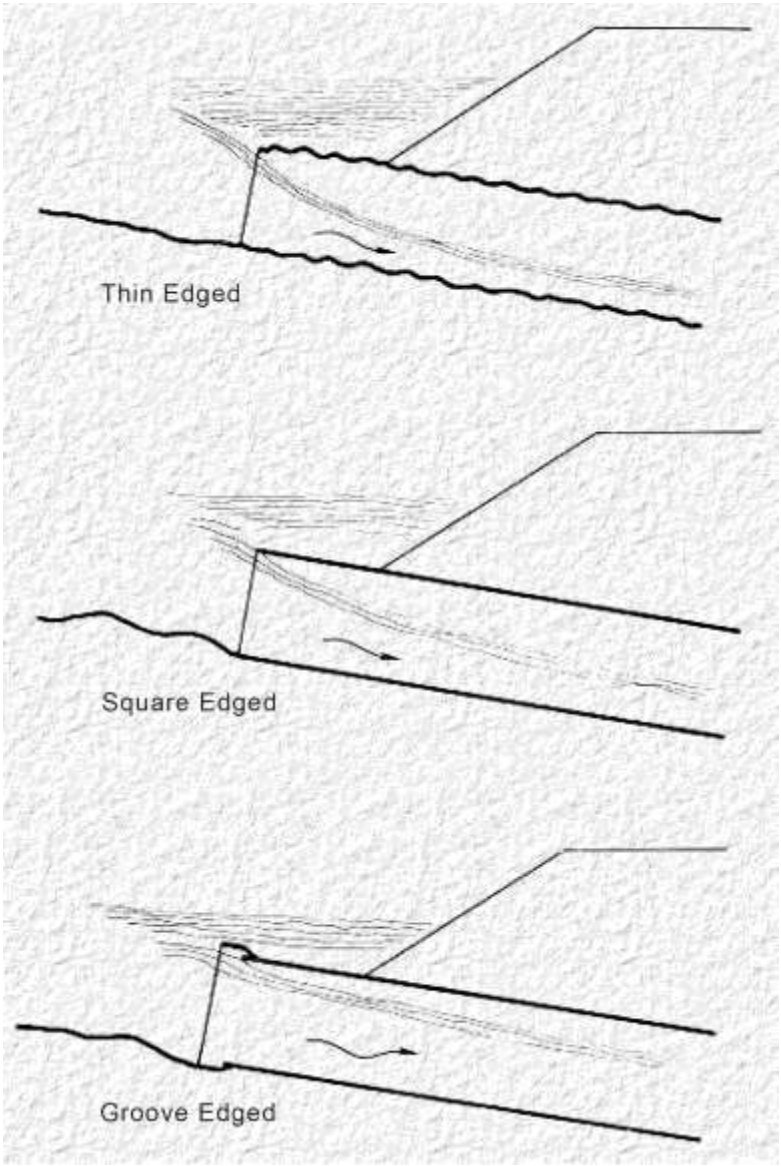


Source: www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec13.pdf

5B.5 Capacity Determinations

It is clear that inlet type and headwater depth determine the capacities of many culverts. For a given headwater, a groove-edged inlet has a greater capacity than a square-edged inlet, which in turn out performs a thin-edged projecting inlet.

The performance of each inlet type is related to the degree of flow contraction. A high degree of contraction requires more energy, or headwater, to convey a given discharge than a low degree of contraction. The figure below shows schematically the flow contractions of the three inlet types.



Source: www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec13.pdf

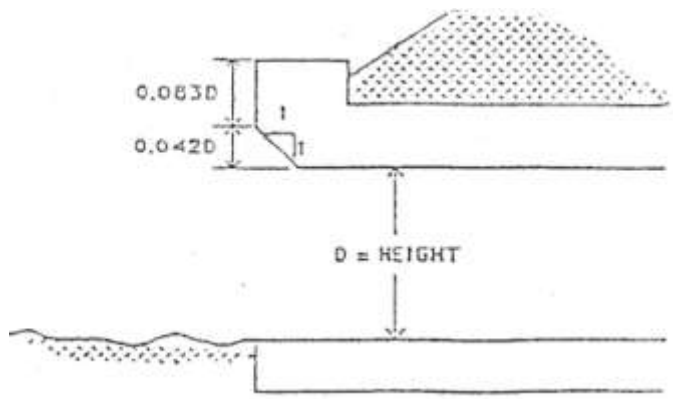
5B.6 Improved Inlets

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side tapered, and slope-tapered inlets. Detailed design criteria and example designs are outlined

in the FHWA, HDS-5 publication. The use of improved inlets must be approved by the City or County Engineering Department.

5B.7 Bevel-edged Inlet

The first degree of inlet improvement is a bevel-edged. The bevel is proportioned based on the culvert barrel or face dimension and operates by decreasing the flow contraction at the inlet. A bevel is similar to a chamfer except that a chamfer is smaller and is generally used to prevent damage to sharp concrete edges during constructions.



Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt4.pdf

Adding bevels to a conventional culvert design with a square-edged inlet increases culvert capacity by 5 to 20 percent. The higher increase results from comparing a bevel-edged inlet with a square-edged inlet at high headwaters. The lower increase is the result of comparing inlets with bevels, with structures having wingwalls of 30 to 45 degrees. Although the bevels referred to in the publication are plane surfaces, rounded edges which approximate the bevels are also acceptable. As a minimum, bevels should be used on all culverts which operate in inlet control, both conventional and improved inlet types. An exception to this is circular concrete culverts where the socket end performs much the same as a beveled edge.

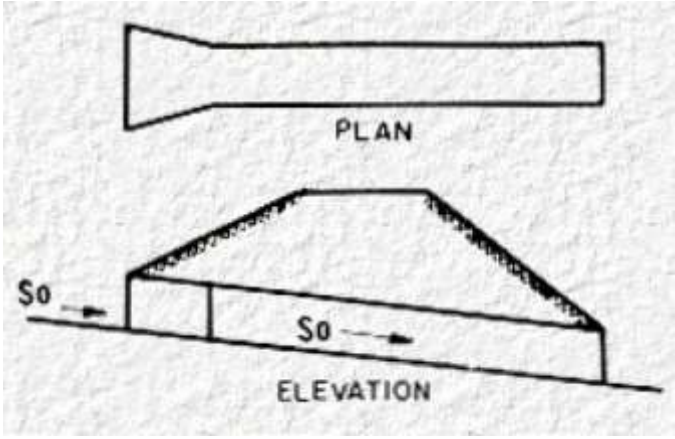
Culverts flowing in outlet control cannot be improved as much as those in inlet control, but the entrance loss coefficient, k_e, is reduced from 0.5 for a square edge to 0.2 for beveled edges.

It is recommended that bevels be used on all culvert entrances if little additional cost is involved.

5B.8 Side-Tapered Inlet

The second degree of improvement is a side-tapered inlet. This inlet has an enlarged face area with the transition to the culvert barrel accomplished by tapering sidewalls. The inlet face has the same height as the barrel, and its top and bottom are extensions of the top and bottom of the barrel. The intersection of the sidewall tapers and barrel is defined as the throat section. If a headwall and wingwall are going to be used at the culvert entrance, side-tapered inlets should add little if any to the overall cost while significantly increasing hydraulic efficiency. The side-tapered inlet provides an increase in flow capacity of 25 to 40 percent over that of a conventional culvert with a square edged inlet.

Whenever increased inlet efficiency is needed or when a headwall and wingwalls are planned to be used for a culvert installation, a side-tapered inlet should be considered.



Source: www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec13.pdf

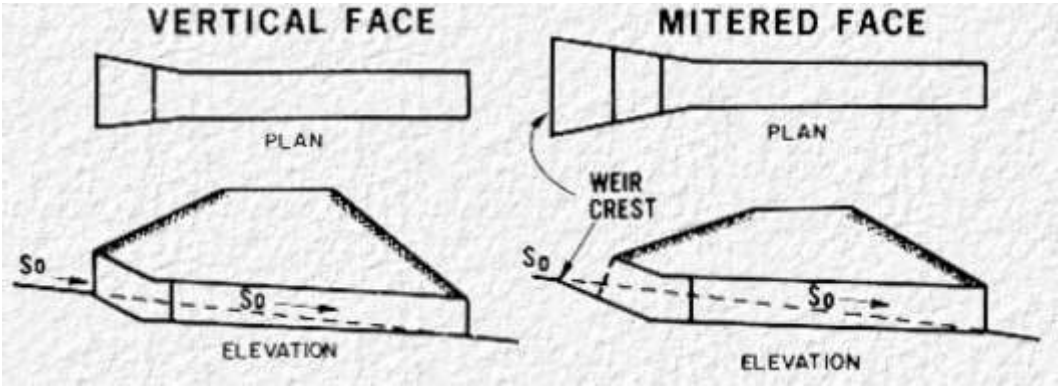
5B.9 Slope-Tapered Inlet

A slope-tapered inlet is the third degree of improvement. Its tapered advantage over the side-tapered inlet without a depression is that more head is available at the inlet. This is accomplished by incorporating a fall in the enclosed entrance section.

The slope-tapered inlet can have over 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends largely upon the amount of fall available. Since this may vary, a range of increased capacities is possible.

Side- and slope-tapered inlets should be used in culvert design when they can economically be used to increase the inlet efficiency over a conventional design.

For a complete discussion of tapered inlets including figures and illustrations, see FHWA, HDS-5, 2005.



Source: www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec13.pdf

5.B.10 Improved Inlet Performance

The two tables below compare the inlet control performance of the different inlet types. The first table shows the increase in discharge that is possible for a headwater depth of 8 feet. The bevel-edged inlet, side-tapered inlet and slope-tapered inlet show increases in discharge over the square-edged inlet of 16.7, 30.4 and 55.6 percent, respectively. It should be noted that the slope-tapered inlet incorporates only a minimum fall. Greater increases in capacity are often possible if a larger fall is used.

The second table depicts the reduction in headwater that is possible for a discharge of 500 cfs. The headwater varies from 12.5 ft for the square-edged inlet to 7.6 ft for the slope-tapered inlet. This is a 39.2 percent reduction in required headwater.

Table 5B-1

Comparison of Inlet Performance at Constant Headwater for 6 ft x 6 ft Concrete Box Culvert

Inlet Type	Headwater	Discharge	% Improvement
Square-edged	8.0 feet	336 cfs	0
Bevel-edged	8.0 feet	392 cfs	16.7
Side-tapered	8.0 feet	438 cfs	30.4
Slope-tapered*	8.0 feet	523 cfs	55.6

*Minimum fall in inlet = D/4 = 6/4 = 1.5 ft

Comparison of Inlet Performance at Constant Discharge for 6 ft x 6 ft Concrete Box Culvert

Inlet Type	Discharge	Headwater	% Improvement
Square-edged	500 cfs	12.5 feet	0
Bevel-edged	500 cfs	10.1 feet	19.2
Side-tapered	500 cfs	8.8 feet	29.6
Slope-tapered*	500 cfs	7.6 feet	39.2

