Town of Waxhaw

Stormwater Design Manual

Version 2.0

Updated: April 22, 2009

April 22, 2009

Dear Professional Engineer,

The purpose of this Stormwater Design Manual is to provide you with a guide to the planning and design of stormwater control structures and systems in the Town of Waxhaw, N.C., that will meet the requirements of the Town's Zoning Ordinance and good engineering practice.

This Manual will prove to be a help to you in the design of stormwater structures and systems. It has been modified from the Charlotte-Mecklenburg Storm Water Design Manual to suite conditions in the Town of Waxhaw.

Due to the expansion of Waxhaw in recent years the Town Planner, the Mayor and, the Board of Commissioners have seen the need for the control of stormwater. In order to protect the streams and public and private properties in the area the Stormwater Design Manual has been produced and implemented since 2000.

Standards and minimum design requirements for projects to be submitted for approval are indicated in the Manual by shading. Additional methods and techniques for analysis are provided for the convenience of the designer, and may be required by the Town Engineer when circumstances warrant. Please note that the procedures and methods included in the Manual are intended to supplement, not replace, the expertise and engineering judgment of the designer. Alternative techniques may be used when their validity can be justified.

Recently, the Stormwater Design Manual has been made available via the internet free of charge, through the town's website. This has phased out the once available hardcopy available for purchase. Periodically the town will update the Design Manual to better suit the towns stormwater needs. These updates will be made available through the town's website. Please feel free to contact us with any questions.

Sincerely,

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Gregory J. Mahar
Director of Planning, Zoning and Engineering Services
Town of Waxhaw, N.C.

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Chapter Table of Contents

1.1 Introduction

Purpose 1.1

This manual has been developed to assist in the design and evaluation of storm water management facilities in the Town of Waxhaw, N.C. area. It provides engineering design guidance to:

- *local agencies responsible to coordinate with the Town of Waxhaw 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*

Application of the procedures and criteria presented in this manual should contribute toward the effective and economical solution of local drainage and flooding problems.

Engineering design methods other than those included in this manual may be used if approved by the Town Engineer. Complete documentation of these methods will be required for approval.

Contents 1.2

The manual presents technical and engineering procedures and criteria needed to comply with Town of Waxhaw storm water regulations. The Town of Waxhaw Stormwater Ordinance is contained in Chapter 2. Following are the chapers included in the manual.

- *Chapter 1 Introduction*
- *Chapter 2 Storm Water Management Regulations*
- *Chapter 3 Hydrology*
- *Chapter 4 Open Channel Hydraulics*
- *Chapter 5 Storm Drainage Systems*
- *Chapter 6 Culverts*
- *Chapter 7 Storage and Detention*
- *Chapter 8 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.

Shaded areas occurring throughout the manual represent design criteria that must be satisfied.

Limitation 1.3

The manual provides a compilation of readily available literature relevant to storm water management activities in the Town of Waxhaw. 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 entirely, the user is encouraged to obtain original or additional reference material, as appropriate. References are included at the end of each chapter.

Updating 1.4

This manual will be updated and revised, as necessary, to reflect up-to-date engineering practices and information applicable to the Town of Waxhaw. Registered manual users who provide a current address will automatically be sent revisions when they occur.

Erosion Control & Stormwater Details 1.5

For erosion and sediment control see the North Carolina Erosion and Sediment Control Planning and Design Manual. The Charlotte-Mecklenburg Land Development Standards Manual will be used as a reference for soil erosion and stormwater details.

Design & Construction Criteria 1.6

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

- *Design and installation of all storm water detention facilities must comply with applicable Federal, State and local laws. Attention should be given to the North Carolina Dam Safety Law of 1967 and its amendments.*
- *In no case shall a building be located with the impoundment area of any storm Water storage facility or over a storm drainage line.*
- *No utilities (sewer lines, power lines, water lines, etc.) shall be located within or Under any detention facility.*
- *Detention facilities located within automobile parking areas shall not exceed a Maximum water depth of 6 inches.*
- *All detention facilities will be considered permanent.*
- *Maintenance of all detention facilities will be the responsibility of the property owner.*
- *Detention facilities which may impact downstream storm drainage systems may Be required to be protected by a ‘Drainage Detention Easement” recorded at the Union County Register of Deeds Office.*
- *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 Union County’s Register of Deeds ‘Office as a permanent “Drainage Detention Easement”. Also, an official commitment to maintenance of the facility will be required.*

Submittals 1.7

All development submittals requiring calculations shall include a bound report sealed by the engineer with a Table of Contents, and shall include:

- *A. A narrative which gives background on the existing/proposed site, means by which hydraulics / hydrology were performed, and any other pertinent information needed to give a better understanding of methodology; including assumptions for design.*
- *B. All necessary charts/tables/spreadsheets used in calculations*
- *C. A USGS map of the site*
- *D. Both offsite and on-site drainage area maps*
- *E. A pre/post analysis using TR-55 methodology or other volume based hydrograph*
- *F. Stormwater management and drainage design*

Chapter 2

ZONING ORDINANCE

SECTION 9-100 OF THE TOWN OF WAXHAW, N.C.
ZONING ORDINANCE
STORMWATER ORDINANCE

101. Drainage plan approval required.

(1) **No development or use of land which involved or would create more than 20,000 square feet of impervious ground cover, except for land development or used for agricultural purposes, shall be permitted without the submission and approval of a drainage plan, in accordance with the requirements of this part. No certificate of zoning compliance, certificate of occupancy, or building permit for such development shall be issued until the drainage plan is approved by the Town Engineer.**

(2) *Impervious ground cover in existence prior to October 1, 1978 of these regulations shall not be Used in measuring the 20,000 square feet identified in Subsection (1) above.*

102. Required contents of drainage plan.

The drainage plan submitted for approval under this Part shall include a site plan showing existing and proposed buildings, storm water drainage facilities and impervious ground cover; site construction plans and grading plans and drainage system; drainage facility design data, including a drainage area map, engineering calculations, area of impervious cover, and total land area; and any other appropriate information requested by the Town Engineer.

103. Standards for plan approval.

The following standards shall be met for approval of a storm water drainage plan:

(1) *The Town Engineer shall review the drainage plan for compliance with the standards contained in the Town of Waxhaw Storm Water Design Manual", Waxhaw, N.C., the current edition of the "Charlotte-Mecklenburg Land Development Standards Manual", and any other relevant and appropriate standard established by the Town Engineer.*

(2) *The Town Engineer will not approved a drainage plan if the impervious ground cover proposed in the plan would increase the peak level of storm water runoff from the site, unless the drainage plan identifies measures to control and limit runoffs to peak levels no greater than would occur from the site if left in its natural, undeveloped condition.*

(3) *All storm water collection and drainage systems shall be designed in compliance with the Charlotte-Mecklenburg Land Development Standards Manual and as stated under (1) above.*

Chapter 3

HYDROLOGY

Chapter Table of Contents

3.1 Hydrologic Design Policies

Factors Affecting Rood Runoff 3.1.1

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

Drainage Basin Characteristics

Size
Shape
Slope
Ground Cover
Land Use
Geology
Soil Types
Surface Infiltration
Ponding and Storage
Watershed Development Potential
Other Characteristics

Stream Channel Characteristics

Geometry and Configuration
Natural Controls
Artificial Controls
Channel Modifications
Aggradation – Degradation
Debris
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
Other Characteristics

Hydrologic Method 3.1.2

Many hydrologic methods are available. Recommended methods and the circumstances for their use are listed in Table 3-1. 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.

The approved methods have been selected for use in the Town of Waxhaw area based on several considerations, including:

- *Verification of their accuracy in duplicating local hydrologic 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 3-1

Recommended Hydrologic Methods

Method	*Size Limitations*[1]	*Comments*
Rational	*0-200 Acres*	*Method can be used for estimating peak flows and the design of small sub-division type storm sewer systems. For storage design, the Rational method may be used to determine the peak discharge rate up to 50 acres. See sections 7.7, 7.8, and 3.11.*
Basin Lag-Time	*> 100 Acres*	*Method can be used for estimating peak flows from urban areas.*
SCS Method (TR-55)	*0-2000 Acres*	*Method can be used for estimating peak flows from urban areas.*
SCS Step Function	*0-5 Acres*	*Method can be used for generating hydrographs and storage design for small sites. Application of the step function and exponential approximation is limited by the boundaries of the method used for estimating the peak discharge Basin Lag-Time, Rational Formula, Simplified SCS Method, etc.)*

Table 3-1 (continued)	Method	Size Limitations'	Comments
	HEC-1 U.S. Corps of Engineers Model, 1981	None	Method can be used for estimating peak flows and hydrographs. Application of HEC-1 is limited by a maximum of 300 ordinates in the hydrograph, and limitations of the particular hydrograph generation technique (SCS Unit Hydrograph, Kinematic Wave, etc.) including the minimum time step interval expressed in the HEC-1 manual. Also, see sections 3.1.5.
	Graphical Method	0-5 acres	Method can be used for sizing detention basins for small sites. Application is limited to storage facilities which can be described with a single stage-discharge and stage storage equation for sites with contributing watershed less than 5 acres. The limitations of the inflow hydrograph generation technique must be applied.

[1]Size limitations refers to the subwatershead size to the point where stormwater management facility (i.e., culvert, inlet) is located.

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

Design Frequency Policy 3.1.3

Culverts transporting storm runoff under roadways shall be designed to accommodate a 25-year flood. The peak flows and hydrographs used for culvert design shall be based on fully developed land use conditions as shown on current County and City Land Use Plans and Zoning Maps.

All detention facilities shall be designed to maintain the pre-developed runoff rate for the 2-year and the 10-year design storm events. Emergency spillway facilities shall be designed to pass the 50-year storm.

RESERVED 3.1.4

HEC-1 Limitations 3.1.5

The following are limitations of the HEC-1 model hydrograph generation routine using the SCS unit dimensions 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.

HEC -1 Limitations (continued)

- *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 SCS 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 park Of the hydrograph. However, the volume under the resulting hydrograph is Correct and all volume computation such as detention storage are correct.*

3.2 Symbols and Definitions

Symbol Table

To provide consistency within this chapter as well as throughout this manual the Following symbols will be used. These symbols were selected because of their Wide use in hydrologic publications. In some cases the same symbol is used in Existing publications for more than one definition. Where this occurs in this Chapter, the symbol will be defined where it occurs in the text or equations.