*Minimum fall in inlet = D/4 = 6/4 = 1.5 ft

APPENDIX 5C
CONVENTIONAL NOMOGRAPHS

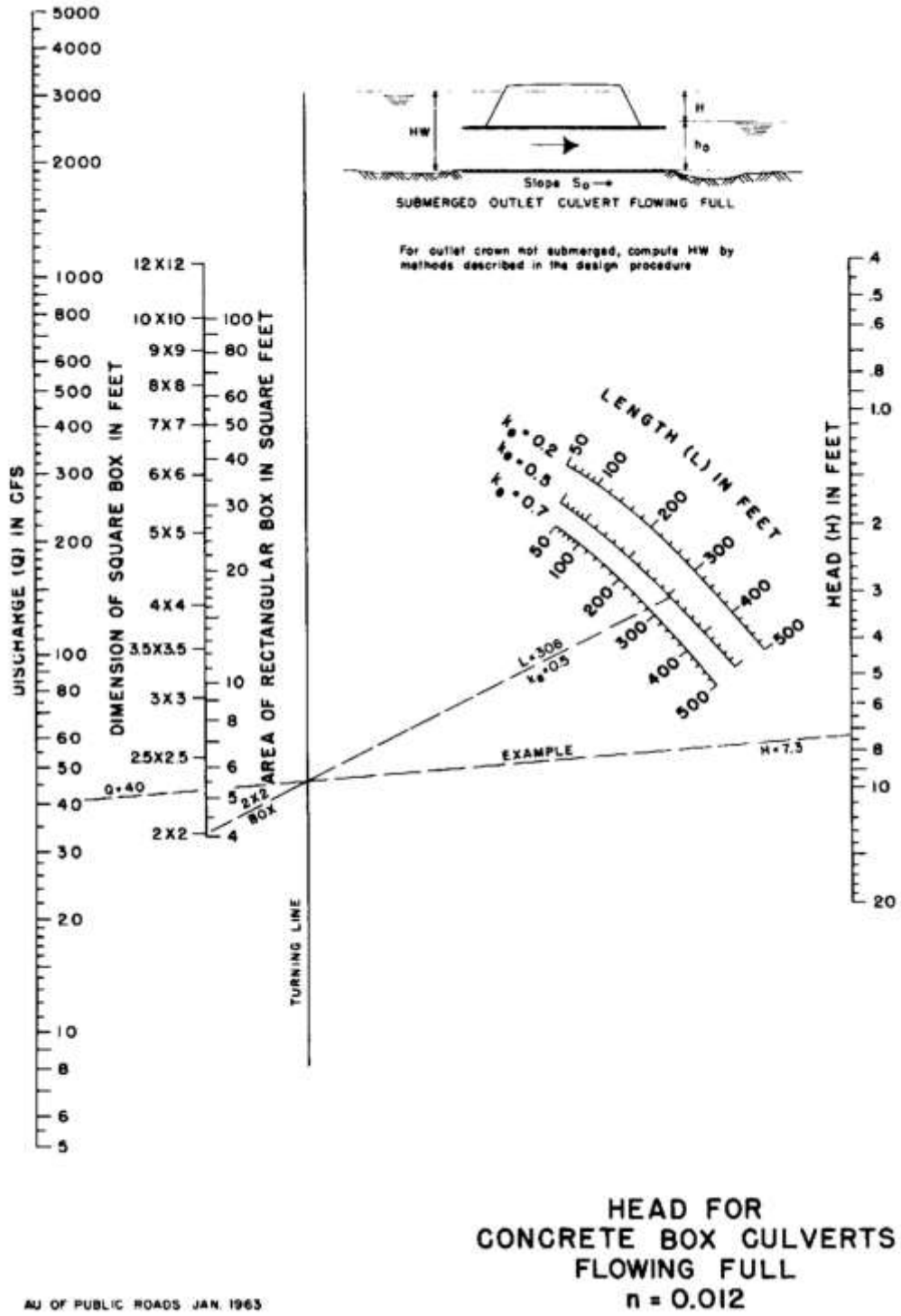


Figure 5C-1 Head for Concrete Box Culverts Flowing Full
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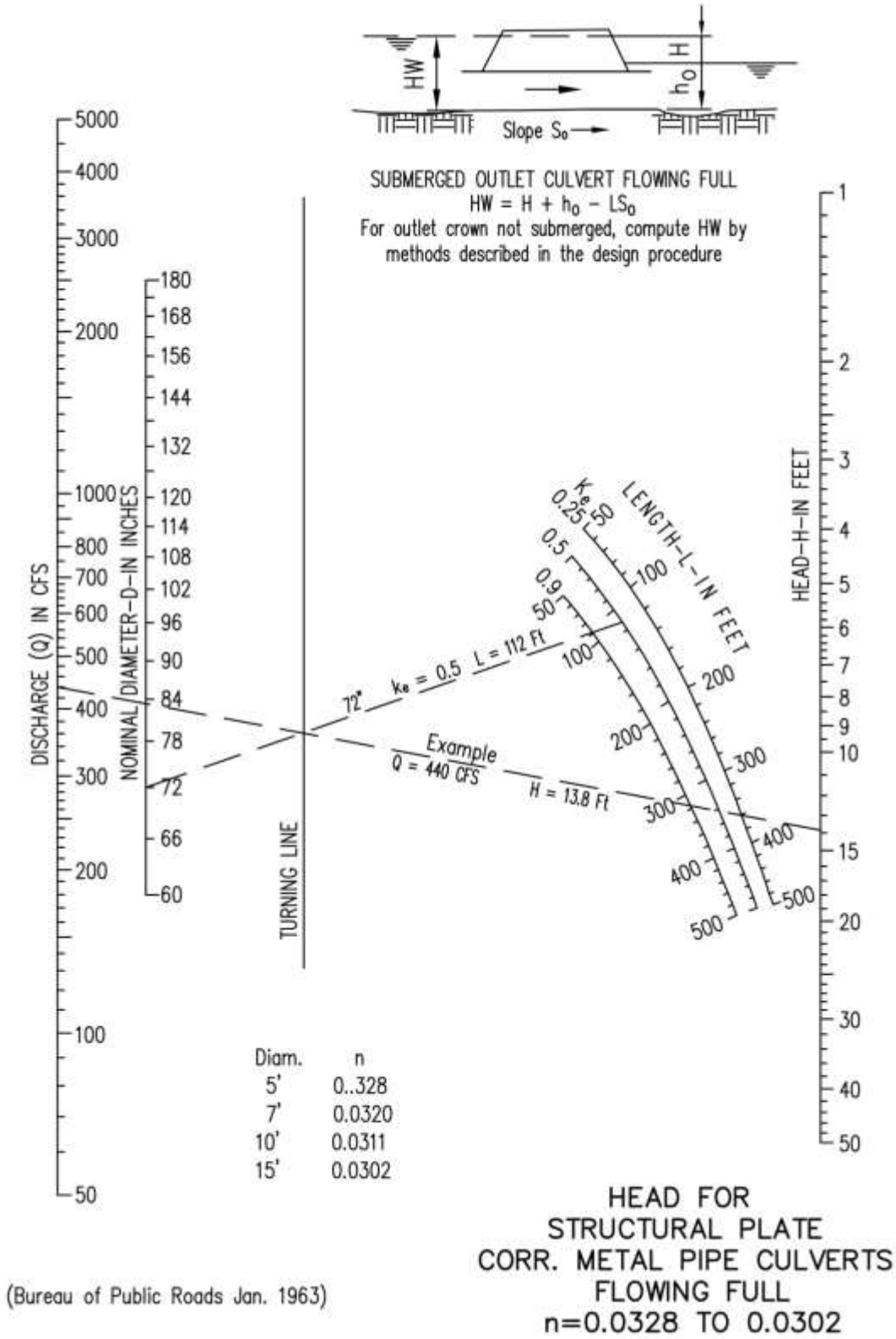

Figure 5C-2 Head for Structural Plate Corr. Metal Pipe Culverts Flowing Full
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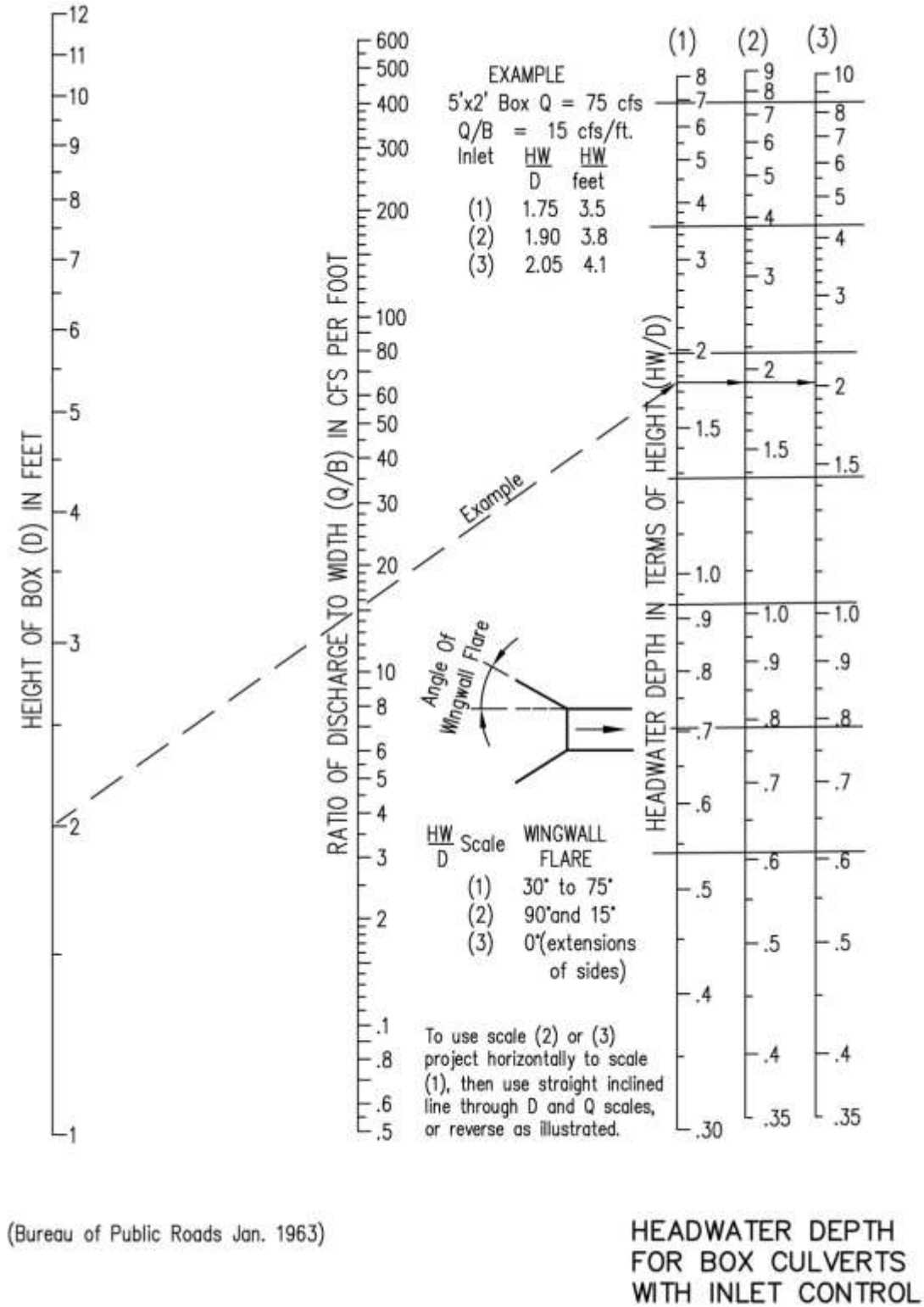

Figure 5C-3 Headwater Depth for Box Culverts with Inlet Control
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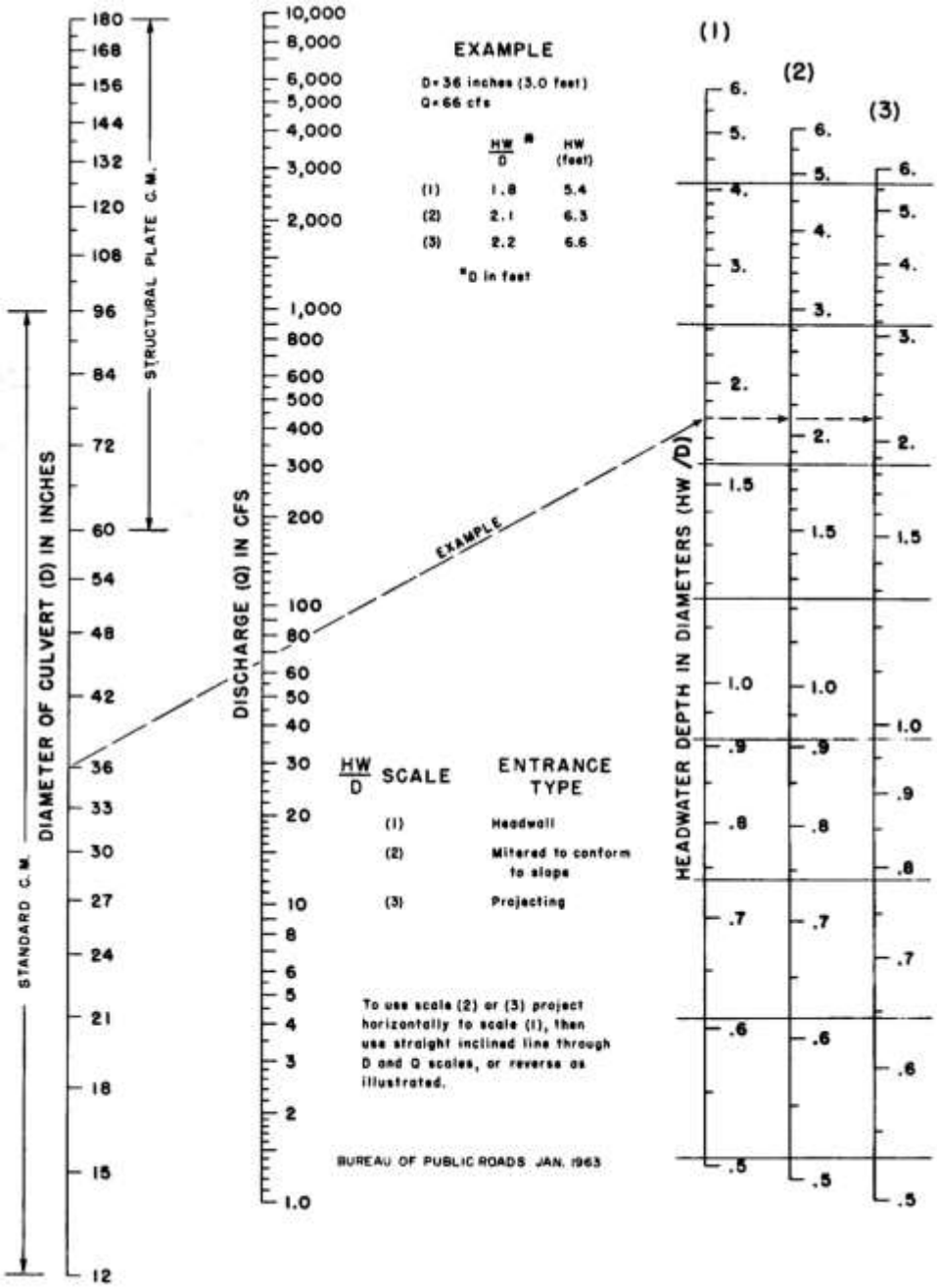