Table 3-2

SYMBOLS AND DEFINITIONS

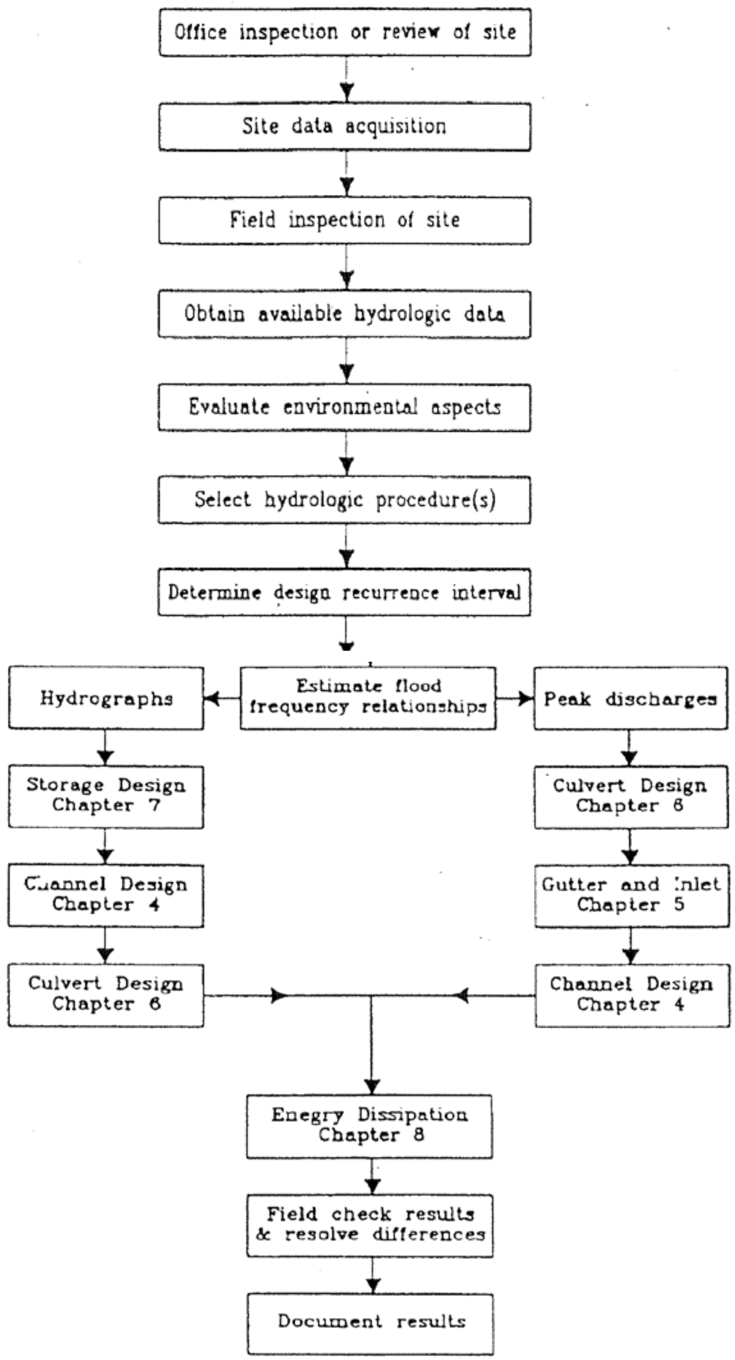
Symbols	Definition	Units
A or a	Drainage area	acres
A_f	Channel flow area	ft2
B	Channel bottom width	ft
C	Runoff Coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
D	Depth of flow	ft
d	Time interval	hours
F_p	Pond and swamp adjustment factor	
I	Runoff intensity	in./hr.
I	Percent of impervious cover	%
I_a	Initial abstraction from total rainfall	in
L	Flow length	ft
n	Manning roughness coefficient	-
P	Accumulated rainfall	in
P_w	Wetted perimeter	ft
Q	Rate of runoff	cfs
q	Storm runoff during time interval	in
q_u	Unit peak discharge	cfs
q_p	Peak rate of discharge	cfs
R or r	Hydraulic radius	ft
S or Y	Ground Slope	ft/ft or %
S	Potential maximum retention	in
S or S	Slope of hydraulic grade line	ft/ft
SCS	Soil Conservation Service	-
T	Channel top width	ft
t_c or T_c	Time of concentration	min
T_L or T	Lag time	hours
T_p	Time to peak	min
T_t	Travel time	hours
V	Velocity	ft/s

3.3 Hydrologic Analysis Procedure Flowchart

Purpose And Use
3.3.1

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.

Design Flowchart
3.3.2



3.4 Concept Definitions

	Following are discussions of concepts which will be important in a hydrologic analysis. These concepts will be used through the remainder of this chapter in dealing with different aspects of hydrologic studies.
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 – say below about the 10-year even threshold. As the frequency of a flood even increases, antecedent moisture has a rapidly decreasing influence on runoff.*
Culvert	*A structure that conveys any flow collected in a open ended pipe (i.e., headwall, flared end section, mitered end), a cross-drain.*
Depression Storage	*Depression storage is the natural depressions within a watershed which store runoff. Generally after the depression storage is filled runoff will commence.*
Frequency	*Frequency is the average time interval between equal magnitude floods. For example, a 25-year flood has the probability of occurrence of one every 25 years on the average, or a 4 percent change of occurrence in any given year.*
Hydraulic Roughness	*Hydraulic roughness is a composite of the physical characteristics which influence the flow of water across the earths 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.*
Hytetograph	*The hyetograph is a graph of the time distribution of rainfall over a watershed.*
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 water runoff begins.*

Interception	*Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.*
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.*
Rainfall Excess	*After interception, depression storage, and infiltration have been satisfied, if there is excess water available to runoff this is the rainfall excess.*
Stage	*The stage of a channel is the elevation of the water surface above some elevation datum.*
Storm Drain	*A conveyance system where flow enters through grated inlets only.*
Thoroughfare	*A street designated as a major or minor thoroughfare as shown on the Town of Waxhaw Thoroughfare Plan.*
Time of Concentration	*The time of concentration is the time required for water to flow from the most 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.*
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 the direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area.*

3.5 Design Frequency

Design Frequencies 3.5.1

Description	*Design Storm*
Storm system pipes	*10 yr.**
Ditch systems	*10 yr.**
Culverts (subdivision streets)	*25 yr.*
Culverts (thoroughfare roads)	*50 yr.*

** Except as required by section 18.10.5, Storm Drainage Not Contained in Street Rights of Way, of the UDO.*

Rainfall Intensity 3.5.2

In addition to the following rainfall intensities (Table 3-3), the most recent Atlas 14 or other acceptable sources may be used for hydrologic analysis

Table 3-3

Rainfall Intensities - Waxhaw, North Carolina

Storm Duration		Rainfall Intensity(in./hr.)						
		Return Period (Years)						
hours	minutes	2	3	5	10	25	50	100
0	5	5.03	5.60	6.30	7.03	8.21	9.00	9.92
	6	4.78	5.33	6.02	6.75	7.89	8.65	9.53
	7	4.55	5.09	5.76	6.49	7.59	8.32	9.17
	8	4.34	4.88	5.53	6.26	7.31	8.03	8.84
	9	4.16	4.68	5.32	6.04	7.06	7.75	8.54
	10	3.99	4.50	5.12	5.84	6.83	7.50	8.26
	15	3.33	3.79	4.35	5.03	5.87	6.46	7.11
	16	3.23	3.67	4.22	4.89	5.72	6.29	6.92
	17	3.13	3.57	4.10	4.77	5.57	6.13	6.74
	18	3.04	3.47	3.99	4.65	5.43	5.97	6.57
	19	2.96	3.37	3.89	4.53	5.30	5.83	6.41
	20	2.88	3.29	3.79	4.43	5.17	5.69	6.26
	21	2.80	3.20	3.70	4.32	5.05	5.56	6.12
	22	2.73	3.12	3.61	4.23	4.94	5.44	5.98
	23	2.66	3.05	3.53	4.14	4.83	5.32	5.85
	24	2.60	2.98	3.45	4.05	4.73	5.21	5.73
	25	2.54	2.91	3.37	3.96	4.63	5.10	5.61
	26	2.48	2.85	3.30	3.88	4.54	5.00	5.50
	27	2.43	2.79	3.23	3.81	4.45	4.90	5.39
	28	2.38	2.73	3.17	3.73	4.36	4.81	5.29
	29	2.33	2.68	3.11	3.66	4.28	4.72	5.19
	30	2.28	2.62	3.05	3.60	4.20	4.64	5.09
	40	1.90	2.20	2.57	3.05	3.56	3.93	4.32
	50	1.64	1.90	2.23	2.66	3.10	3.43	3.76
1		1.45	1.68	1.98	2.36	2.76	3.05	3.34
2		0.88	1.03	1.21	1.45	1.70	1.89	2.06
3		0.65	0.76	0.90	1.07	1.25	1.40	1.52
6		0.38	0.44	0.53	0.62	0.73	0.82	0.89
12		0.22	0.26	0.31	0.36	0.42	0.47	0.51
24		0.13	0.15	0.18	0.20	0.24	0.27	0.29

Taken from equation for IDF curve for Charlotte, N.C.

3.6 Rational Method

Introduction 3.6.1

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 should be designed for future land use conditions as specified in the County and City Land Use Plans.*

- *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 judgement in applying these design aids and should make appropriate adjustments when specified site characteristics dictate that these adjustments are appropriate.*

Runoff Equation 3.6.2

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

$$Q = CIA \qquad (3.1)$$

Where: *Q = maximum rate of runoff (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 location (acres)

Infrequent Storms 3.6.3

The coefficients given in Table 3-5 are applicable for storms of 2-yr to 10-yr. frequencies. Less frequent higher intensity storms will require modification of the coefficient because infiltration and other losses have proportionally smaller effect on runoff (Wright-McLaughlin 1969). The adjustment of the rational method for use with major storms can be made by multiplying the right side of the rational formula by a frequency factor C_f. The rational formula now becomes:

$$Q = CC_fIA \qquad (3.2)$$

Infrequent Storms (continued)

The C_f values that can be used are listed below in Table 3-4. The product of C_f multiplied by C shall not exceed 1.0.