Figure 5C-4 Headwater Depth for CM Pipe Culverts with Inlet Control
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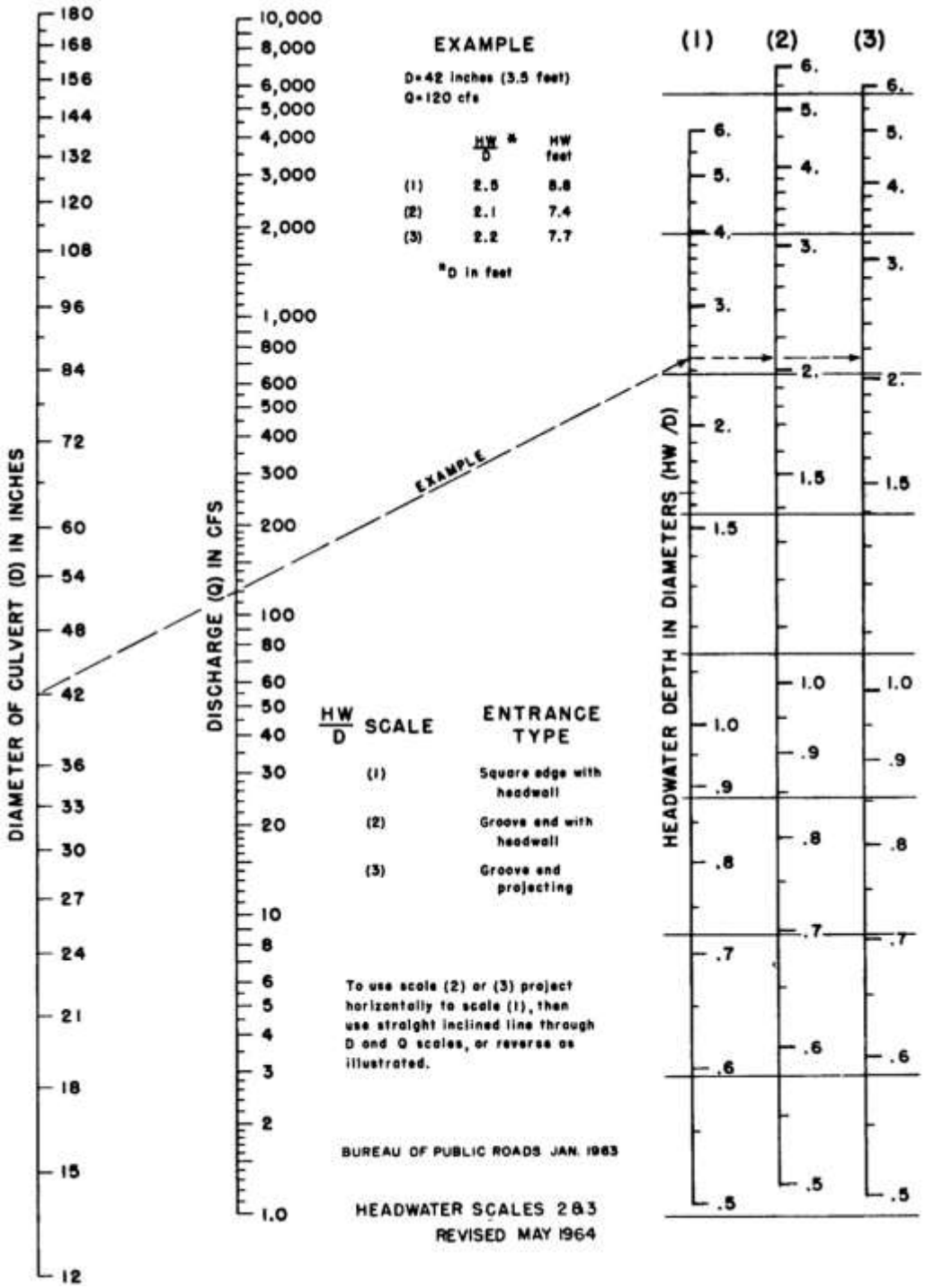

Figure 5C-5 Headwater Depth for Concrete Pipe Culverts with Inlet Control
Source: www.deldot.gov/information/pubs_forms/manuals/road_design/pdf/supp_figures_chap6.pdf

CHAPTER 6

STORAGE AND DETENTION

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6.1 OVERVIEW

6.1.1 Introduction

The traditional design of storm systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this type of design can cause major drainage and flooding problems downstream. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs either on-site or in some suitable off-site location. This chapter provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary sizing and final reservoir routing calculations.

6.1.2 Detention Facilities Used for Credits

If interested in obtaining a storm water fee credit, please refer to the Storm Water Fee Credit manual for application and requirements.

6.1.3 Location Considerations

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system which could decrease or increase flood peaks in different downstream locations. Thus it is important for the engineer to design storage facilities both as drainage structures controlling runoff from a defined area and as facilities that will interact with other drainage structures within the drainage basin.

6.1.4 Detention and Retention

Urban storm water storage facilities are often referred to as either detention or retention facilities. For the purposes of this chapter, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. Retention facilities are designed to contain a permanent pool of water. Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities these will be specified.

6.1.5 Computer Programs

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and very repetitive. To assist with these calculations there are many reservoir routing computer programs which can be used. There are also several simplified techniques that have been developed to design storage facilities which do not involve routing flows through the detention facility and rely on a water balance equation, equating storage to inflow minus outflow. Since these methods do not give accurate and reliable results, they should not be used and will not be presented in this chapter.

6.2 DESIGN CRITERIA

6.2.1 Introduction

If you do not comply with the Post Construction Control Ordinance then you must follow requirements in the zoning ordinance and the control requirements of 6.2.3 and 6.2.4 apply.

6.2.2 General Criteria

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. Possible dispersed or on-site storage may be developed in depressed areas in parking lots, road embankments and freeway interchanges, parks and other recreation areas, and small lakes, ponds and depressions within urban developments. The utility of any storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows that might be expected to pass through the storage facility should be included in the analysis (i.e., 100-year flood).

The design criteria for storage facilities should include:

- Release rate
- Storage volume
- Grading and depth requirements
- Outlet works
- Location

Note: The same hydrologic procedure shall be used to determine pre- and post- development hydrology.

6.2.3 Release Rate

Control structure release rates shall approximate pre-developed peak runoff rates for the 2-year and 10-year storms, with emergency overflow capable of handling to 50-year discharge. Design calculations are required to demonstrate that the facility will limit the 2- and 10-year developed discharge rates to pre-developed peak discharge rates. If so, intermediate storm return periods can be assumed to be adequately controlled. Multi-stage control structures may be required to control both the 2- and 10-year storms.

6.2.4 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates for the 2-year and 10-year storms. Routing calculations must be used to demonstrate that the storage volume is adequate. For detention basins, all detention basins, all detention volume shall be drained within 72 hours.

6.2.5 Storage within Parking Areas

Storage within code-required parking areas are allowed a maximum depth of 6 inches; additional parking areas 10 inches; and 15 inches is allowed in truck storage and loading areas.

6.2.6 Grading and Depth of Earthen Storage Facility

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated and riprap protected embankments shall have side slopes no steeper than 2:1 (H:V) and shall meet requirements of the Dam Safety Act when necessary.

Areas above the normal high water elevations of storage facilities should be sloped at a minimum of 5 percent toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2 percent bottom slope is required. A concrete paved low flow or pilot channel (minimum slope of ½ %) constructed across the facility bottom from the inlet to the outlet shall be considered for conveyance of low flows to prevent standing water conditions.

A minimum freeboard of 6 inches above the 50-year design storm high water elevation shall be provided all impoundments. Storage facilities under the jurisdiction of the North Carolina Department of Environment and Natural Resources (NDCENR), Dam Safety Program of Land Quality are in addition subject to the requirements of the Safe Dams Act (see Section 6.2.9).

6.2.7 Outlet Works

Outlet works selected for storage facilities typically include a control structure and an emergency outlet and must be able to accomplish the design functions of the facility. Outlet works can take the form of any combination of drop inlets, pipes, weirs, and orifices. Curb openings may be used for parking lot storage. The control structure is intended to convey the design storm without allowing flow to enter an emergency outlet. Selecting a magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum storm to be used to size the emergency outlet is the 50-year storm. The sizing of a particular outlet works shall be based on results of hydrologic routing calculations. Minimum barrels through embankments are 12 inch pipes with corresponding orifice plates. Any orifice smaller than 4 inches in diameter must be protected to prevent blockage. A 2 foot by 2 foot concrete pad must be placed in front of any orifice plate at the invert of the outlet. If the spillway is in fill material then the spillway must be lined.

6.2.8 Off-site Storm Water Detention Facilities

When off-site storm water detention facilities (or for storm water release through recorded easements to regulated floodway in lieu of detention) the following requirement must be met:

- All pipes/channels leading from the subject site to the off-site storm water detention facility (or regulated floodway) must be sized to carry the 10-year storm water runoff.
- A “Permanent Detention Easement” leading from the subject site to the off-site detention facility (or regulated floodway) must be shown on a map which has been recorded with the Mecklenburg County Register of Deeds Office. This easement should be centered on the pipe or channel and must be at least 15 feet wide for pipes and 20 feet wide for channels (refer to City Std. #20.30 for easement widths). A metes and bounds description is not required for this portion of the easement.
- A “Permanent Detention Easement” which encompasses the detention facility must be shown on the recorded map. This easement must be described by metes and bounds.
- The recorded map must have a note which clearly states who is responsible for maintenance of the detention facility, pipes, and/or channels located within the Permanent Detention Easements (these easements will not be maintained by the City).

- For off-site detention facilities the recorded map must have a note stating, "The purpose of the Permanent Detention Easement is to provide storm water detention for Lot(s) _____. The pipes and/or channels located within the Permanent Detention easement and leading to the detention facility carry unrestricted storm water flow from the developed upstream Lot(s) ___.
- For storm water released to regulated floodway through an easement, the recorded map must have a note stating "The purpose of the Permanent Detention Easement is to allow storm water release directly to regulated floodway in lieu of on-site storm water detention. The pipes and/or channels located within the Permanent Detention Easement and leading to the regulated floodway carry unrestricted storm water flow from the developed upstream Lot(s) ___.

6.2.9 Dam Safety Law

Under the Dam Safety Law regulations, a dam is a structure and appurtenant works erected to impound or divert water that is 25 feet or greater in height and has a maximum storage volume of 50 acre-feet or more. A number of exemptions are allowed from the Dam Safety Law and any questions concerning a specific design or application should be addressed to the NCDENR, Dam Safety Program of Land Quality (919-707-9220).

6.3 GENERAL PROCEDURE

6.3.1 Data Needs

The following data will be needed to complete storage design and routing calculations.

- Inflow hydrograph for all design storms for fully developed and pre-developed conditions.
- Stage-storage curve for proposed storage facility (see Figure 6-1 for an example).
- Stage-discharge curve for all outlet control structures (see Figure 6-2 for an example).

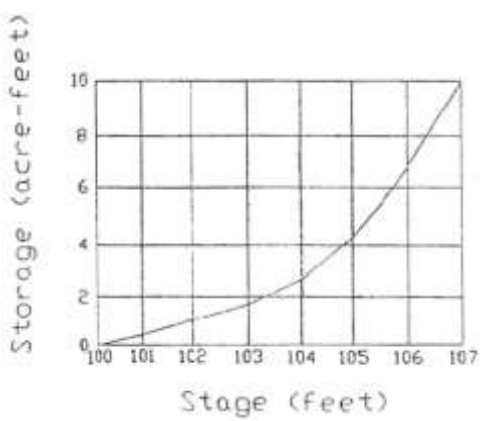

Figure 6-1 Example Stage-Storage Curve

Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt6.pdf

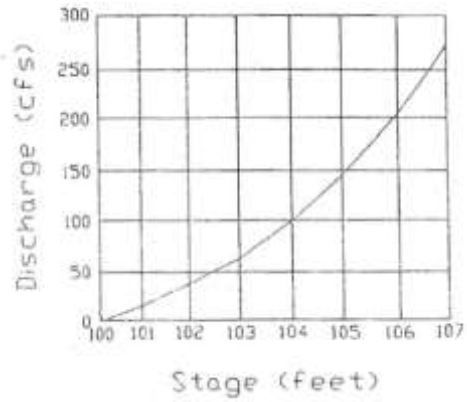

Figure 6-2 Example Stage-Discharge Curve

Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt6.pdf

Using these data a trial and error design procedure is used to route the inflow hydrograph through the storage facility until the desired outflow hydrograph is achieved.