Table 3-4

Frequency Factors For Rational Formula

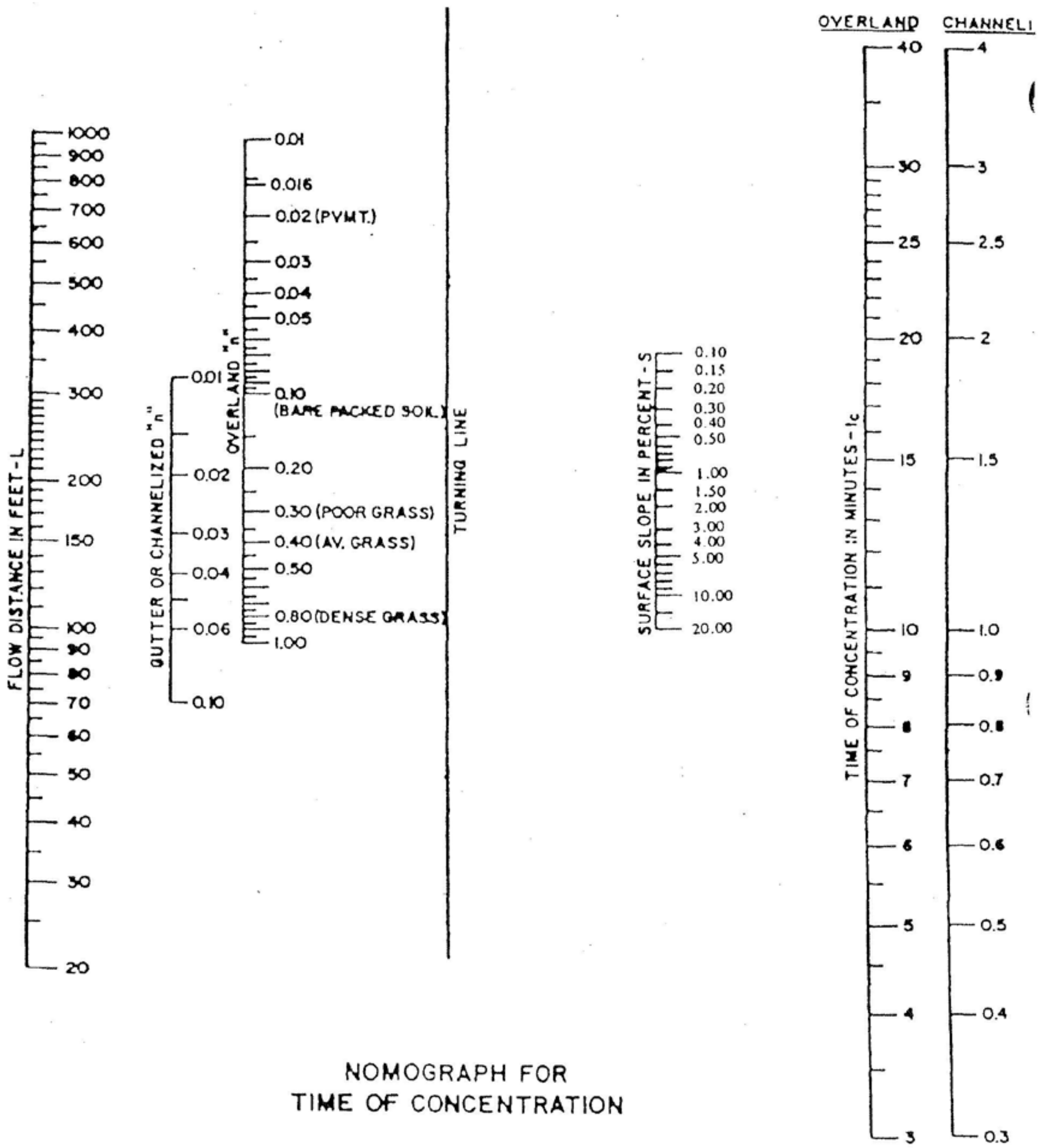
Recurrence Interval (years)	*C_f*
25	*1.1*
26	*1.2*
100	*1.25*

Time of Concentration 3.6.4

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 3-3. The time of concentration consists of an overland flow time to the point where the runoff enters a defined drainage feature (i.e., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 3-1 can be used to estimate the time of concentration. For each drainage area, distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. Other methods and charts may be used to calculate overland flow time if approved by the Town Engineer. Note: time of concentration cannot be less than 5 minutes.

Two common errors should be avoided when calculating t_c. First, 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. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 100 feet in urban areas and 300 feet in rural areas should be done only after careful consideration.



NOMOGRAPH FOR
TIME OF CONCENTRATION

Figure 3-1

Source: Austin Texas Drainage Manual

Rainfall Intensity 3.6.5

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 3-3. Straight-line interpolitian can be used to obtain rainfall intensity values for storm durations between the values given in Table 3-3.

Runoff Coefficient 3.6.6

The runoff coefficient (C) is the variable of the rational method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 3-5 below gives the recommended runoff coefficients for the Rational Method.

Table 3-5 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
Parks & cemeteries	0.30
Residential (including streets):	
Single-Family (Lot < 20,000 SF)	0.60
Single-Family (Lot > 20,000 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

Composite Coefficients 3.6.7

It is often desirable to develop a composite runoff coefficient based on the percentage of difference types of surfaces in the drainage areas. Composites can be made with the values from Table 3-5 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 or 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.

3.7 Example Problem – Rational Method

Introduction	*Following is an example problem which illustrated 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-yr and 100-yr 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:* *Average overland slope = 2.0%* *Length of overland flow = 100 ft* *Length of main basin channel = 300 ft* *Slope of channel = 2.0%* *Roughness coefficient (n) of channel was estimated to be 0.030*
Land Use	*From existing land use maps, land use for the drainage basin was estimated to be:* *Single Family (< 20,000 SF) 80%* *Park 20%*
Overland Flow	*A runoff coefficient (C) for the overland flow area, which is 100% park property, is determined from Table 3-5 to be 0.30.*
Time of Concentration	*From Figure 3-1 with an overland flow length of 100 ft, slope of 2.0 percent and n=0.35, the overland flow time is 11.3 min. Channel flow time Is also determined form Figure 3-1 to be 3.8 min (n= 0.030, S= .020, and L = 300 ft).* *Total t_c = 3.8 min + 11.3 min = 15.1 min*
Rainfall Intensity	*From Table 3-3, with a duration equal to 15.1 minutes, the intensity can be selected by interpolation* *I_{25} (25 year return period) = 5.86 in/hr* *I_{100} (100 year return period) = 7.09 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 3-5.

Land Use	*(1) Percent Of Total Land Area*	*(2) Runoff Coefficient*	*(3) Weighted Runoff Coefficient**
Residential (single family, < .5 ac.)	*.80*	*.60*	*.48*
Park	*.20*	*.30*	*.06*
Total Weighted Runoff Coefficient			*.54*

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

Peak Runoff

From the rational method equation:

$Q_{25} = C_fCIA = 1.1 \times .54 \times 5.86$ in/hr $\times$ 18 acres = 62.7 cfs

$Q_{100} = C_fCIA = 1.25 \times .54 \times 7.09$ in/hr $\times$ 18 acres = 86.1 cfs

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

3.8 Basin Lag-Time Method

Introduction 3.8.1

The Basin Lag-Time method is a mathematical regression model developed for the Piedmont of North Carolina by the U.S.G.S. The method can be used to calculate peak runoff rates for drainage areas greater than 100 acres. Using the method involved a two-step process consisting of a basin lag-time estimate and a calculation of peak discharge rates. Details of the Basin Lag-Time Method can be found in the 1972 open file report, Effect of Urban Development on Roods in the Piedmont Province of North Carolina by Arthur L. Putnam, prepared by the U.S.G.S.

Basin Lag-Time 3.8.2

Basin Lag-Time is defined as the average time interval in hours between the center of mass of the rainfall excess and the center of mass of the resultant runoff. This interval is usually shortened as hydraulic systems are improved, pipe systems enlarged and channels improved. The equation used in this method for estimating basin lag-time is:

$$\mathbf{T = 0.49\ [(L/s^{0.5})]^{0.5}\ (I/100)^{-0.57}} \qquad \mathbf{(3.3)}$$

Where: *T = Lag time in hours*
L = Length of main watercourse, in miles. As used in this expression, Length shall be equal to 75% of the maximum water travel distance.
s = Stream bed slope of the main watercourse, in feet per mile. Slope shall be determined from the difference in elevation between a point 10% from the bottom and the point 15% from the top of the maximum water travel distance.
I = The percentage of impervious cover within the design drainage basin.

Peak Discharge 3.8.3

Once the basin lag-time has been estimated, the following equations can be used for computing the peak discharge for the 2, 5, 10, 25, 50, 100, and 500 year floods.

$$Q_2 = 221\ A^{0.87} \times T^{-0.60} \qquad (3.4)$$
$$Q_5 = 405\ A^{0.80} \times T^{-0.52} \qquad (3.5)$$
$$Q_{10} = 560\ A^{0.76} \times T^{-0.48} \qquad (3.6)$$
$$Q_{25} = 790\ A^{0.71} \times T^{-0.42} \qquad (3.7)$$
$$Q_{50} = 990\ A^{0.67} \times T^{-0.37} \qquad (3.8)$$
$$Q_{100} = 1200\ A^{0.63} \times T^{-0.33} \qquad (3.9)$$
$$Q_{500} = 1800\ A^{0.57} \times T^{-0.24} \qquad (3.10)$$

Where: *Q_i = Peak discharge for the flood having the reoccurrence interval Indicated by the subscript in cfs.*
A = Drainage area in square miles
T = Lag-Time, in hours

Applicability of Method 3.8.4

This hydrologic method is dependent on two relationships. One relates the average basin lag time to the length of the main watercourse, the slope of the main watercourse, and the ratio of the area of impervious cover to the total drainage area. The other relates the flood-peak discharge to the size of drainage area and the average basin lag time. The combination of these two relationships provides a means of determining a flood-peak discharge for any location on a stream, gaged or ungaged, rural or urban. The U.S.G.S recommends using the relations for watersheds where an estimate of urban flooding is needed. The estimating relations are limited to providing flood discharge estimates at open channel site sin the Piedmont province of North Carolina where the runoff is unaffected by artificial storage or diversion. The estimates are most reliable for smaller size floods where drainage area ranges 0.3 and 150 square miles, where the L/(s)0.5 ratio ranges between 0.1 and 9.0 and where impervious cover of less than 30 percent is uniformly distributed over the basin.

Example Problem 3.8.5

Estimate the 10- and 100-year peak discharges from a drainage area having the following characteristics.

- *Drainage area = 735 acres = 1.15 mi^2*
- *Length of main watercourse = 10,400 feet = 1.97 miles*
- *Stream slope = 3.5% = 185 ft/mi*
- *Impervious cover = Residential-Low Density = 15%*

Using equation 3.3, $T = 0.49\ [1.97/(185)^{0.5}]^{0.5}\ (.15)^{-0.57} = 0.55$ *hrs.*

Using equation 3.6, $Q_{10} = 560\ (1.15^{\ 0.76})\ (0.55^{\ -0.48}) = 830$ *cfs*

Using equation 3.9, $Q_{100} = 1200\ (1.15^{0.63})\ (0.55^{\ -0.33}) = 1{,}600$ *cfs*

Table of Impervious Areas 3.8.6

Table 3-6

Impervious Areas for Basin Lag-Time[1]

Type of Development	*Sampled % Impervious Cover*
Urban core (central business district)	*88.0*
Arterial commercial (commercial strip development)	*68.8*
Office-regional shopping	*48.9*
Urban node (neighborhood shopping areas)	*47.1*
Industrial-high density (heavy intercity industry)	*56.5*
Industrial-medium density (light industry)	*44.0*
Industrial-low density (modern industrial parks)	*23.8*
Residential-high density (concentrated mulit-family –units)	*34.0*
Residential-medium density (density of 3-6 units per acre)	*21.6*
Residential-low density (density of 1-2 units per acre)	*15.0*
Exurban (density of less than 1 unit per acre)	*1.3*

[1]*Taken from study by Charlotte-Mecklenburg Planning Commission of impervious areas within Mecklenburg County.*

3.9 SCS Unit Hydrograph

Introduction 3.9.1

The Soil Conservation Service (SCS) hydrologic method requires basic data similar to the Rational Method; drainage area, a runoff factor, time of concentration, and rainfall. The SCS 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 a infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4.

The SCS method includes the following basic steps:

1. *Determination of curve numbers which represent different land uses within the drainage area.*

2. *Calculation of time of concentration to the study point.*

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

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

Unit Hydrograph 3.9.2

Two types of hydrographs are used in the SCS procedure, dimensionless hydrographs and composite hydrographs. A unit hydrograph (Fig. 3-2) represents the time distribution of flow resulting from one inch of direct runoff occurring over the watershed in a specified time. Unit hydrograph time and discharge ratios are shown in Table 3-7. The SCS method uses the unit hydrograph to estimate the triangular hydrograph from small durations of the total design storm. The triangular hydrographs are then combined into a composite hydrograph for the drainage area.

Characteristics of the dimensionless hydrograph vary with the size, shape, and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a given rainfall. Basin lag in this method is defined as the time from the center of mass of rainfall excess to the hydrograph peak. The following equation is used to determine basin lag time:

$$\mathbf{T_L = 0.6\ T_C} \qquad \mathbf{(3.11)}$$

The hydrograph computation interval must be less than lag time multiplied by 0.29.

Equations and Concepts 3.9.3

The following discussion outlines the equation and basic concepts utilized in the SCS 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 points of interest.

Equations and Concepts (continued)

Rainfall- The SCS method applicable to the Town of Waxhaw area is based on a storm which has a Type II time distribution. Figure 3-3 shows this distribution. To use this distribution it is necessary for the user to obtain the 24-hour rainfall volume (P24 in Figure 3-3) from Table 3-3 for the frequency of the design storm. This volume is then distributed according to Figure 3-3. Rainfall may also be distributed using a 'balanced" distribution. Tabls 3-8 and 3-9 show a balanced distribution for the 2-year 6-hour and 10-year 6 hour storms.

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

$$Q = (P - I_a)^2 / (P - I_a) + S \qquad (3.12)$$

Where: Q = *accumulated direct runoff*

P = *accumulated rainfall (potential maximum runoff)*

I_a = *initial abstraction including surface storage, interception, and Infiltration prior to runoff.*

S = *potential maximum soil retention*

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

$$I_a = 0.2S \qquad (3.13)$$

Substituting 0.2S for I_a in equation 3.12, the SCS rainfall-runoff equation becomes:

$$Q = (P – 0.2S)^2 / (p + 0.8S) \qquad (3.14)$$

Where: S = *1000/CN – 10*

CN = *SCS curve number*

Figure 3-4 on page 3-24 shows a graphical solution of this equation which enables the precipitation excess from a storm to be obtained it 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.

Time Ratios (t/T_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Qa/Q)
0.0	0.000	0.000
0.1	0.030	0.001
0.2	0.100	0.006
0.3	0.190	0.017
0.4	0.310	0.035
0.5	0.470	0.065
0.6	0.660	0.107
0.7	0.820	0.163
0.8	0.930	0.228
0.9	0.990	0.300
1.0	1.000	0.375
1.1	0.990	0.450
1.2	0.930	0.522
1.3	0.860	0.589
1.4	0.780	0.650
1.5	0.680	0.705
1.6	0.560	0.751
1.7	0.460	0.790
1.8	0.390	0.822
1.9	0.330	0.849
2.0	0.280	0.871
2.2	0.207	0.908
2.4	0.147	0.934
2.6	0.107	0.953
2.8	0.077	0.967
3.0	0.055	0.977
3.2	0.040	0.984
3.4	0.029	0.989
3.6	0.021	0.993
3.8	0.015	0.995
4.0	0.011	0.997
4.5	0.005	0.999
5.0	0.000	1.000