6.3.2 Procedure

A general procedure for using the above data in the design of storage facilities is presented below:

1. Compute inflow hydrograph for the 2-, 10-, and 50-year design storms using the procedures outlined in the Hydrology Chapter. Both pre- and post-development hydrographs are required for the 2- and 10-year design storms. Only the post-development hydrograph is required for the 50-year design storm.

2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Appendix 6A). If storage requirements are satisfied for the 2- and 10-year design storms, intermediate storms are assumed to be controlled.

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used.

4. Size the outlet structure. Estimate the peak stage for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

5. Perform routing calculations using inflow hydrographs from Step 1 to check preliminary design using the storage routing equations. If the routed post-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to step 3.

6. Consider emergency overflow from the 50-year (or larger) design storm and establish freeboard requirements, as referenced within this manual and the Dame Safety Act, whichever is more restrictive.

   This procedure can involve a significant number of reservoir routing calculations to obtain the desired results. Computer based methods, such as HEC-1/HEC-HMS, are widely available to perform these iterations quickly. Other computer programs can

provide similar results. For guidelines for using other party software, please refer to Section 2.1.2 – Hydrologic Method.

6.4 OUTLET HYDRAULICS

6.4.1 Outlets

Sharp-crested weir flow equation for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data.

6.4.2 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated below. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = CLH^{1.5}} \qquad \textbf{(6.1)}$$

Where: Q = discharge (cfs)

H = head above weir crest excluding velocity head (ft)

L= horizontal weir length (ft)

C= sharp crested weir coefficient, use 3.3

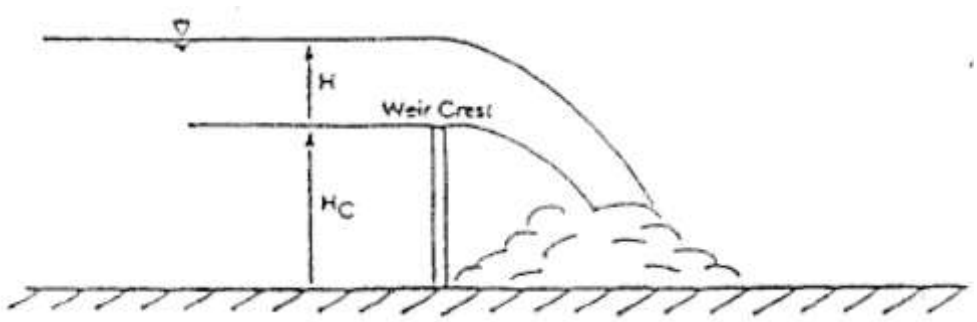

Figure 6-3 Sharp-Crested Weir

Source: www.georgiastormwater.com/vol2/2-3.pdf

6.4.3 Broad-Crested Weirs

The equation generally used for the broad-crested weir (see sketch below) is (Brater and King, 1976):

$$\mathbf{Q = CLH^{1.5}} \qquad \textbf{(6.2)}$$

Where: Q = discharge (cfs)

C = broad-crested weir coefficient, use 3

L = horizontal weir length (ft)

H = head above weir crest (ft)

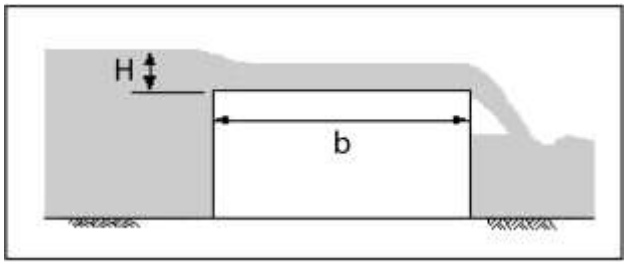

Figure 6-4 Broad-Crested Weir
Source: www.georgiastormwater.com/vol2/2-3.pdf

6.4.4 V-Notch Weirs

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

$$\mathbf{Q = 2.5 \tan(\theta/2) H_v^{2.5}} \qquad \textbf{(6.3)}$$

Where: Q = discharge (cfs)

θ = angle of v-notch (degrees)

H_v = head on vortex of notch (ft)

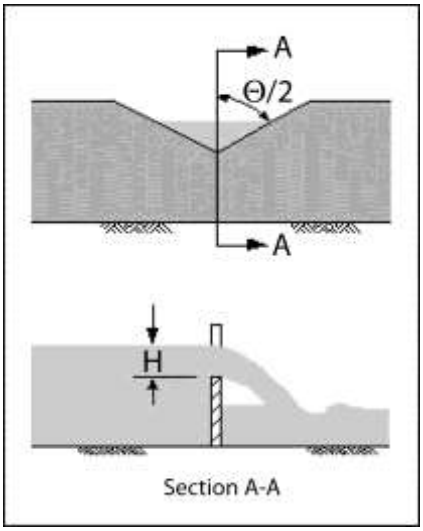

Figure 6-5 V-Notch Weir
Source: www.georgiastormwater.com/vol2/2-3.pdf

6.4.5 Orifices

The general equation for discharge through a submerged orifice is:

$$Q = CA(2gH)^{0.5} \qquad (6.4)$$

Where: Q = discharge (cfs)

A = cross-section area of smallest section (ft^2)

g = acceleration due to gravity, 32.2 ft/s^2

H = head on pipe, or the vertical distance from the center of the orifice to the upstream free-water surface

C = discharge coefficient, use .60

6.5 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage facilities. To assure acceptable performance and function, storage facilities that require extensive maintenance are discouraged. The following maintenance problems are typical of urban detention facilities and facilities shall be designed to minimize such problems.

- weed growth
- grass and vegetation maintenance
- sedimentation control
- bank deterioration
- standing water or soggy surfaces
- mosquito control
- blockage of outlet structures
- litter accumulation
- maintenance of fences and perimeter plantings

Proper design should focus on the elimination or reduction of maintenance requirements by addressing the potential for problems to develop.

- Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers.
- Sedimentation may be controlled by constructing traps to contain sediment for easy removal or low-flow channels to reduce erosion and sediment transport.
- Bank deterioration can be controlled with protective lining or by limiting bank slopes.
- Standing water or soggy surfaces must be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or by constructing underdrain facilities to lower water tables.
- In general, when these problems are addressed, mosquito control will not be a major problem.
- Outlet structures should be selected to minimize the possibility of blockage (i.e., very small pipes tend to become blocked quite easily and should be avoided). Outlets shall be no less than 4 inches in diameter.

- Finally, one way to deal with the maintenance associated with litter and damage to fences and perimeter plantings is to locate the facility for easy access where this maintenance can be conducted on a regular basis.

6.6 UNDERGROUND STORAGE

If surface ponding is not feasible, underground storage may be necessary. This can be accomplished by installation of a storage facility under a parking or grassed area. This area shall be required to have appropriate access to allow for its maintenance. In structures over 3'-6" in depth, steps shall be provided in accordance with Charlotte Land Development Standards Manual. The storage facility shall also be required to have all joints properly sealed to prevent undermining of the structures.

When storage is used within a pipe system all pipes shall have sealed joints. The use of o-rings on reinforced concrete or neoprene gaskets for coupling on corrugated metal pipe is necessary. Metal pipe will not be required to have paved inverts. However, they should be designed to prevent corrosion with the use of aluminum pipe and corrosion resistant coatings. The minimum slope on any underground storage structure is 0.5%.

Appendix 6A
CALCULATIONS

6A.1 Routing Calculations

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

1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility. For example of stage-storage and state-discharge curves see Figures 6-1 and 6-2.

2. Select a routing time period, d_t, to provide at least three points on the rising limb of the inflow hydrograph.

3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S \pm (0/2)d_t$ versus stage. An example tabulation of storage characteristics curve data is shown below.

(1) Stage (Hs) (ft)	(2) Storage[1] (S) (ac-ft)	(3) Discharge[2] (O) (cfs)	(4) (ac-ft/hr)	(5) $S - (O/2)\ d_t$ (ac-ft)	(6) $S + (O/2)\ d_t$ (ac/ft)
100	0.05	0	0	0.05	0.05
101	0.30	15	1.24	0.20	0.40
102	0.80	35	2.89	0.56	1.04
103	1.60	63	5.21	1.17	2.03
104	2.80	95	7.85	2.15	3.45
105	4.40	143	11.82	3.41	5.39
106	6.60	200	16.53	5.22	7.98
107	10.00	275	22.73	8.11	11.89

[1] Obtained from the Stage-Storage Curve above.
[2] Obtained from the Stage-Discharge Curve above.
Note: d_t = 10 minutes = 0.167 hours and 1 cfs = 0.0826 ac-ft/hr

Note: If detention facility contains a permanent pool of water, this can be accounted for by considering the water surface as the zero stage.

4. For a given time interval, I_1 and I_2 from the post development hydrograph are known. Given the depth of storage or stage H_{s1}, at the beginning of that time interval, $S_1 - (O_1/2)\ d_t$ can be determined from the appropriate storage characteristics curve (example given below).

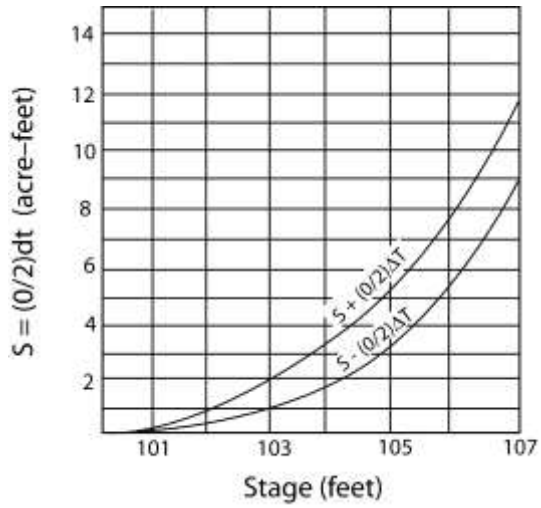

Figure 6A-1 Storage Characteristics Curve

Source: www.lincoln.ne.gov/city/pworks/watrshed/require/drainage/pdf/chapt6.pdf

5. Determine the value of S2 + (O2/2) d_t from the following equation:

$$\mathbf{S_2 + (O_2/2)\, d_t = [S_1 - (O_1/2)\, d_t] + [\{I_1 + I_2\}\, d_t]} \qquad \mathbf{(6.7)}$$

Where : S_2 = storage volume at time 2 (ft^3)

O_2 = outflow rate at time 2 (cfs)

d_t = routing time period (sec)

S_1 = storage volume at time 1 (ft^3)

O_1 = outflow rate at time 1 (cfs)

I_1 = inflow rate at time 1 (cfs)

I_2 = inflow rate at time 2 (cfs)

Other consistent units are equally appropriate.

6. Enter the storage characteristics curve at the calculated value of $S_2 + (O_2/2)\, d_t$ determined in Step 5 and read off a new depth of water, H_{s2}.

7. Determine the value of O_2, which corresponds to a storage of H_{s2} determined in Step 6, using the stage-discharge curve.

8. Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1, and H_{s1} equal to the previous I_2, O_2, S_2, and H_{s2} and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

   When the proposed outlet is large and proposed storage is small, this routing technique may initially be numerically unstable. The actual effect is that the outflow hydrograph and inflow hydrograph virtually match each other during the earliest portion of the design

storm. Mathematically, negative storage results on the routing calculation table, and in most spreadsheet applications this will effectively stop the calculation process. The routing may be re-initialized as follows:

1. Set outflow equal to inflow.

2. Set stage based on outflow by referring to the already developed stage-discharge function.

3. Set storage based on stage by referring to the already developed stage-storage function.

4. Restart the routing and repeat steps 1-3 until the system behaves.

(Adapted from H. R. Malcom, P.E., *Elements of Urban Stormwater Design*, NCSU Press, 1989)

APPENDIX 6B
EXAMPLE PROBLEM

6B.1 Example

This example demonstrates the application of the methodology presented in this chapter in routing an inflow hydrograph through a storage facility.

6.B.2 Inflow Hydrograph

Following is the inflow hydrograph for this example (methods described in the Hydrology Chapter would be used to develop this hydrograph).

Time (min)	Inflow (cfs)	Time (min)	Inflow (cfs)
0	0	90	91
10	2	100	61
20	27	110	37
30	130	120	20
40	300	130	11
50	360	140	5
60	289	150	1
70	194	160	0
80	133		

6B.3 Stage-Storage Curve

From the physical characteristics of the storage facility to be used, a stage-storage is developed. Use Figure 6-1 for this example.

6B.4 Stage-Discharge Curve

From the characteristics of the outlet device, a stage-discharge curve is developed. Use Figure 6-2 for this example.

6B.5 Storage Characteristics Curve

A routing time period (d_t) of 10 minutes is selected for this example. Using this routing time period, the stage-storage curve and the stage-discharge curve, storage characteristics curves can be developed. Use Figure A-1 for this example.

6B.6 Routing Calculations

Following are routing calculations for this example. Table B-1 on the next pages gives the results of these calculations.