Ratios for Dimensionless Unit Hydrograph and Mass Curve

Table 3-7

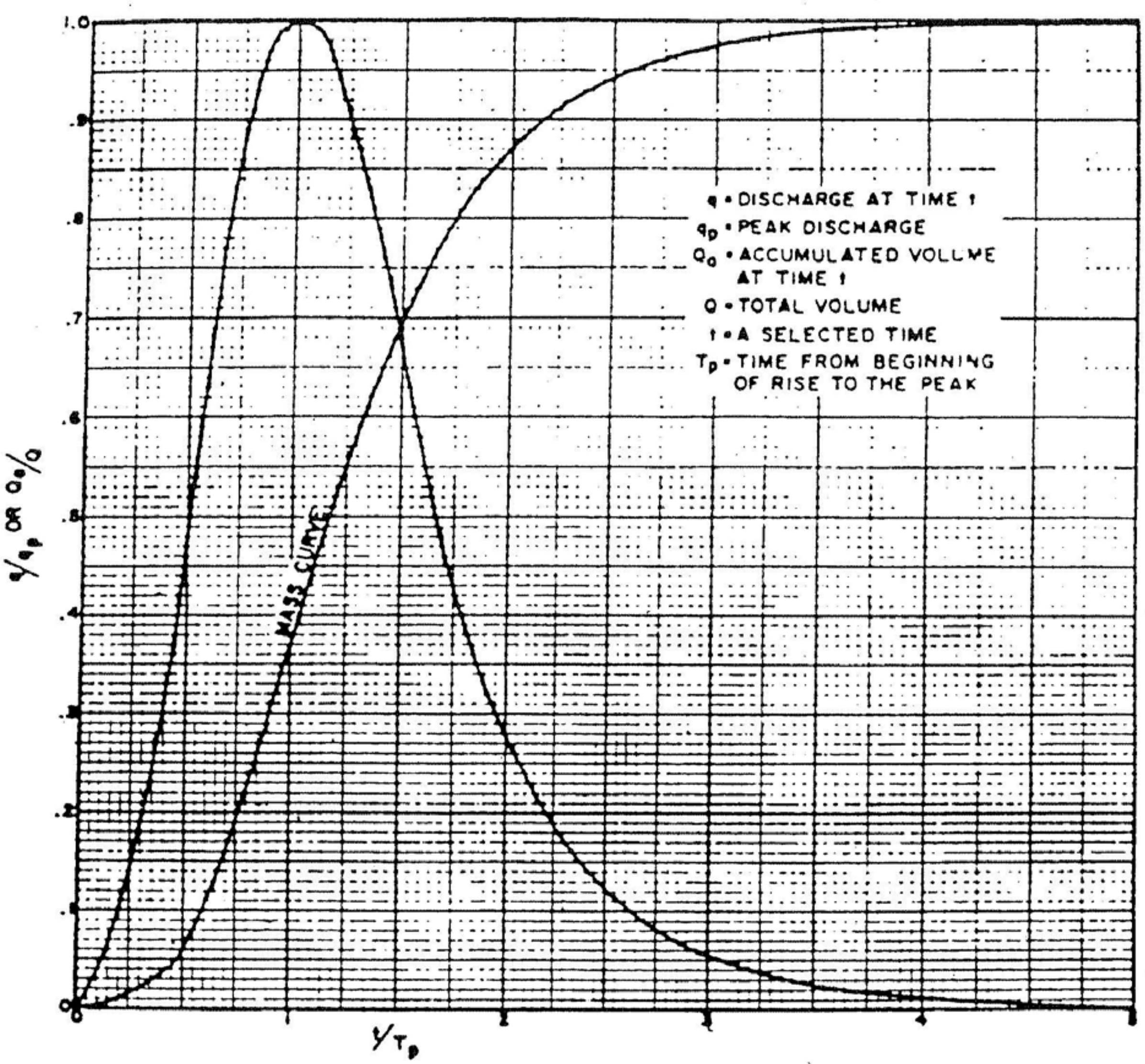
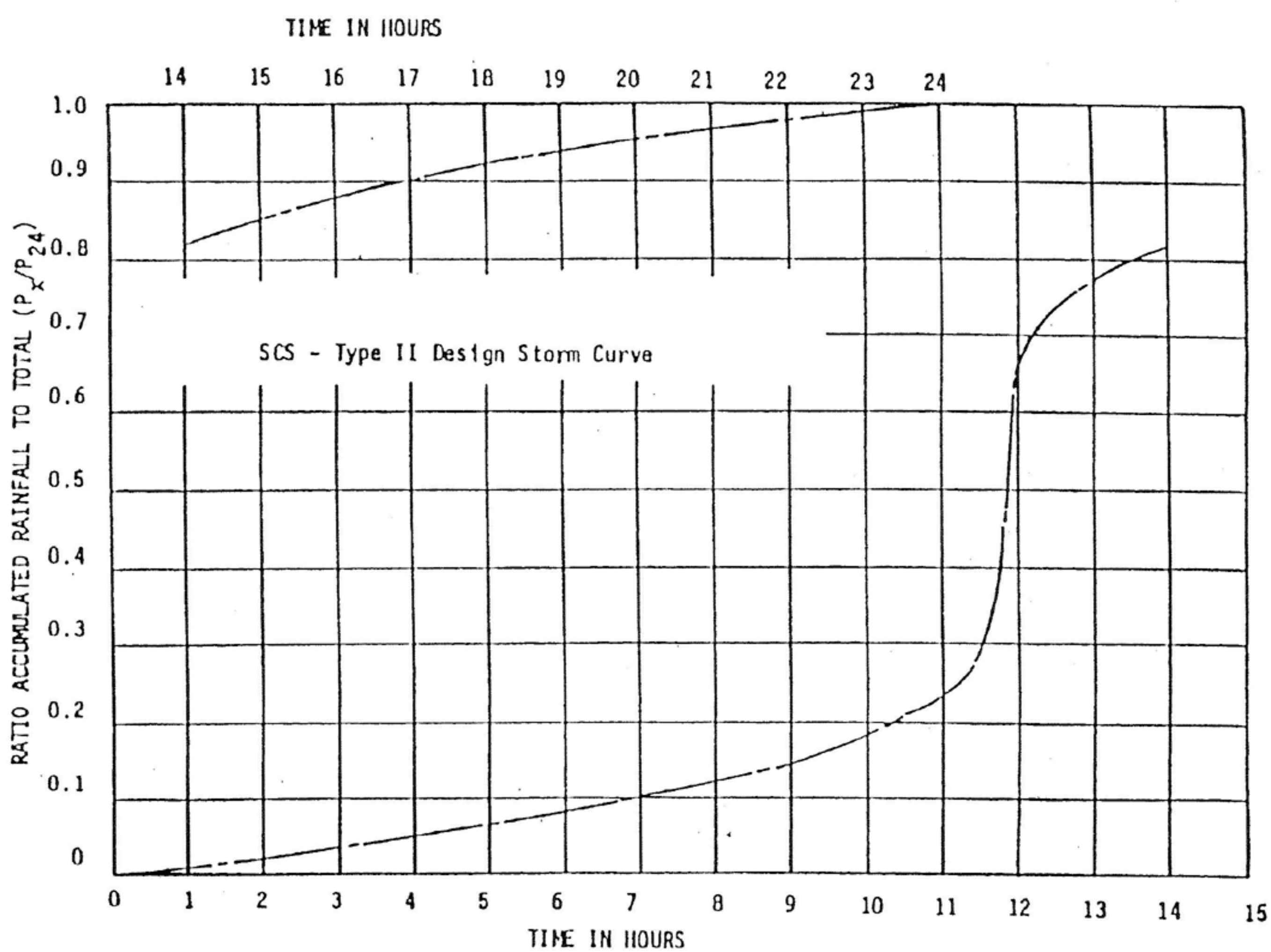
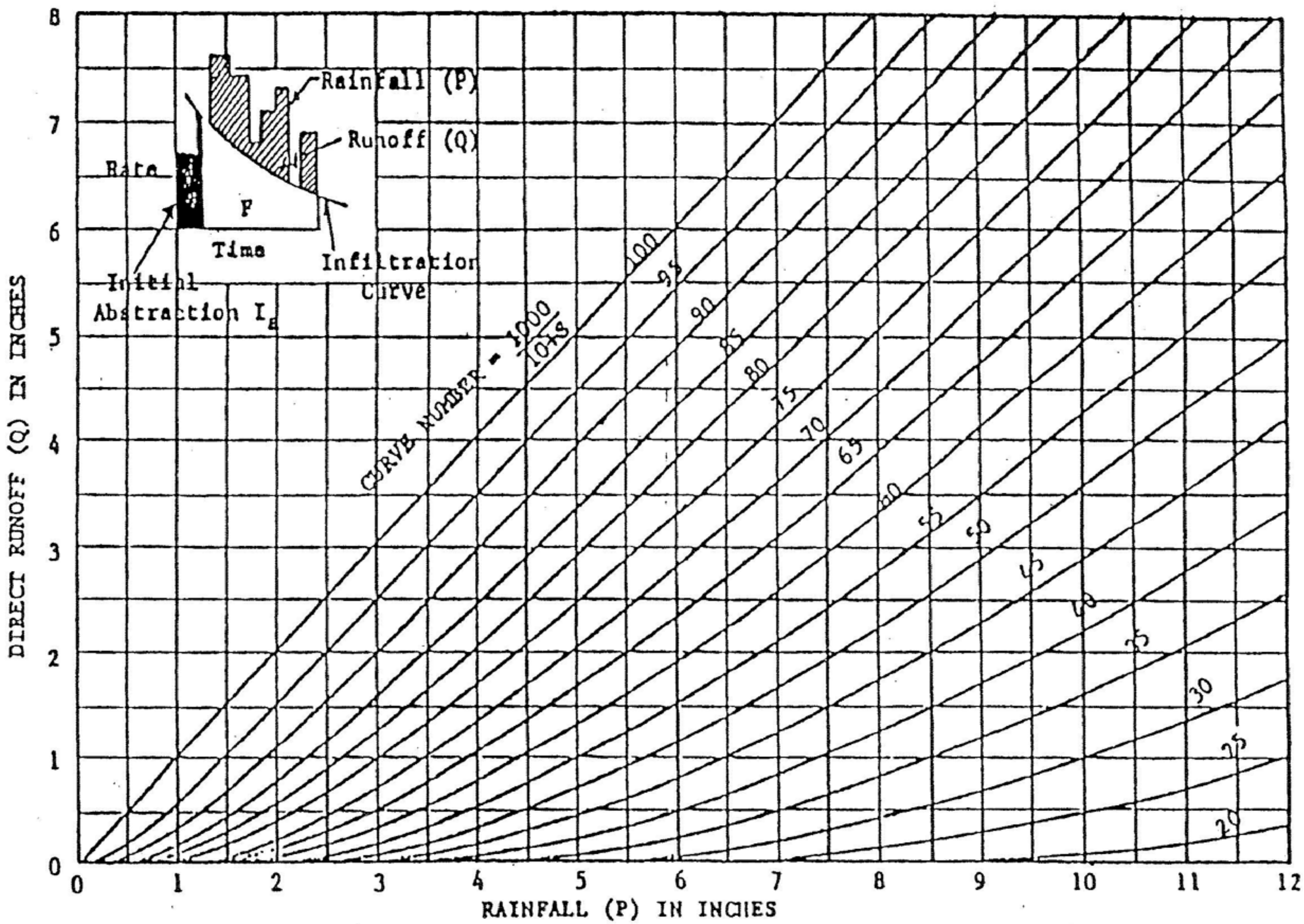


Figure 3-2
Dimensionless Unit Hydrograph and Mass Curve



Source: SCS-TP-149

SCS Type II Design Storm Curve
Figure 3 – 3



Solution of the Runoff Equation, $Q = \frac{(P - 0.2S)^2}{P + 0.8S}$ Where $I_a = 0.2S$

Source: Soil Conservation Service

SCS Solution of the Runoff Equation
Figure 3 – 4

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	2.23

2-YEAR, 6-HOUR STORM 5-MINUTE TIME INCREMENT											
Time (min)	Rain (in)	Time (min)	Rain (in)	Time (min)	Rain (in)	Time (min)	Rain (in)	Time (min)	Rain (in)	Time (min)	Rain (in)
000	.000	065	.010	125	.021	185	.420	245	.018	305	.010
005	.007	070	.010	130	.022	190	.168	250	.017	310	.009
010	.007	075	.011	135	.024	195	.100	255	.016	315	.009
015	.008	080	.011	140	.026	200	.064	260	.016	320	.009
020	.008	085	.011	145	.028	205	.054	265	.015	325	.009
025	.008	090	.012	150	.031	210	.047	270	.014	330	.008
030	.008	095	.014	155	.044	215	.033	275	.012	335	.008
035	.008	100	.014	160	.050	200	.029	280	.012	340	.008
040	.009	105	.015	165	.058	225	.027	285	.011	345	.008
045	.009	110	.016	170	.089	230	.025	290	.011	350	.008
050	.009	115	.017	175	.115	235	.023	295	.010	355	.007
055	.009	120	.018	180	.242	240	.021	300	.010	360	.007
060	.010									365	.000

Table 3-8

2-Year 6-Hour Balanced Storm Distribution

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 5-MINUTE TIME INCREMENT											
Time (min)	Rain (in)	Time (min)	Rain (in)	Time (min)	Rain (in)	Time (min)	Rain (in)	Time (min)	Rain (in)	Time (min)	Rain (in)
000	.000	065	.015	125	.036	185	.590	245	.030	305	.015
005	.011	070	.016	130	.039	190	.275	250	.028	310	.015
010	.011	075	.016	135	.042	195	.177	255	.027	315	.014
015	.012	080	.017	140	.045	200	.112	260	.025	320	.014
020	.012	085	.018	145	.049	205	.096	265	.024	325	.013
025	.012	090	.018	150	.054	210	.084	270	.023	330	.013
030	.012	095	.023	155	.079	215	.057	275	.019	335	.013
035	.013	100	.024	160	.089	220	.051	280	.018	340	.012
040	.013	105	.025	165	.103	225	.047	285	.017	345	.012
045	.013	110	.026	170	.161	230	.043	290	.017	350	.012
050	.014	115	.027	175	.201	235	.040	295	.016	355	.011
055	.014	120	.029	180	.395	240	.038	300	.016	360	.011
060	.015									365	.000

Table 3-9

10-Year 6-Hour Balanced Storm Distribution

Runoff Factor
3.9.4

The principle physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types and land slope. The SCS 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 SCS 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 or 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 surface, and shallow soils over nearly impervious parent material.

A list of soils for the Town of Waxhaw and their hydrologic classification is presented in Table 3-10 below. Soil Survey maps can be obtained from local SCS offices.

Table 3-10 Hydrologic Soil Groups for Waxhaw

Series Name	*Hydrologic Group*	*Series Name*	*Hydrologic Group*
Tatum	*B*	*Badin*	*B*
Cecil	*B*	*Chewacla*	*C*
Goldston	*C*	*Secrest*	*C*
Pacolet	*B*		

Runoff Factor (continued)	*Consideration should be given to the effects of urbanization 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 3-11 on the next page gives recommended curve number values for a range of Different land uses, and soil types.*
Urban Modifications 3.9.5	*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, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?* *The curve number values given in Table 3-11 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 urban 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 B at the end of this chapter.*

Table 3-11

Runoff Curve Numbers[1]

Cover Description		*Crurve Numbers for Hydrologic Soil Groups*			
Cover Type and Hydrologic Condition	*Average Percent Impervious Area*[2]	*A*	*B*	*C*	*D*
<u>Fully Developed Urban Areas (vegetation established)</u>					
Open space (lawns, parks, 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.		*98*	*98*	*98*	*98*
Paved streets and roads		*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 district 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*
<u>Agricultural Lands</u>					
Pasture, grassland, or range (continuous forage for grazing)[4]					
Poor hydrologic condition		*68*	*79*	*86*	*89*
Fair hydrologic condition		*49*	*69*	*79*	*84*
Good hydrologic condition		*39*	*61*	*74*	*80*
Meadow (continuous grass, protected from grazing and generally Mowed for hay)		*30*	*58*	*71*	*78*
Woods[5]					
Poor hydrologic condition		*45*	*66*	*77*	*83*
Fair hydrologic condition		*36*	*60*	*73*	*79*
Good hydrologic condition		*30*	*55*	*70*	*77*
<u>Developing Urban Areas</u>					
Newly graded areas (pervious areas only, no vegetation)		*77*	*86*	*91*	*94*

Table 3-11 (continued)

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

[2]The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

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

[4]Poor: <50% ground cover or heavily grazed with not mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

[5]Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest letter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Source: Soil Conservation Service Technical Release No. 55.

Travel Time Estimation 3.9.6

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.

Travel Time 3.9.6.1

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:

$$\mathbf{T_t = L / (3600V)} \qquad \mathbf{(3.15)}$$

Where: T_t *= travel time (hr),*
L = flow length (ft),
V = average velocity (fts), and
3600 = conversion factor from seconds to hours.

Time of Concentrations 3.9.6.2

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.

$$\mathbf{T_c = T_{t1} + T_{t2} \ldots T_n} \qquad \mathbf{(3.16)}$$

Where: *Tc = time of concentration (hr) and*
n = number of flow segments.

Sheet Flow 3.9.6.3

Sheet flow is flow over plane surfaces. It usually 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.

For sheet flow less than 300 feet in rural areas and less than 100 ft. in urban areas (Overton and Meadows 1976) use the following formula to compute T_t:

$$\mathbf{T_t = [0.007\,(nL)^{0.8} / (P_2)^{0.5}(S)^{0.4}]} \qquad \textbf{(3.17)}$$

Where: T_t = travel time (hr),
n = Manning roughness coefficient
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:

$$\mathbf{Tt = [0\text{-}238\,(nL)^{0.8}] / (S)^{0.4}]} \qquad \textbf{(3.18)}$$

Thus the final equations for paved and unpaved areas are:

Paved

$$\mathbf{T_t = 0.0065[(L)^{0.8} / (S)^{0.4}]} \qquad \textbf{(3.19)}$$
$$\mathbf{V = 2.56(S)^{0.4}(L)^{0.2}} \qquad \textbf{(3.20)}$$

Unpaved

$$\mathbf{T_t = 0.076\,[(L)^{0.8} / (S)^{0.4}]} \qquad \textbf{(3.21)}$$
$$\mathbf{V = 0.22(S)^{0.4}(L)^{0.2}} \qquad \textbf{(3.22)}$$

Where **V = velocity (fps)**

Table 3-12

Roughness Coefficients (Manning's n)[1] for Sheet Flow

Surface Description	*n*
Smooth surfaces (concrete, asphalt Gravel, or bare soil)	*0.011*
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)	
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: SCS, TR-55, Section Edition, June 1986.

Shallow Concentrated Flow 3.9.6.4

After a maximum of 300 feet in rural areas or 100 feet in urban areas, sheet flow usually becomes shallow concentrated flow. The average velocity for this can be determined from Figure 3-5 on the next page, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be Computed from using Figure 3-5, or the following equations. These equations can also be Used for slopes less then 0.005 ft/ft.

Unpaved $\mathbf{V = 16.1345(S)^{1/2}}$ **(3.23)**

Paved $\mathbf{V = 20.3282(S)^{1/2}}$ **(3.24)**

Where: *V = average velocity (ft/s), and*
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 Figure 3-5 or equations 3.23 or 3.24, use Equation 3.15 to estimate travel time for the shallow concentrated flow segment.

Open Channels 3.9.6.5

Open channels are 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. 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 equation is $\mathbf{V = [1.49(r)^{2/3}(s)^{1/2}]/n}$ **(3.25)**

Where: *V = average velocity fts),*
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 and
n = Manning's roughness coefficient for open channel flow.

After average velocity is computed using equation 3.25, Tt for the channel segment can be estimated using equation 3.15.

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

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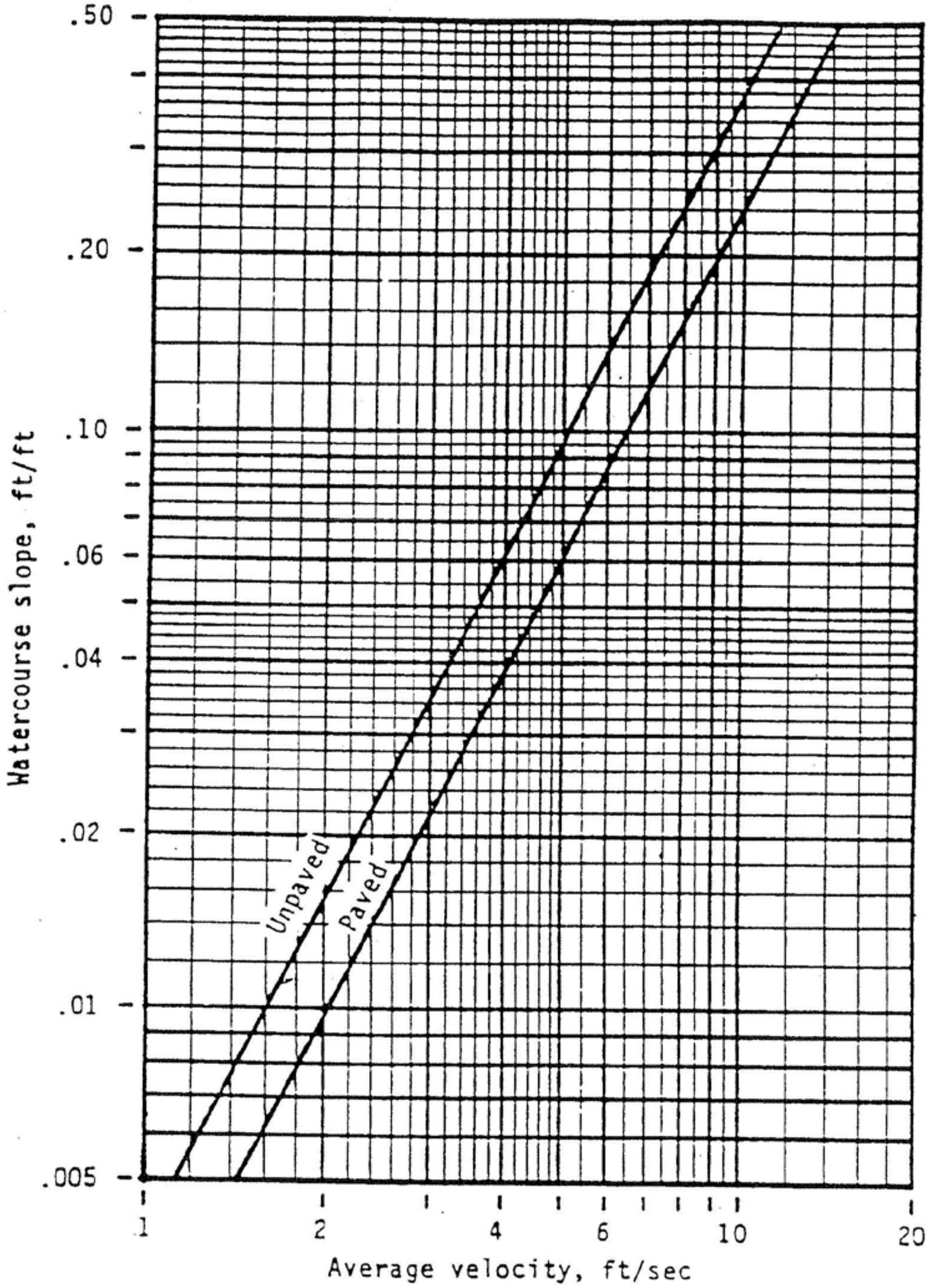


Source: USDA, SCS, TR55(1986)

Figure 3-5

Average Velocities- Shallow Concentrated Flow

Reservoirs And Lakes 3.9-6.6

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.

Limitations 3.9.6.7

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

- *In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c.*

- *A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert.*

Example Problem 3.9.7

Compute a 2-year, 6-hour hydrograph given the following information:

Basin area = 0.029 square miles
CN = 75.0
T_c = 0.75 hours
T_L = 0.45 hours

Computations

A spreadsheet format is used to organize and compute the hydrograph data (Table 3-13). The spreadsheet columns are organized as follows:

Column A: Computation ordinate number. The number of ordinates must be set to ensure coverage of the entire expected hydrograph duration which typically is twice the rainfall Duration. In the case of the example, a total of 144 ordinates (5 minute computational interval) is selected in order to provide a computation that generates 12 hours of runoff hydrograph.

Column B: Computation interval. The interval is selected for accuracy purposes. A smaller computation interval will allow a greater degree of accuracy, but will require greater calculation effort. Typical computation interval is 5 minutes, but consideration should be given to 1 or 3 minute(s) for watersheds smaller than 5 acres (lag time multiplied by 0.25 must be larger than the computation interval).

Column C: Incremental precipitation. The appropriate incremental precipitation for each computation interval is shown in column C. Any storm duration or distribution may be entered including an SCS Type II distribution with a 24-hour duration. The example uses the 2-year, 6-hour balanced storm distribution from Table 3-8.

Column D: Accumulated precipitation. Precipitation is accumulated from column C; for example, D4 = C4 + C3 + C2 + C1.

Column E: Maximum retention. Column E represents the maximum retention after runoff begins (documented in the publication Urban Hydrology for Small Watersheds, TR-55). This parameter is constant for all computation intervals and is calculated as:

$$\mathbf{S = (1000/CN) - 10}$$

Column F: Initial Abstraction. This value is assumed to be equal to 0.2S.