1. Given that $S_1 - (O_1/2)d_t = 0.05$ acre-foot for $H_{s1} = 0$ foot, find $S_2 + (O_2/2)\ d_t$ by adding 0.05 + 0.01 (column 5 value plus column 3 value) and tabulate 0.06 acre-foot in column 6 of Table B-1.

2. Enter the $S + (O/2)\ d_t$ storage characteristics curve and read the stage at the value of 0.06 acre-foot. This value is found to be 100.10 feet and is tabulated as stage H_{s2} in column 7 of Table B-1.

3. Using the stage of 100.10 feet found in step 2, enter the stage-discharge curve and find the discharge corresponding to that stage. In this case, outflow is approximately 1 cfs and is tabulated in column 8 of Table B-1.

4. Assign the value of H_{s2} to H_{s1}, find a new value of $S_1 - (O_1/2)\ d_t$ and repeat the calculations for steps 1, 2, and 3. Continue repeating these calculations until the entire inflow hydrograph has been routed through the storage facility.

5. The routing calculations give a peak outflow of 220 cfs. The inflow hydrograph has a peak rate of 360 cfs, so a reduction of approximately 40 percent is calculated.

6. If the 40 percent reduction is acceptable then the calculations are complete. If more or less reduction is needed, then new values for stage-storage or stage-discharge must be assigned and the calculations repeated. To comply with local regulations by keeping the peak developed outflow the same as the peak undeveloped outflow for the 2- through 10-year floods, many iterations of the routing calculations may be needed.

(1) Time	(2) Inflow	(3) $[(I_1+I_2)/2]dt$	(4) H_1	(5) $S_1-(O/2)dt$	(6) $S_2+(O_2/2)dt$	(7) H_2	(8) Outflow
0	0						
10	2	0.01	0.00	0.05	0.06	100.10	1
20	27	0.20	100.10	0.06	0.26	101.10	16
30	130	1.08	101.10	0.21	1.29	102.20	41
40	300	2.96	102.20	.061	3.57	104.10	100
50	360	4.55	104.10	2.20	6.75	105.60	175
60	289	4.47	105.60	4.40	8.87	106.25	217
70	194	3.33	106.25	5.80	9.13	106.30	220
80	133	2.25	106.30	5.90	8.15	105.05	205
90	91	1.54	106.05	5.30	6.84	105.65	177
100	61	1.05	105.65	4.50	5.55	105.10	147
110	37	0.67	105.10	3.60	4.27	104.50	116
120	20	0.39	104.50	2.70	3.09	103.80	87
130	11	0.21	103.80	1.90	2.11	103.05	64
140	5	0.11	103.05	1.18	1.29	102.25	43
150	1	0.04	102.25	0.63	0.67	101.40	22
160	0	0.00	101.40	0.35	0.35	100.70	10

Table 6B-1

Storage Facility Routing Calculations

CHAPTER 7

ENERGY DISSIPATION

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7.1 OVERVIEW

The purpose of this chapter is to aid in selecting and designing energy dissipators for controlling erosive velocities. Since some outlet protection is required downstream from all drainage facilities, the design of an energy dissipator becomes an integral part of the drainage facility design.

7.2 DESIGN CRITERIA

7.2.1 General Criteria

Energy dissipators shall be employed whenever the velocity of flows leaving a storm water management facility exceeds the erosive velocity of the downstream channel system. Several standard energy dissipator designs have been documented by the U.S. Department of Transportation including impact basins, drop structures, stilling wells, and riprap.

7.2.2 Erosion Hazards

Erosion problems at the outlets of culverts or detention basins are common. Determination of the flow conditions, scour potential, and channel erosion resistance should be standard procedure for all designs. The only safe procedure is to design on the basis that erosion at a culvert outlet and the downstream channel is to be expected.

Two types of scour can occur in the vicinity of culvert and other outlets; general channel degradation and local scour. Channel degradation may proceed in a fairly uniform manner over a long length, or may be evident in one or more abrupt drops progressing upstream with every runoff event. The abrupt drops, referred to as head cutting, can be detected by location surveys or by periodic maintenance following construction.

Local scour is the result of high-velocity flow at the culvert outlet, but its effect extends only a limited distance downstream. The highest outlet velocities will be produced by long, smooth-barrel culverts and channels on steep slopes. At most sites these cases will require protection of the outlet. However, protection is also often required for culverts and channels on mild slopes. For these culverts flowing full, the outlet velocity will be critical velocity with low tailwater and full barrel velocity for high tailwater.

Standard practice is to use the same treatment at the culvert entrance and exit. It is important to recognize that the inlet is designed to improve culvert capacity or reduce headloss while the outlet structure should provide a smooth flow transition back to the natural channel or into an energy dissipator. Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

7.2.3 Recommended Dissipators

For many designs, the following outlet protection and energy dissipators provide sufficient protection at a reasonable cost.

- Plunge pools
- Riprap apron
- Baffled outlets

This chapter will focus on riprap aprons. The reader is referred to the North Carolina Erosion and Sediment Control Planning and Design Manual and to the Federal Highway Administration Hydraulic Engineering Circular No. 14 (HEC- 14: Hydraulic Design of Energy Dissipators For Culverts and Channels) for design procedures of other energy dissipators.

7.3 RIPRAP APRONS

7.3.1 Uses

A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel, or onto a flat, open surface. Protection is provided primarily by having sufficient length, roughness, and flare to dissipate the energy by allowing the flow to expand and slow down. Riprap aprons shall be used when the culvert outlet velocity exceeds the erosive velocity of the receiving channel or soil. Riprap aprons are appropriate when the culvert outlet velocity is less than or equal to 10 fps for pipes ≤ 48 inches in diameter. Unincorporated Mecklenburg County and the six Towns require a velocity less than or equal to 5 fps for all pipes.

Riprap aprons downstream from flared end sections must adhere to standard 20.23 of the Charlotte and Mecklenburg County Land Development Standards Manuals.

7.3.2 Procedure

The procedure presented in this section is taken from USDA, SCS (19765). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d_{50}. If tailwater conditions are known, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, the can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

1. If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 7-1 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 7-2 should be used.

2. Determine the correct apron length and median riprap diameter, d50, using the appropriate curves from Figure 7-1 and 7-2. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 7-3.

   a. For pipes flowing full:

      Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50}, from the appropriate curves.

   b. For pipes flowing partially full:

      Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v

intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, L_a, from the scale on the left.

c. For box culverts:

Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.

3. If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 7-3. This will provide protection under either of the tailwater conditions.

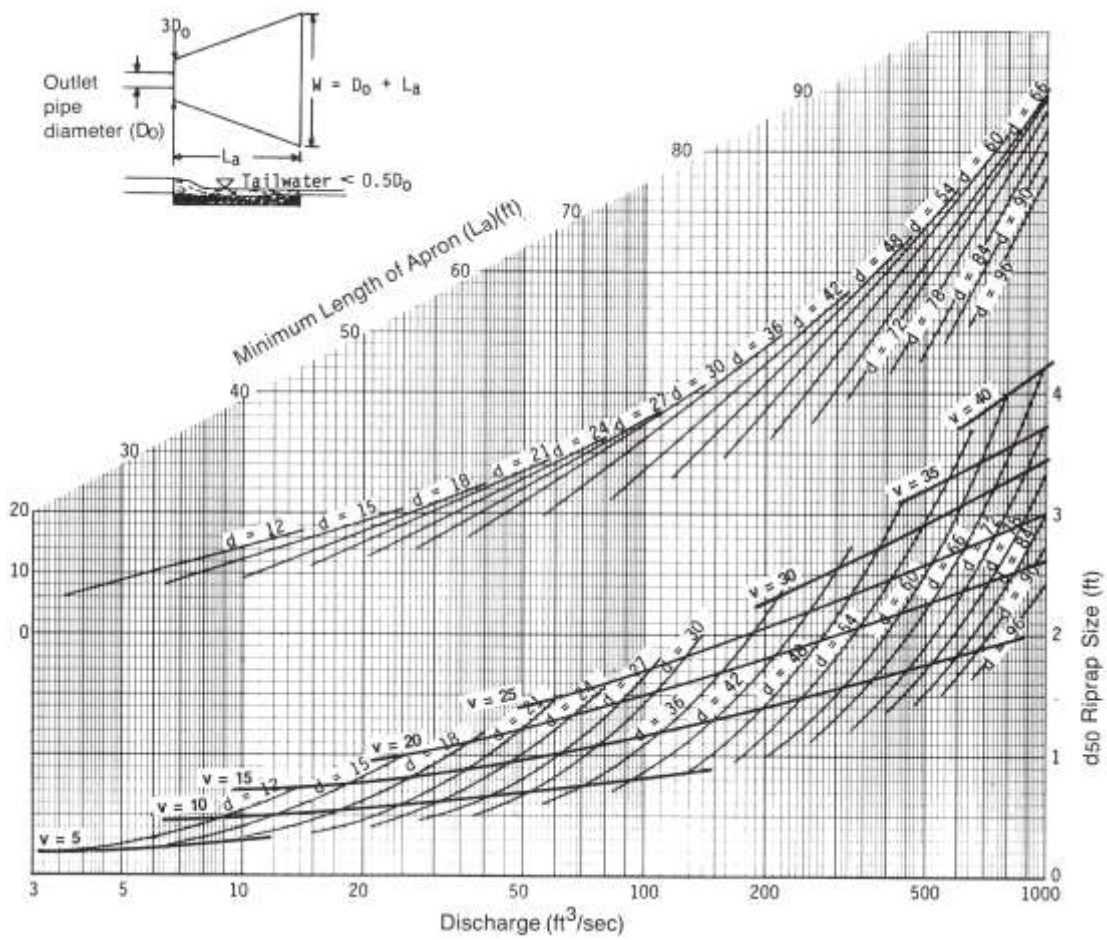

Figure 7-1 Design of Riprap Apron Under Minimum Tailwater Conditions

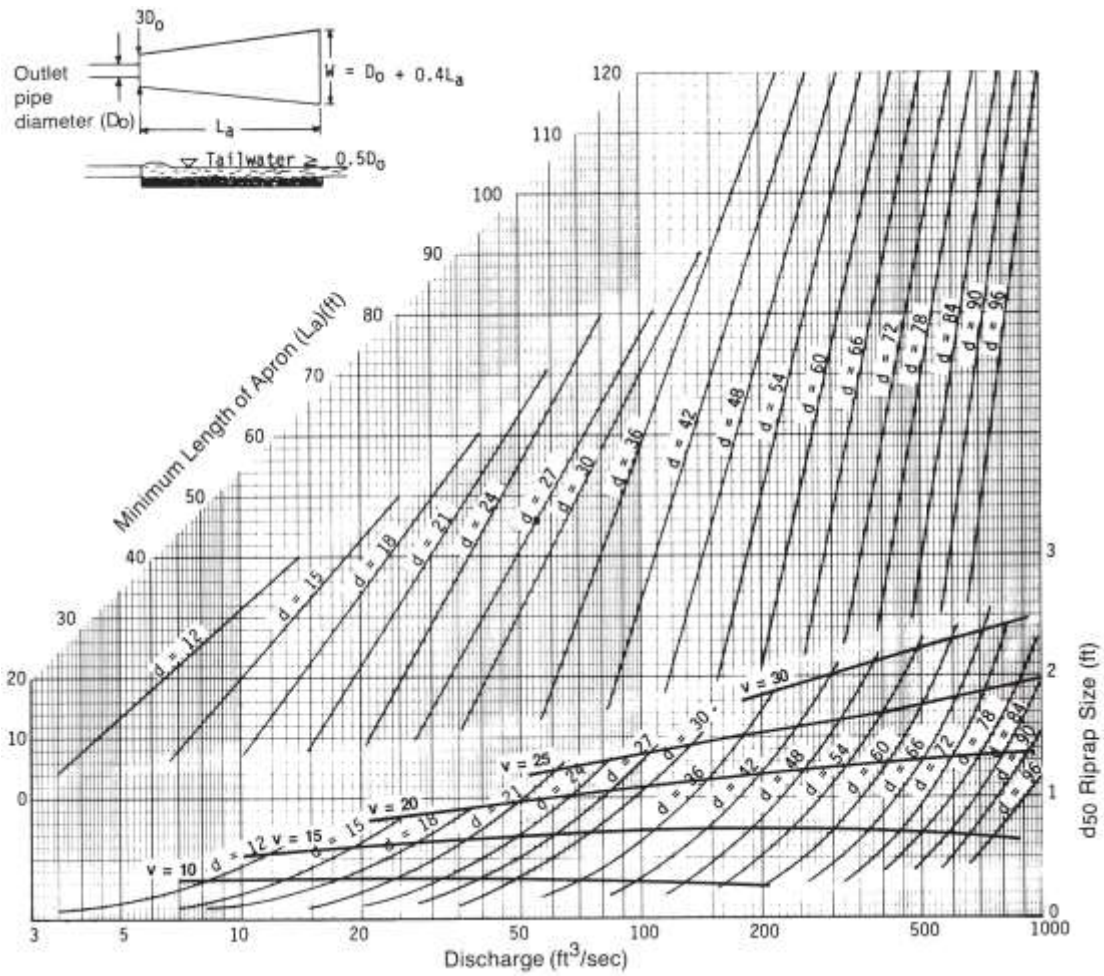

Figure 7-2 Design of Riprap Apron Under Maximum Tailwater Conditions

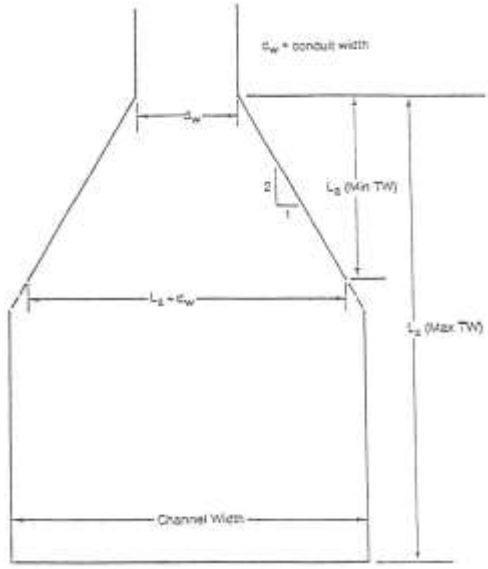

Figure 7-3 Riprap Apron Schematic For Uncertain Tailwater Conditions

7.3.3 Design Considerations

The following items should be considered during riprap apron design:

1. The maximum stone diameter should be 1.5 times the median riprap diameter.

   $d_{max} = 1.5 \times d_{50}$

   d_{50} = the median stone size in well-graded riprap apron. (See Figure 7-1 or 7-2)

2. The riprap thickness should be 1.5 times the maximum stone diameter or 10 inches, whichever is greater.

   Apron thickness = $1.5 \times d_{max}$

   After the d_{50}, d_{max} and the rip rap depth have been determined, specify the class of rip rap that will be used for the construction of the apron. Normally, Class 1 or Class 2 rip rap will be specified. Occasionally, Class B stone will be specified. Refer to the NCDOT Standard Specifications for Roads and Structures for stone grading and size information.

3. The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, d_w. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height, and should taper to the flat surface at the end of the apron.

4. If there is a well-defined channel, the apron length should be extended as necessary so that the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.

5. If the ground slope downstream of the apron is steep, channel erosion may occur. Either the apron should be extended as necessary until the slope is gentle enough to prevent erosion, or the pipe should be extended to a drop structure at the toe of slope, and the riprap apron placed on a flat surface or other in-stream drop structure created.

6. Keep the apron as straight as possible and align it with the flow in the receiving channel. DO NOT place apron at 90 degrees to the receiving waterway as it will cause erosion of the opposite bank. Discharge at an angle to the stream flow to smoothly combine the flows.

7. The approved plans shall accurately depict the limits of the apron as determined from the calculations. Easement widths shall encompass the entire apron.

8. The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete grout.

7.4 EXAMPLE PROBLEMS

7.4.1 Example 1

Riprap Apron Design for Minimum Tailwater Conditions

A flow of 280 cfs discharges from a 66-inch pipe with a tailwater of 2 ft above the pipe invert. Find the required design dimensions for a riprap apron.

1. Minimum tailwater conditions, d_e = 66 in = 5.5 ft
   Therefore, 0.5 d_e = 2.75 ft.

2. Since TW = 2 ft, use Figure 7-1 for minimum tailwater conditions

3. By Figure 7-1, the apron length, L_a, and median stone size, d_{50}, are 38 ft and 1.2 ft, respectively.

4. The downstream apron width equals the apron length plus the pipe diameter:

   $W = d + L_a = 5.5 + 38 = 43.5$ ft

5. Maximum riprap diameter is 1.5 times the median stone size:

   $1.5\ (d_{50}) = 1.5\ (1.2) = 1.8$ ft

6. Riprap depth = 1.5 (d_{max}) = 1.5 (1.8) = 2.7 ft. Using NCDOT Standard Specifications for Roads and Structures, Class 2 riprap should be specified.

7.4.2 Example 2

Riprap Apron Design for Maximum Tailwater Conditions

A concrete box culvert 5.5 ft high and 10 ft wide conveys a flow of 600 cfs at a depth of 5.0 ft. Tailwater depth is 5.0 ft above the culvert outlet invert. Find the required design dimensions for a riprap apron.

1. Compute 0.5 d_e = 0.5 (5.0) = 2.5 ft.

2. Since TW = 5.0 ft is greater than 2.5 ft, use Figure 7-2 for maximum tailwater conditions.

   $v = Q/A = 600/[(5)\ (10)] = 12$ ft/s

3. On Figure 7-2, at the intersection of the curve, de = 60 in and v = 12 ft/s, d_{50} = 0.4 foot. Reading up to the intersection with d = 60 in, find L_a = 40 ft

4. Apron width downstream = $d_w + 0.4\ L_a = 10 + 0.4\ (40) = 26$ ft.

5. Maximum stone diameter = 1.5 d_{50} = 1.5 (.4) = 0.6 foot

6. Riprap depth = 1.5 d_{max} = 1.5 (0.6) = 0.9 foot.

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CHAPTER 7 – ENERGY DISSIPATION

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Revision Log
Charlotte-Mecklenburg Storm Water Design Manual
October 7, 2013

General

1. Added a Log Sheet to the manual that will document the original date and future revision dates.
2. Deleted all text shading in manual.
3. Throughout manual replaced all references of "Charlotte-Mecklenburg Land Development Standards Manual" with "Charlotte Land Development Standards Manual".
4. Replaced the term urban with "developed" and the term rural with "undeveloped" throughout manual.
5. Replaced SCS with NRCS throughout manual.
6. Deleted Appendix A.
7. Revised example in Appendix B.
8. Reorganized data in Appendix C to make more readable.
9. Replaced all "storm sewer" terms throughout manual with "storm drainage".

Chapter 1

1. Page 1-2, Section 1.1: Revised text to read, "When accompanied by sound engineering judgement, application of the procedures and criteria presented in this manual should contribute toward the effective and economical solutions of local drainage and flooding problems."
2. Page 1-2, Section 1.2: Deleted "Shaded areas occurring throughout the manual represent design criteria that must be satisfied."
3. Page 1-3, Section 1.4: Replaced current text with the following: "The Charlotte-Mecklenburg Storm Water Design Manual is a dynamic document and is available at www.charmeck.org/Departments/StormWater/Contractors/storm+water+design+manual.htm and is updated periodically. It is the responsibility of the user to make sure the most recent standards are being specified. Users may request to receive notification of future revisions. The current manual is available via download only from this website. Hard copies or CDs of the Design Manual are no longer available from the City."
4. Page 1-3, Section 1.5: Deleted section 1.5 in its entirety.
5. Page 1-3, Section 1.6: Added the following bullet, "Discharge leaves site in the same direction and relative location as pre-developed condition."
6. Page 1-3, Section 1.6: Revised the text in bullet 3 to read, "Design and installation of all storm water detention facilities must comply with applicable federal, state and local laws. Attention should be given to the applicable Soil Erosion and Sediment Control Ordinances, Post Construction Stormwater Ordinance, and the North Carolina Dam Safety Law."
7. Page 1-3, Section 1.6: Revised text in bullet 4 to read, "Detention facilities located within code-required automobile parking areas shall not exceed a maximum water depth of 6 inches. Additional criteria are listed in Chapter 6, section 6.2.5."
8. Page 1-3, Section 1.6: Revised text in bullet 5 to read, "With the exception of erosion control facilities, all detention facilities shall be considered permanent."
9. Page 1-3, Section 1.6: Revised text in bullet 6 to read, "Routine maintenance of all detention facilities shall be the responsibility of the property owner or appointed designee."
10. Page 1-3, Section 1.6: Removed bullet 5 in its entirety.
11. Page 1-3, Section 1.6: Revised the text in bullet 9 to read, "Off site detention facilities are acceptable provided the land area involved with the facility is delineated on an acceptable

map and officially recorded at the Mecklenburg County's Register of Deeds Office as a "Permanent Detention Easement". Also, an official commitment to maintenance of the facility will be required."

12. Added new section 1.6 "City and County Storm Water Regulations".
    "It is the responsibility of the user to be fully aware of all applicable federal, state and local code requirements when using the Storm Water Design manual such as, but not limited to:
    - Floodplain Regulations
    - Soil Erosion & Sedimentation Control
    - Stormwater Ordinance
    - Subdivision Ordinance
    - Zoning Ordinance
13. Added new section 1.7 "Concept Definitions". Reviewed and revised all concept definitions throughout the manual and made sure there is consistency with the Floodway Ordinance and Subdivision Ordinance. All concept definitions were moved to under section 1.7 "Concept Definitions". A new definition was added for "Crown".

Chapter 2

1. Deleted Chapter 2 in its entirety (which means the new chapter two will be Hydrology).

Chapter 3

1. Renamed Chapter 2 – Hydrology.
2. Page 3-3, Section 3.1.1: Added "Hydraulic roughness" before Manning's n under Stream Channel Characteristics.
2. Page 3-3, section 3.1.1: Added "Storm Frequency Events" to Meterological Characteristics.
3. Page 3-4, section 3.1.2: Deleted "If other methods are used, they must first be calibrated to local conditions and tested for accuracy and reliability. In addition, complete source documentation must be submitted for approval."
4. Page 3-4, Table 2-1: Comment for Rational Method: Deleted "For storage design, the Rational method may be used to determine the peak discharge rate up to 50 acres. See section 7.7, 7.8, and 3.11"
5. Page 3-4, Table 2-1: Deleted Basin Lag-Time from the table – Method, Size Limitations and Comments.
6. Page 3-4, Table 3-1: Deleted SCS Step Function from the table – Method, Size Limitations and Comments.
7. Page 3-5, Table 3-1: Replaced "HEC-1" with "HEC-1/HEC-HMS". Revised the maximum number of ordinates in the Comments from "300" to "2,000".
8. Page 3-5, Table 3-1: Deleted Graphical Method from the table – Method, Size Limitations and Comments.
9. Page 3-5, Table 3-1: Added the following note to the table, "If other methods are used, they must first be calibrated to local conditions and tested for accuracy and reliability by the user. Third party computer software not identified in this table must be independently verified and calibrated to the recommended methods by the professional prior to its use. If other software is used, it will be compared to HEC-1/HEC-HMS to make sure it reproduces equivalent results. In addition to verifying results, complete source documentation for the software must be submitted for approval."
10. Page 3-5, Section 3.1.3: Revised title to read, "Storm Water Conveyance Design Policy." Replaced current text with: "All storm water conveyances shall be designed based on fully developed land use conditions as shown on current County and City Land Use Plans and Zoning Maps or existing land use, whichever generates the higher runoff rate."
11. Page 3-5, Section 3.1.4: Deleted Section 3.1.4 in its entirety.

12. Page 3-5a, Section 3.1.5: Added the following text to the first bullet, "The computation interval, when multiplied by the number of hydrograph ordinates, must also be greater than the storm duration which is planned to be studied (6 hour, 24 hour, etc.). Not having the program set to allow the storm to run causes hydrographs to be inappropriately peaked due to the lack of necessary time to fit in the needed runoff hydrograph."
13. Page 3-6, Section 3.2: Deleted Section 3.2 in its entirety.
14. Page 3-8, Section 3.4: Deleted Section 3.4 in its entirety.
15. Page 3-10, Section 3.5.1: Revised the section to read:

Description	***Design Storm***
Storm system pipes	10 year
Ditch systems	10 year
Culverts/Cross-drain (subdivision streets)	25 year
Culverts/Cross-drain (thoroughfare roads)	50 year
Culverts (over regulated floodways)	100 year
Culverts/Cross-drain (primary access streets)	No overtopping in 100 year
Usable and functionable part of structure or building (as defined in the Subdivision Ordinance)	100 year + 1 foot

16. Page 3-10, Table 3-3: Added intensity equation and P_{24} (inches) records at bottom of the table.
17. Page 3-11, Section 3.6.1: Revised the first bullet to read, "In determining the C value (land use) for the drainage area, hydrologic analysis should take into account future land use changes. Drainage facilities shall be designed for future land use conditions as specified in the County and City Land Use Plans and Zoning Maps (or existing land use, whichever generates the higher runoff rate)."
18. Page 3-11, Section 3.6.2: Revised equation 3.1 to include the frequency factor (C_f) from Section 3.6.3.
19. Page 3-11, Section 3.6.3: Deleted Section 3.6.3in its entirety.
20. Page 3-12, Table 3-4: Updated table to include the 2 and 10-year recurrence intervals and corresponding frequency factor.
21. Page 3-12, Section 3.6.4: Reworded Time of Concentration section and included the Kirpich Equation.
22. Page 3-13, Figure 3-1: Delete Figure 3-1 Nomograph for Time of Concentration
23. Page 3-14, Table 2-5: Changed "Parks & cemeteries" to "Bare soils". Added "Note: The above runoff coefficients are valid for 2-year to 10-year storm frequencies only. Coefficients must be accompanied with a C_f factor when used for less frequent, higher intensity storms."
24. Page 3-16, Section 3.7: Updated Rational Method Example Problem.
25. Page 3-18, Section 3.8: Deleted Section 3.8 in its entirety.
26. Page 3-20, Section 3.9.2: Deleted Section 3.9.2 in its entirety.
27. Page 3-21, Section 3.9.3, Updated Rainfall section to read, "Rainfall—The NRCS method applicable to the Charlotte-Mecklenburg area is based on a storm event which has a Type II time distribution. Figure 2-1 shows this distribution. To use this distribution for the one year storm event it is necessary for the user to obtain the 24-hour rainfall intensity from Table 2-3 and multiply by the 24-hour duration to obtain the depth(P_{24} in Figure 2-1) from Table 2-6. This depth is then distributed according to Figure 2-1. To use this distribution for other storm events it is necessary for the user to obtain the 6-hour rainfall intensity from Table 2-3 and multiply by the 6-hour duration to obtain the depth (P_6 in Figure 2-1) from Appendix 2B for the frequency of the design storm. This depth is then distributed according to Figure 2-1. Rainfall may also be distributed using a center weighted "balanced" distribution. Tables 2-6 thru 2-11 show a balanced distribution for various storm events."