Column G: Excess Precipitation. Excess precipitation is the precipitation that is not lost to the initial abstractions. This is computed by subtracting column F from column D until the initial abstractions have been filled.

umn A	Column B	Column C	Column D	Column E	Column F	Column G	Column H	Column I	Column J	Column K
1	0	0.000	0.000	3.3333	0.6667	0.000	0.0000	0.0000	1.9983	0.0000
2	5	0.007	0.007	3.3333	0.6667	0.000	0.0000	0.0000	6.9942	0.0000
	10	0.007	0.014	3.3333	0.6667	0.000	0.0000	0.0000	13.9884	0.0000
4	15	0.008	0.022	3.3333	0.6667	0.000	0.0000	0.0000	21.9818	0.0000
5	20	0.008	0.030	3.3333	0.6667	0.000	0.0000	0.0000	27.0062	0.0000
6	25	0.008	0.038	3.3333	0.6667	0.000	0.0000	0.0000	27.9768	0.0000
7	30	0.008	0.046	3.3333	0.6667	0.000	0.0000	0.0000	27.0062	0.0000
8	35	0.009	0.055	3.3333	0.6667	0.000	0.0000	0.0000	23.0095	0.0000
9	40	0.009	0.064	3.3333	0.6667	0.000	0.0000	0.0000	19.0128	0.0000
10	45	0.009	0.073	3.3333	0.6667	0.000	0.0000	0.0000	12.9892	0.0000
11	50	0.009	0.082	3.3333	0.6667	0.000	0.0000	0.0000	9.9917	0.0000
2	55	0.009	0.091	3.3333	0.6667	0.000	0.0000	0.0000	7.9934	0.0000
3	60	0.010	0.101	3.3333	0.6667	0.000	0.0000	0.0000	5.9950	0.0000
4	65	0.010	0.111	3.3333	0.6667	0.000	0.0000	0.0000	3.9967	0.0000
5	70	0.010	0.121	3.3333	0.6667	0.000	0.0000	0.0000	2.9975	0.0000
6	75	0.011	0.132	3.3333	0.6667	0.000	0.0000	0.0000	2.9975	0.0000
7	80	0.011	0.143	3.3333	0.6667	0.000	0.0000	0.0000	1.9983	0.0000
8	85	0.011	0.154	3.3333	0.6667	0.000	0.0000	0.0000	0.9992	0.0000
9	90	0.012	0.166	3.3333	0.6667	0.000	0.0000	0.0000	0.9992	0.0000
0	95	0.014	0.180	3.3333	0.6667	0.000	0.0000	0.0000	0.9992	0.0000
1	100	0.014	0.194	3.3333	0.6667	0.000	0.0000	0.0000	0.9992	0.0000
2	105	0.015	0.209	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
3	110	0.016	0.225	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
4	115	0.017	0.242	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
5	120	0.018	0.260	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
[illegible]	125	0.021	0.281	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
7	130	0.022	0.303	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
3	135	0.024	0.327	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
[illegible]	140	0.026	0.353	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
[illegible]	145	0.028	0.381	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
[illegible]	150	0.031	0.412	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
[illegible]	155	0.044	0.456	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
[illegible]	160	0.050	0.506	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
[illegible]	165	0.058	0.564	3.3333	0.6667	0.000	0.0000	0.0000	0.0000	0.0000
[illegible]	170	0.089	0.653	3.3333	0.6667	0.000	0.0000	0.0030	0.0000	0.0060
[illegible]	175	0.115	0.768	3.3333	0.6667	0.768	0.0030	0.0291	0.0000	0.0790
[illegible]	180	0.242	1.010	3.3333	0.6667	1.010	0.0321	0.1102	0.0000	0.4653
[illegible]	185	0.420	1.430	3.3333	0.6667	1.430	0.1422	0.0616	0.0000	1.3652
[illegible]	190	0.168	1.598	3.3333	0.6667	1.598	0.2034	0.0403	0.0000	2.7692
[illegible]	195	0.100	1.698	3.3333	0.6667	1.698	0.2437	0.0272	0.0000	4.4823
	200	0.064	1.762	3.3333	0.6667	1.762	0.2709	0.0238	0.0000	6.0154
	205	0.054	1.816	3.3333	0.6667	1.816	0.2977	0.0213	0.0000	7.0632
	210	0.047	1.863	3.3333	0.6667	1.863	0.3160	0.0153	0.0000	7.6106
	215	0.033	1.896	3.3333	0.6667	1.896	0.3312	0.0136	0.0000	7.5450
	220	0.029	1.925	3.3333	0.6667	1.925	0.3448	0.0129	0.0000	7.2035
	225	0.027	1.952	3.3333	0.6667	1.952	0.3577	0.0120	0.0000	6.4503

Table 3 – 13
Hydrograph Spreadsheet

Column A	Column B	Column C	Column D	Column E	Column F	Column G	Column H	Column I	Column J	Column K
47	230	0.025	1.977	3.3333	0.6667	1.977	0.3697	0.0112	0.0000	5.7734
48	235	0.023	2.000	3.3333	0.6667	2.000	0.3809	0.0103	0.0000	5.1855
49	240	0.021	2.021	3.3333	0.6667	2.021	0.3913	0.0089	0.0000	4.6244
50	245	0.018	2.039	3.3333	0.6667	2.039	0.4032	0.0085	0.0000	4.0849
51	250	0.017	2.056	3.3333	0.6667	2.056	0.4087	0.0081	0.0000	3.6590
52	255	0.016	2.072	3.3333	0.6667	2.072	0.4168	0.0081	0.0000	3.3603
53	260	0.016	2.088	3.3333	0.6667	2.088	0.4249	0.0077	0.0000	3.0333
54	265	0.015	2.103	3.3333	0.6667	2.103	0.4329	0.0072	0.0000	2.7182
55	270	0.014	2.117	3.3333	0.6667	2.117	0.4397	0.0062	0.0000	2.5023
56	275	0.012	2.129	3.3333	0.6667	2.129	0.4459	0.0062	0.0000	2.3416
57	280	0.012	2.141	3.3333	0.6667	2.141	0.4521	0.0057	0.0000	2.1856
58	285	0.011	2.152	3.3333	0.6667	2.152	0.4578	0.0057	0.0000	1.9624
59	290	0.011	2.163	3.3333	0.6667	2.163	0.4636	0.0052	0.0000	1.8018
60	295	0.010	2.173	3.3333	0.6667	2.173	0.4688	0.0053	0.0000	1.6757
61	300	0.010	2.183	3.3333	0.6667	2.183	0.4741	0.0053	0.0000	1.5714
62	305	0.010	2.193	3.3333	0.6667	2.193	0.4794	0.0048	0.0000	1.4808
63	310	0.009	2.202	3.3333	0.6667	2.202	0.4842	0.0048	0.0000	1.4012
64	315	0.009	2.211	3.3333	0.6667	2.211	0.4890	0.0048	0.0000	1.3369
65	320	0.009	2.220	3.3333	0.6667	2.220	0.4938	0.0048	0.0000	1.2802
66	325	0.009	2.229	3.3333	0.6667	2.229	0.4986	0.0043	0.0000	1.2313
67	330	0.008	2.237	3.3333	0.6667	2.237	0.5029	0.0043	0.0000	1.1879
68	335	0.008	2.245	3.3333	0.6667	2.245	0.5072	0.0043	0.0000	1.1490
69	340	0.008	2.253	3.3333	0.6667	2.253	0.5115	0.0043	0.0000	1.1127
70	345	0.008	2.261	3.3333	0.6667	2.261	0.5153	0.0043	0.0000	1.0802
71	350	0.008	2.269	3.3333	0.6667	2.269	0.5202	0.0038	0.0000	1.0525
72	355	0.007	2.276	3.3333	0.6667	2.276	0.5240	0.0038	0.0000	1.0265
73	360	0.007	2.283	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.9931
74	365	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.9403
75	370	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.8630
76	375	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.7580
77	380	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.6350
78	385	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.5123
79	390	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.3967
80	395	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.3004
81	400	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.2208
82	405	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.1656
83	410	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.1240
84	415	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0910
85	420	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0661
86	425	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0488
87	430	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0364
88	435	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0244
89	440	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0163
90	445	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0120
91	450	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0076
92	455	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0038
93	460	0.000	2.592	3.3333	0.6667	2.283	0.5278	0.0000	0.0000	0.0000

Table 3 – 13
Hydrograph Spreadsheet (continued)

Example Problem (continued)

Column H: Accumulated Runoff. Column H: is the accumulated runoff resulting from the excess precipitation. Each ordinate is computed for each excess precipitation by equation 3.12 as follows:

$$Q = ((P - I_a)^2)/((P - Ia) + S)$$

Where: Q = runoff (in) – Column H:
P = rainfall (in) – Column G,
S = maximum retention (in) – Column
Ia = initial abstraction (in) – Column F.

Column I: Incremental Runoff. Column I represents the incremental runoff of the storm event which is calculated by determining the differences between each cell of column H: beginning with the bottom cell of the range, i.e. I174 = H74 – H73, I73 = H73 – H72, etc.

Column J: Unit Hydrograph. Column J is the ordinate values of the unit hydrograph developed from basin characteristics. The parameters T_p *and* q_p *used to develop the hydrograph are defined as follows:*

$$T_p = (D/2) + T_L$$

Where: T_p = time to peak (hours)
D = computation interval (hours)
T_L = lag time of the watershed (hours)

$$q_p = (484*A)/T_p$$

Where: q_p = peak discharge (cfs)
A = watershed area (square miles)
T_p = time to peak

Values for T_p *and* q_p *are calculated to find the unit hydrograph time and discharge values as shown in Table 3-14. The time and discharge values are then used to plot the unit hydrograph (Figure 3-6). Discharge values corresponding to the computational interval (5, 10, 15 …etc.) are entered in column J.*

Column K: Resulting hydrograph. Column K represents the resulting hydrograph computed by inverting the incremental runoff of column I and multiplying it with the unit hydrograph of column J as shown below:

$K1 = I1 * J1$
$K2 = (I1 * J2) + (I2 * J1)$
$K3 = (I1 * J3) + (I2 * J2) + (I3 * J1)$
$K4 = (I4 * J4) + (I2 * J3) + (I3 * J2) + (I4 * J1)$
.
.
.

The resulting hydrograph is shown in Figure 3-7.