28. Page 3-22, Table 3-7: Deleted Table 3-7 Ratios for Dimensionless Unit Hydrograph and Mass Curve.
29. Page 3-23, Figure 3-2: Deleted Figure 3-2 Dimensionless Unit Hydrograph and Mass Curve.
30. Page 3-26, Table 3-8: Updated Table 3-8 to include 1-year, 25-year, 50-year and 100-year storm data.
31. Page 3-29, Section 3.9.5: Renamed section "Modifications for Developed Conditions".
32. Page 3-30, Table 3-11: Updated Table 3-11. Removed notes 4 and 5 from the table.
33. Page 3-31, Section 3.9.6.1: Revised travel time equation (3.15).
34. Page 3-32, Section 3.9.6.3: Added the following text, "Also please note, when designing a drainage system, the sheet flow path is not necessarily the same before and after development and grading operations have been completed. Selecting sheet flow paths in excess of 100 feet in developed areas and 300 feet in undeveloped areas should be done only after careful consideration."
35. Page 3-32, Section 3.9.6.3: Adjusted Equation 3.17 to report time of concentration in minutes.
36. Page 3-33, Table 3-12: Updated Manning's *n* value for smooth surfaces (concrete, asphalt, gravel or bare soil).
37. Page 3-34, Section 3.9.6.5: Renamed the section "Channelized Flow".
38. Page 3-34, Section 3.9.6.4: Added the following text, "Flow within pipes and culverts not under pressure is considered closed channel flow" and "Manning's velocity for pipes assumes a fully flowing condition."
39. Page 3-35, Figure 3-5: Deleted Figure 3-5 Average Velocities – Shallow Concentrated Flow.
40. Page 3-36, Section 3.9.6.7: Delete the third bullet.
41. Page 3-37, Section 3.9.7: Remove section 3.9.7 in its entirety.
42. Page 3-38, Table 3-13: Deleted Table 3-13 Hydrograph Spreadsheet.
43. Page 3-42, Figure 3-6: Deleted Figure 3-6 Unit Hydrograph.
44. Page 3-43, Figure 3-7: Deleted Figure 3-7 Resulting Hydrograph.
45. Page 3-44, Section 3.10: Deleted Section 3.10 in its entirety.
46. Page 3-46, Figure 3-8: Deleted Figure 3-8 SCS Type II Unit Peak Discharge Graph.
47. Page 3-47, Table 3-15: Deleted Table 3-15 I_a Values for Runoff Curve Numbers.
48. Page 3-51, Section 3.11: Deleted Section 3.11 in its entirety.
49. Page 3-55, Table 3-16: Deleted Table 3-16 Example Hydrograph Results.
50. Page 3-58, Figure 3-9: Deleted Figure 3-9 Resulting Hydrographs.
51. Page 3-60, Appendix A: Deleted Appendix A in its entirety.
52. Page 3-61, Appendix B: Renamed to Appendix A.
53. Page 3-63, Table B-1: Expanded Table B-1 to include additional data.
54. Page 3-64, Appendix C: Renamed to Appendix B.

<u>Chapter 4</u>

1. Renamed Chapter 3 – Open Channel Hydraulics.
2. Page 4-2, Section 4.1.1: Added the following text, "For information on roadside ditch requirements see Chapter 4."
3. Page 4-2, Section 4.1.2.2: Revised the last sentence to read, "The use of flexible lining may be restricted where space is limited, since the introduction of linings with higher roughness values generally results in the need for greater channel capacity."
4. Page 4-4, Section 4.2: Deleted Section 4.2 in its entirety.
5. Page 4-5, Section 4.3.1: Revised the text to read, "The following criteria shall be used for open channel design:

1. Channel side slopes shall be stable throughout the entire length and slope shall be a maximum of 2:1.
2. Superelevation of the water surface at horizontal curves shall be accounted for by increased freeboard.
3. A minimum freeboard of 6" must be provided in the 10-year design storm.
4. Transition from closed systems to channel sections (or between transitioning channel sections) shall be smooth and gradual, with a minimum of 5:1 taper.
5. Low flow sections shall be considered in the design of channels with large cross-sections
   (Q > 100 cfs). Some channel designs will be required to have increased freeboard (see North Carolina Erosion and Sediment Control Planning and Design Manual, Section 8.05.21)."

6. Page 4-5, Section 4.3.2: Revised the first sentence to read, "Open channel drainage systems shall be designed to convey a 10-year design storm. The peak flow rate for the 100-year storm shall be computed at appropriate points within the drainage system to determine if a 100 + 1 flood study is required as described in section 3.2.3."
7. Page 4-5, Section 4.3.3: Replaced "streams" with "storm water conveyances" in the second sentence.
8. Page 4-5, Section 4.3.3: Revised the bullets to read as follows:
   2. The 100-year storm water surface elevations should be calculated using a method acceptable to the City/County Engineering Department, as further described in Section 3.6.
   3. The peak flow rate used in the 100+1 analysis shall be based on an assumption of full build out of the tributary drainage area. The assumption of full build out shall be defined as either full development per the current zoning of the property, the existing land use, or adopted area land use plans, whichever generates the higher runoff rate.
   4. For drainage systems within development projects subject to the Subdivision Ordinance, the 100+1 elevation and flood limits shall be shown on the recorded maps associated with the subdivision as further described in the Subdivision Ordinance.
   5. For drainage systems within development projects not subject to the Subdivision Ordinance, the City/County Engineering Department may require that the 100+1 elevation be shown on a recorded map if the engineering analysis indicates that one of the following conditions is present:
      - The 100+1 line would exceed the set-back limits.
      - The estimated runoff or proposed modifications to a storm water conveyance would create a hazard for the adjacent properties or residents.
      - The flood limits would be of such magnitude that adjacent property owners should be informed of these limits.

6. Page 4-6, Section 4.3.4: Revised the text to read, "The design of open channels should be consistent with the velocity limitations for the selected channel lining. For design information see section 3.4 Open Channel Design."
7. Page 4-7, Section 4.4: Moved Section 4.4 to Appendix 3A.
8. Page 4-11, Table 4-2: Revised recommended Manning's n values in Table 4-2.
9. Page 4-14, Section 4.6: Moved Section 4.6 to Appendix 3B. Removed the following sentence from Section 4.6.1, "A minimum freeboard of 6" must be provided."
10. Page 4-16, Section 4.7: Moved Section 4.7 to Appendix 3C.
11. Page 4-17, Section 4.7.4: Deleted Section 4.7.4 in its entirety.

12. Page 4-18, Figure 4-4: Deleted Figure 4-4 Nomograph for the Solution for Manning's Equation.
13. Page 4-19, Figure 4-5: Deleted Figure 4-5 Solution of Manning's Equation for Trapezoidal Channels.
14. Page 4-23, Section 4.8: Moved Section 4.8 to Appendix 3D.
15. Page 4-23, Section 4.8.1: Revised Equation 4.13.
16. Page 4-26, Section 4.8.3: Revised Equation 4.15.
17. Page 4-27, Section 4.9: Incorporated text, figures and tables from the North Carolina Erosion and Sediment Control Planning and Design Manual.
18. Page 4-28, Section 4.10.1: Deleted the following text, "The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). Also, added the following sentence, "Where riprap is used in ditches/channels that are 2% or less, the design should take into account the potential for sedimentation."
19. Added Figure 3-2 K_b Factor for Maximum Shear Stress on Channel Bends.
20. Added Figure 3-3 Protection Length, $_{LP}$, Downstream from Channel Bend.
21. Page, 4-34, Section 4.11: Deleted Section 4.11 in its entirety.
22. Page 4-35, Table 4-5: Deleted Table 4-5 Water Surface Profile Computation Form for the Direct Step Method.
23. Page 4-38, Table 4-6: Deleted Table 4-6 Water Surface Profile Computation Form for the Standard Step Method.
24. Page 4-40, Section 4.12: Deleted Section 4.12 in its entirety.
25. Page 4-41, Table 4-7: Deleted Table 4-7.
26. Page 4-43, Table 4-8: Deleted Table 4-8.
27. Page 4-45, Section 4.13: Revised last sentence to read, "This requires a backwater analysis to determine the stream flow depth. The HEC-RAS software package is an acceptable method."

<u>Chapter 5</u>

1. Renamed Chapter 4 – Storm Drainage Systems.
2. Updated/Deleted standard numbers or replaced with a description.
3. Page 5-2, Section 5.1.1: Replaced "HEC -12" with "HEC-22 (USDOT, FHWA, 1996)".
4. Page 5-2, Section 5.1.2: Replaced the work "sump" with "sag".
5. Page 5-2, Section 5.1.3: Revised the criteria as follows:
    a. Replaced the word "sump" with "sag".
    b. Moved the Design Frequencies section to Chapter 2, Section 2.3.1 and added a reference for this.
    c. Clarified the design criteria for sag inlets in 25-year event for local roads and added the 50-year event design criteria for thoroughfare roads, based on the NC Division of Highways Standard.
    d. Added criteria for no surcharging of sag inlets in the 25-year event for local roads and 50-year event for thoroughfare roads.
    e. Added criteria to evaluate the need for flanking inlets near sag inlets based on slope check points.
    f. Added criteria for concentrated flow over City sidewalks.
    g. Added design criteria for roadside ditches and driveway culvert sizing for the 25-year event for local roads and the 50-year event for thoroughfare roads.
6. Page 5-3, Section 5.2: Deleted Section 5.2 in its entirety.
7. Page 5-4, Section 5.3: Deleted Section 5.3 in its entirety.
8. Page 5-6, Section 5.4.6: Revised text to read, "Curb and gutter installation shall be designed in accordance with the relevant standards for the jurisdiction."

9. Page 5-7, Section 5.4.8" Revised the text to read, "Roadside ditches (when allowed) will be required behind the shoulder of roadways without curb and gutter to convey storm drainage away from the pavement to a discharge point. The steepest side slope allowed is 3:1 (horizontal to vertical) on the roadside of the ditch and 2:1 on the side closest to the right-of-way line. The ditch shall be graded to a minimum longitudinal slope of 1 percent and a maximum velocity of 4 ft/sec. For grass lined channels with velocities up to 7 ft/sec, permanent matting may be approved on a case by case basis. For velocities greater than 7 ft/sec, a concrete lined ditch may be required. Riprap will not be allowed for stabilization within the street right-of-way (except as outlet protection on culverts).

   In addition to the design of roadside ditches, a design shall be provided for driveway culverts for each individual lot on the plan. The use of a small driveway culvert, 15 inches minimum diameter, in conjunction with overtopping of the driveway itself will be allowed as further described in Section 4.1.3. Sizes for all driveway culverts shall be shown in tabular form on the plans, and each culvert shall be designed for the highest ditch flow applicable for the lot."
10. Page 5-9, Section 5.5.2: Replaced and "Q" with "Q_{cap}" to be consistent.
11. Page 5-11, Section 5.6.11: Clarified to also use "Q_{cap}" consistently.
12. Page 5-16, Section 5.6.2.2: Replaced "drop inlet" with "grated drop inlet"
13. Added open throat (slab top) catch basin design information with an example problem after the Drop Inlet section (Section 5.6.2.2). Named this section 4.4.2.3 Open Throat (Slab Top) Catch Basin.
14. Page 5-23, Section 5.9.2: Added the following design criteria, "The maximum discharge velocity at pipe outlets is 10 fps except for pipes greater than 48 inches in diameter unless velocity is further restricted for energy dissipation."
15. Page 5-24, Section 5.9.4: Deleted Section 5.9.4 in its entirety.
16. Page 5-25, Figure 5-6: Deleted Figure 5-6 Nomograph for Solution of Manning's Formula for Flow in Storm Sewers.
17. Page 5-26, Figure 5-7: Deleted Figure 5-7 Nomograph for Computing Required Size of Circular Drain, Flowing Full – n = 0.013 or 0.015.
18. Page 5-27, Figure 5-8: Deleted Figure 5-8 Concrete Pipe Flow Nomograph.
19. Page 5-28, Figure 5-9: Deleted Figure 5-8 Values of Various Elements of Circular Section for Various Depths of Flow.
20. Page 5-29, Section 5.9.5: Added the following sentence, "If computer models are utilized then results should be consistent with the procedure outlined below. All input data should be supplied, including loss coefficients, and output should be in similar format to Figure 4-6."
21. Page 5-24, Section 5.9.3: Corrected Equation 5.20.
22. Page 5-29, Section 5.9.5.1: Corrected Equation 5.22.
23. Page 5-31, Section 5.9.6: Corrected Equation 5.25.
24. Page 5-36, Section 5.10: Deleted Section 5.10 in its entirety.

<u>Chapter 6</u>

1. Renamed Chapter 5 – Design of Culverts
2. Page 6-4, Section 6.2: Deleted Section 6.2 in its entirety.
3. Page 6-6, Section 6.4: Deleted Section 6.4 in its entirety.
4. Page 6-8, Section 6.5.3: Revised language to read as follows, "The appropriate flood frequency for determining the flood carrying-capacity of a culvert is dependent upon:

- the level of risk associated with failure of the culvert crossing; and
- the level of risk associated with increasing the flood hazard to upstream (backwater) or downstream (redirections of floodwaters or loss of attenuation) properties.

For specific design storm frequencies for culvert crossings, reference Section 2.3.1, Design Frequencies. Also, in compliance with the National Flood Insurance Program, it is necessary to consider the 100-year frequency flood at locations identified as being special flood hazard areas. The design engineer should review the City and County floodway regulations for more information related to floodplain regulations."

5. Page 6-8, Section 6.5.4: Deleted the following text, "The maximum allowable velocity within corrugated metal pipe is 10 fps. There is no specific maximum allowable velocity within reinforced concrete pipe, but outlet protection shall be provided where discharge velocities will cause erosion problems. The maximum discharge velocity at pipe outlets is 10 fps except for pipes > 48 inches in diameter."
6. Page 6-12, Section 6.5.9: Revised the text to read, "The relative efficiency of the inlet depends on the conduit. Headwalls are required for all metal and HDPE culverts. Concrete flared end sections may be utilized in lieu of headwalls upon approval of the City Engineer. The figure below illustrates the use of headwalls and wingwalls. Corrugated metal pipe in a headwall is essentially square-edged with an inlet coefficient of about 0.5."
7. **P**age 6-13, Section 6.5.12: Revised the text to read, "For culvert selection, only reinforced concrete pipe is allowed within the street right-of-way except for culverts equal to or greater than 60 inches. For culverts equal to or greater than 60 inches in diameter, aluminum or aluminized steel pipe is allowed."
8. Page 6-13, Table 6-3: Revised Table 6-3 and moved it to Table 4-3 in Chapter 4.
9. Page 6-14, Section 6.5.14: Revised the text to read, "In addition to controlling erosion, sedimentation and debris at the culvert site, care must be exercised in selecting the location of the culvert site. Environmental considerations are a very important aspect of culvert selection and design.

   This selection must consider the entire site and include provisions for maintaining existing stream cross section at the inlet and outlet while providing passage of aquatic life."
10. Page 6-19, Section 6.7.1: Deleted the following text, "It is recommended that the HYDRAIN (HY8) computer model be used for culvert design since it will allow the engineer to easily develop performance curves rather than only examining one design situation. The personal computer system HYDRAIN uses the theoretical basis for the nomographs to size a culvert. In addition, this system can evaluate improved inlets, route hydrographs, consider road overtopping and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scours."
11. Page 6-24, Section 6.7.7: Removed Section 6.7.7 in its entirety.
12. Page 6-30, Section 6.10: Moved this section to Appendix 5B.
13. Page 6-35, Section 6.11: Deleted Section 6.11 in its entirety.
14. Page 6-40, Appendix B: Deleted Appendix B in its entirety.

<u>Chapter 7</u>

1. Renamed Chapter 6 - Storage and Detention
2. Page 7-2, Section 7.1.2: Revised Section 7.1.2 to refer to Fee Credit Manual
3. Page 7-3, Section 7.2: Deleted section 7.2 in its entirety.

4. Page 7-4, Section 7.3: Added Section 6.2.1 Introduction and the following text, "If you do not comply with the Post Construction Control Ordinance then you must follow requirements in the zoning ordinance and the control requirements of 6.2.3 and 6.2.4 apply."
5. Page 7-4, Section 7.3.3: Added a new section 6.2.5 Storage within Parking Areas and moved the following text from Section 7.3.3 to Section 6.2.5, "Storage within code-required parking areas are allowed a maximum depth of 6 inches; additional parking areas 10 inches; and 15 inches is allowed in truck storage and loading areas."
6. Page 7-5, Section 7.3.4: Revised the text to read, "A concrete paved low flow or pilot channel (minimum slope of ½ %) constructed across the facility bottom from the inlet to the outlet shall be considered for conveyance of low flows to prevent standing water conditions."
7. Page 7-5, Section 7.3.4: Revised the text to read, "A minimum freeboard of 6 inches above the 50-year design storm high water elevation shall be provided all impoundments. Storage facilities under the jurisdiction of the North Carolina Department of Environment and Natural Resources (NDCENR), Dam Safety Program of Land Quality are in addition subject to the requirements of the Safe Dams Act (see Section 6.2.9)."
8. Page 7-5, Section 7.3.5: Added the following text, "Minimum barrels through embankments are 12 inch pipes with corresponding orifice plates. Any orifice smaller than 4 inches in diameter must be protected to prevent blockage. A 2 foot by 2 foot concrete pad must be placed in front of any orifice plate at the invert of the outlet. If the spillway is in fill material then the spillway must be lined."
9. Page 7-5: Added Section 6.2.8 Off-site Storm Water Detention Facilities and the following text, "When off-site storm water detention facilities (or for storm water release through recorded easements to regulated floodway in lieu of detention) the following requirement must be met:
   - All pipes/channels leading from the subject site to the off-site storm water detention facility (or regulated floodway) must be sized to carry the 10-year storm water runoff.
   - A "Permanent Detention Easement" leading from the subject site to the off-site detention facility (or regulated floodway) must be shown on a map which has been recorded with the Mecklenburg County Register of Deeds Office. This easement should be centered on the pipe or channel and must be at least 15 feet wide for pipes and 20 feet wide for channels (refer to City Std. #20.30 for easement widths). A metes and bounds description is not required for this portion of the easement.
   - A "Permanent Detention Easement" which encompasses the detention facility must be shown on the recorded map. This easement must be described by metes and bounds.
   - The recorded map must have a note which clearly states who is responsible for maintenance of the detention facility, pipes, and/or channels located within the Permanent Detention Easements (these easements will not be maintained by the City).
   - For off-site detention facilities the recorded map must have a note stating, "The purpose of the Permanent Detention Easement is to provide storm water detention for Lot(s) _____. The pipes and/or channels located within the Permanent Detention easement and leading to the detention facility carry unrestricted storm water flow from the developed upstream Lot(s) ___.
   - For storm water released to regulated floodway through an easement, the recorded map must have a note stating "The purpose of the Permanent Detention Easement is to allow storm water release directly to regulated floodway in lieu of on-site storm water detention. The pipes and/or channels located within the Permanent Detention Easement and leading to the regulated floodway carry unrestricted storm water flow from the developed upstream Lot(s) ___."
10. Page 7-5, Section 7.3.6: Updated text to read, "Under the Dam Safety Law regulations, a dam is a structure and appurtenant works erected to impound or divert water that is 25 feet

or greater in height and has a maximum storage volume of 50 acre-feet or more. A number of exemptions are allowed from the Dam Safety Law and any questions concerning a specific design or application should be addressed to the North Carolina Department of Environment and Natural Resources, Dam Safety Program of Land Quality (919-707-9220)."

11. Page 7-7, Section 7.4.2: Add the following language, "Computer based methods, such as HEC-1/HEC-HMS, are widely available to perform these iterations quickly. Other computer programs can provide similar results. For guidelines for using other party software, please refer to Section 2.1.2 – Hydrologic Method."
12. Page 7-8, Section 7.5.2: Updated the equation for a sharp-crested weir with no end contractions (Equation 7.1). Removed formulas for sharp-crested weir with two end contractions (Equation 7.2) and the discharge equation for a sharp-crested submerged weir (Equation 7.3).
13. Page 7-9, Section 6.4.3: Revised Equation 7.4 so L is defined as the horizontal weir length (ft). Removed the following text, "If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 7.2 on the next page."
14. Page 7-10, Table 7-2: Deleted Table 7-2 Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breath and Head.
15. Page 7-11, Section 7.5.5: Deleted Section 7.5.5 in its entirety.
16. Page 7-11, Section 7.5.6: Revised Equation 7.8 so that C = 0.60. Removed the following text, "Any orifice smaller than 4 inches in diameter must be protected to prevent blockage."
17. Page 7-12, Section 7.6: Moved Section 7.6 to Appendix A
18. Page 7-15, Section 7.7: Deleted Section 7.7 in its entirety.
19. Page 7-23, Section 7.8: Deleted Section 7.8 in its entirety.
20. Page 7-25, Figure 7-3: Deleted Figure 7-3 Triangular Approximation.
21. Page 7-28, Figure 7-3: Deleted Figure 7-4 Variation in Attenuation Ratio as a Function of N_r and a for Reservoirs with Spillway Outlet.
22. Page 7-28, Figure 7-3: Deleted Figure 7-5 Variation in Attenuation Ratio as a Function of N_r and a for Reservoirs with Orifice Outlet.
23. Page 7-31, Section 7.9: Moved Section 7.9 to Appendix A.
24. Page 7-35, Section 7.10: Revised bullet 6 under proper design to read, "Outlet structures should be selected to minimize the possibility of blockage (i.e., very small pipes tend to become blocked quite easily and should be avoided). Outlets shall be no less than 4 inches in diameter."
25. Page 7-35, Section 7.10: Revised bullet 4 under proper design to read, "Standing water or soggy surfaces <u>must</u> be eliminated…"
26. Page 7-35, Section 7.10: Revised bullet 6 under proper design to read, "Outlet structures should be selected to minimize the possibility of blockage (i.e., very small pipes tend to become blocked quite easily and should be avoided). Outlets shall be no less than 4 inches in diameter."

<u>Chapter 8</u>

1. Renamed Chapter 7 – Energy Dissipation.
2. Page 8-2, Section 8.2: Deleted Section 8.2 in its entirety.
3. Page 8-4, Section 8.4.1: Revised to read as follows "A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel, or onto a flat, open surface. Protection is provided primarily by having sufficient length,

roughness, and flare to dissipate the energy by allowing the flow to expand and slow down. Riprap aprons shall be used when the culvert outlet velocity exceeds the erosive velocity of the receiving channel or soil. Riprap aprons are appropriate when the culvert outlet velocity is less than or equal to 10 fps for pipes ≤ 48 inches in diameter. Unincorporated Mecklenburg County and the six Towns require a velocity less than or equal to 5 fps for all pipes."

4. Page 8-8, Section 8.4.3: Added the following text under bullet 2, "After the d_{50}, d_{max} and the rip rap depth have been determined, specify the class of rip rap that will be used for the construction of the apron. Normally, Class 1 or Class 2 rip rap will be specified. Occasionally, Class B stone will be specified. Refer to the NCDOT Standard Specifications for Roads and Structures for stone grading and size information."
5. Page 8-8, Section 8.4.3: Revised bullet 5 to read, "If the ground slope downstream of the apron is steep, channel erosion may occur. Either the apron should be extended as necessary until the slope is gentle enough to prevent erosion, or the pipe should be extended to a drop structure at the toe of slope, and the riprap apron placed on a flat surface or other in-stream drop structure created."
6. Page 8-8, Section 8.4.3: Revised bullet 6 to read, "Keep the apron as straight as possible and align it with the flow in the receiving channel. DO NOT place apron at 90 degrees to the receiving waterway as it will cause erosion of the opposite bank. Discharge at an angle to the stream flow to smoothly combine the flows."
7. Page 8-8, Section 8.4.3: Added the following text for bullet 7, "The approved plans shall accurately depict the limits of the apron as determined from the calculations. Easement widths shall encompass the entire apron."
8. Page 8-8, Section 8.4.3: Added the following text for bullet 8, "The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete grout."
9. Page 8-9, Section 8.4.4: Delete d 8.4.4 in its entirety.
10. Page 8-10, Figure 8-4: Deleted Figure 8-4 Riprap Apron Design Graph
11. Page 8-11, Figure 8-5: Deleted Figure 8-5 Riprap Apron Design Chart
12. Page 8-12, Figure 8-6: Deleted Figure 8-6 Maximum Stone Size for Riprap
13. Page 8-13, Figure 8-7: Deleted Figure 8-7 Gradation of Riprap