Table 3-14
Unit Hydrograph Time and Discharge Values

Time Ratios	Time (col. 1x0.49x60) (minutes)	Discharge Ratios	Discharge (col. 3x28.64) (cfs)
0.00	0.00	0.000	0.00
0.10	2.94	0.030	0.86
0.20	5.88	0.100	2.86
0.30	8.82	0.190	5.44
0.40	11.76	0.310	8.88
0.50	14.70	0.470	13.46
0.60	17.64	0.660	18.90
0.70	20.58	0.820	23.48
0.80	23.52	0.930	26.64
0.90	26.46	0.990	28.35
1.00	29.40	1.000	28.64
1.10	32.34	0.990	28.35
1.20	35.28	0.930	26.64
1.30	38.22	0.860	24.63
1.40	41.16	0.780	22.34
1.50	44.10	0.680	19.48
1.60	47.04	0.560	16.04
1.70	49.98	0.460	13.17
1.80	52.92	0.390	11.17
1.90	55.86	0.330	9.45
2.00	58.80	0.280	8.02
2.20	64.68	0.207	5.93
2.40	70.56	0.147	4.21
2.60	76.44	0.107	3.06
2.80	82.32	0.077	2.21
3.00	88.20	0.055	1.58
3.20	94.08	0.040	1.15
3.40	99.96	0.029	0.83
3.60	105.84	0.021	0.60
3.80	111.72	0.015	0.04
4.00	117.60	0.011	0.32
4.50	132.30	0.005	0.14
5.00	147.00	0.000	0.00

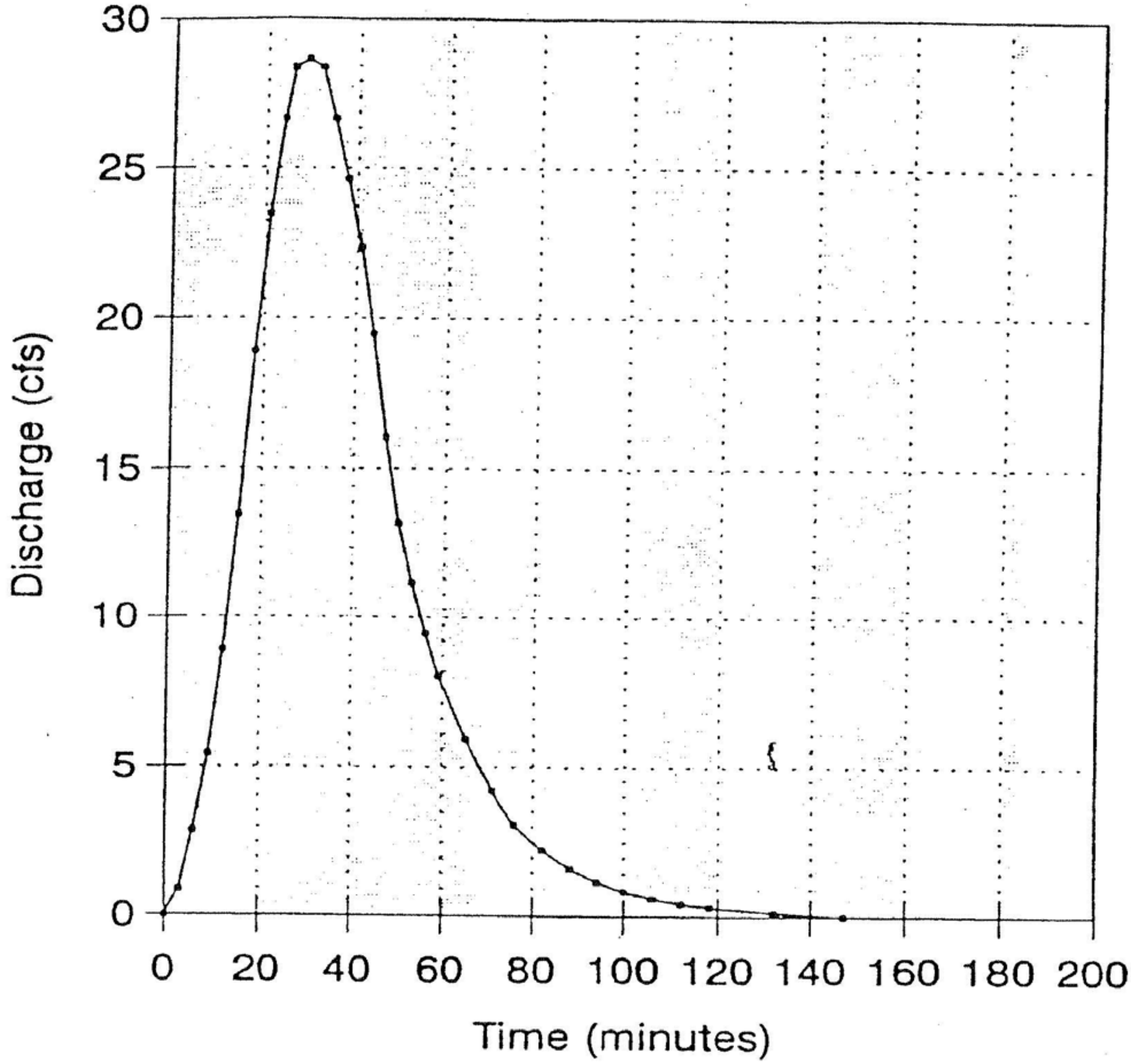


Figure 3-6

Unit Hydrograph

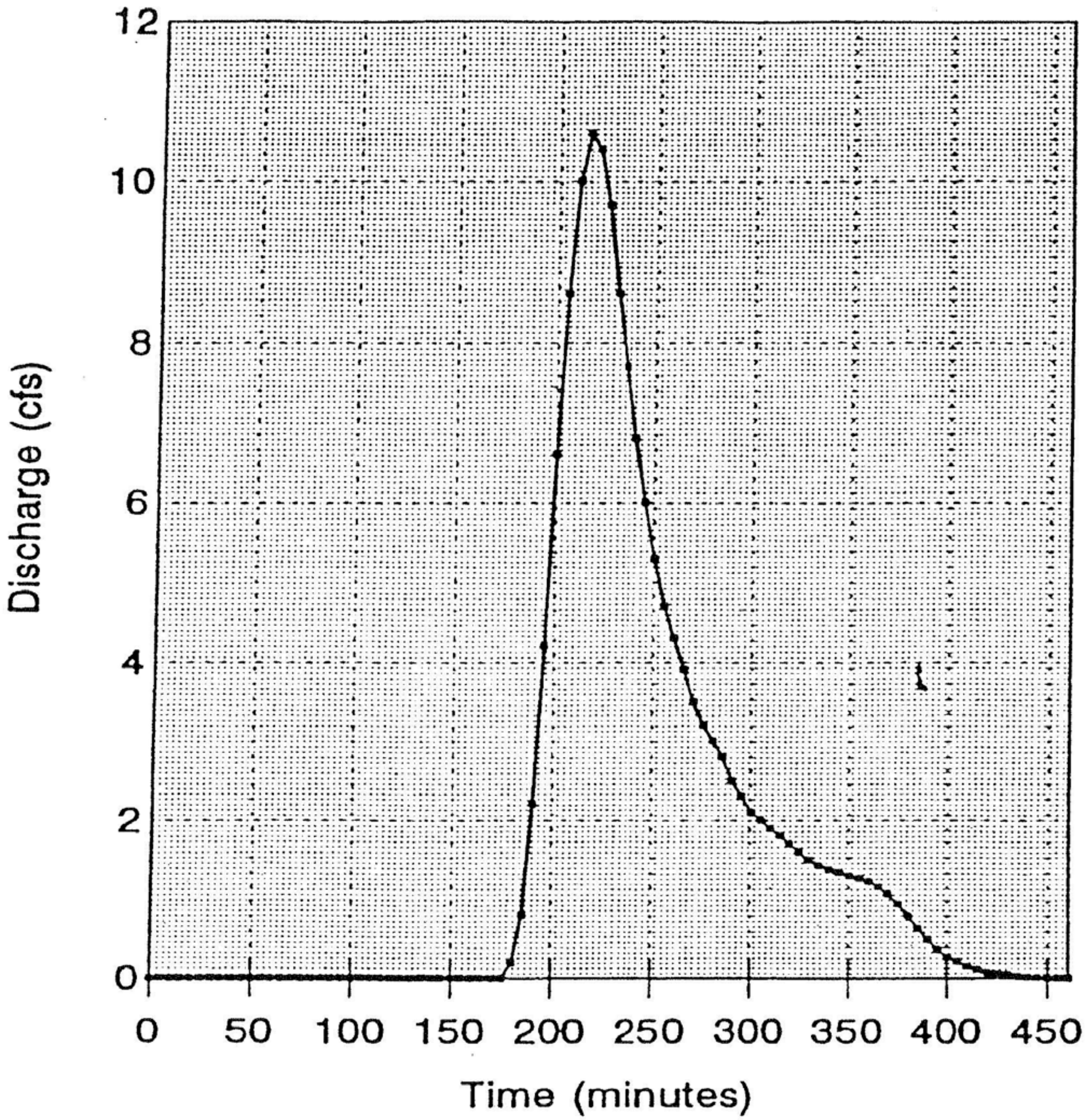


Figure 3-7

Resulting Hydrograph

3.10 Simplified SCS Method

Overview 3.10.1

The following SCS procedures were taken from the SCS Technical Release 55 (TR-55) which presents simplified procedures to calculate storm runoff volumes, peak rate of discharges and hydrographs. These procedures are applicable to small drainage areas and include provisions to account for urbanization. The following procedures outline the use of the SCS-TR 55 method.

Peak Discharges 3.10.2

The SCS peak discharge method is applicable for estimating the peak runoff rate from watersheds with homogeneous land uses. The following method is based on the results of computer analyses performed using TR-20, "Computer Program for Project Formulation-Hydrology" (SCS 1983).

The peak discharge equation is:

$$\mathbf{Q_p = q_u A Q F_p} \qquad \mathbf{(3.26)}$$

Where: Q_p = *peak discharge (cfs)*
q_u = *unit peak discharge* (cfs/mi^2/in)
A = *drainage area* (mi^2)
Q = *runoff (in)*
F_p = *pond and swamp adjustment factor*

The input requirements for this method are as follows:

1. T_c-*hours*
2. *Drainage area* – Mi^2
3. *Type II rainfall distribution*
4. *24-hour design rainfall*
5. *CN value*
6. *Pond and Swamp adjustment factor (If pond and swamp areas are spread throughout the watershed and are not considered in the* T_c *computation, and adjustment is needed.)*

Computations 3.10.3

Computations for the peak discharge method proceed as follows:

1. *The 24-hour rainfall depth is determined from the following table for the selected return frequency.*

Frequency	*24-Hour Rainfall*
2-year	*3.12 inches*
5-year	*4.32 inches*
10-year	*4.80 inches*
25-year	*5.76 inches*
50 year	*6.48 inches*
100-year	*6.96 inches*

Computations (continued)

2. *The runoff curve number, CN and direct runoff, Q are estimated using procedures in Section 3.9.*

3. *The CN value is used to determine the initial abstraction I_a, from Table 3-15 and the ratio I_a/P is then computed. (P = accumulated rainfall or potential maximum runoff).*

4. *The watershed time of concentration is computed using the procedures in Section 3.9 and is used with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 3-8. If the ration I_a/P lies outside the range shown in Figure 3-8, either the limiting values or another peak discharge method should be used.*

5. *The pond and swamp adjustment factor, F_p, is estimated from below:*

Pond and Swamp Areas (%)	F_p
0	*1.00*
0.2	*0.97*
1.0	*0.87*
3.0	*0.75*
5.0	*0.72*

6. *The peak runoff rate is computed using equation 3.26.*

Limitation 3.10.4

The accuracy of the peak discharge method is subject to specific limitations, including the following:

1. *The watershed must be hydrologically homogeneous and describable by a single CN value.*

2. *The watershed may have only one main stream, or if more than one, the individual branches have nearly equal time of concentration.*

3. *Hydrologic routing cannot be considered.*

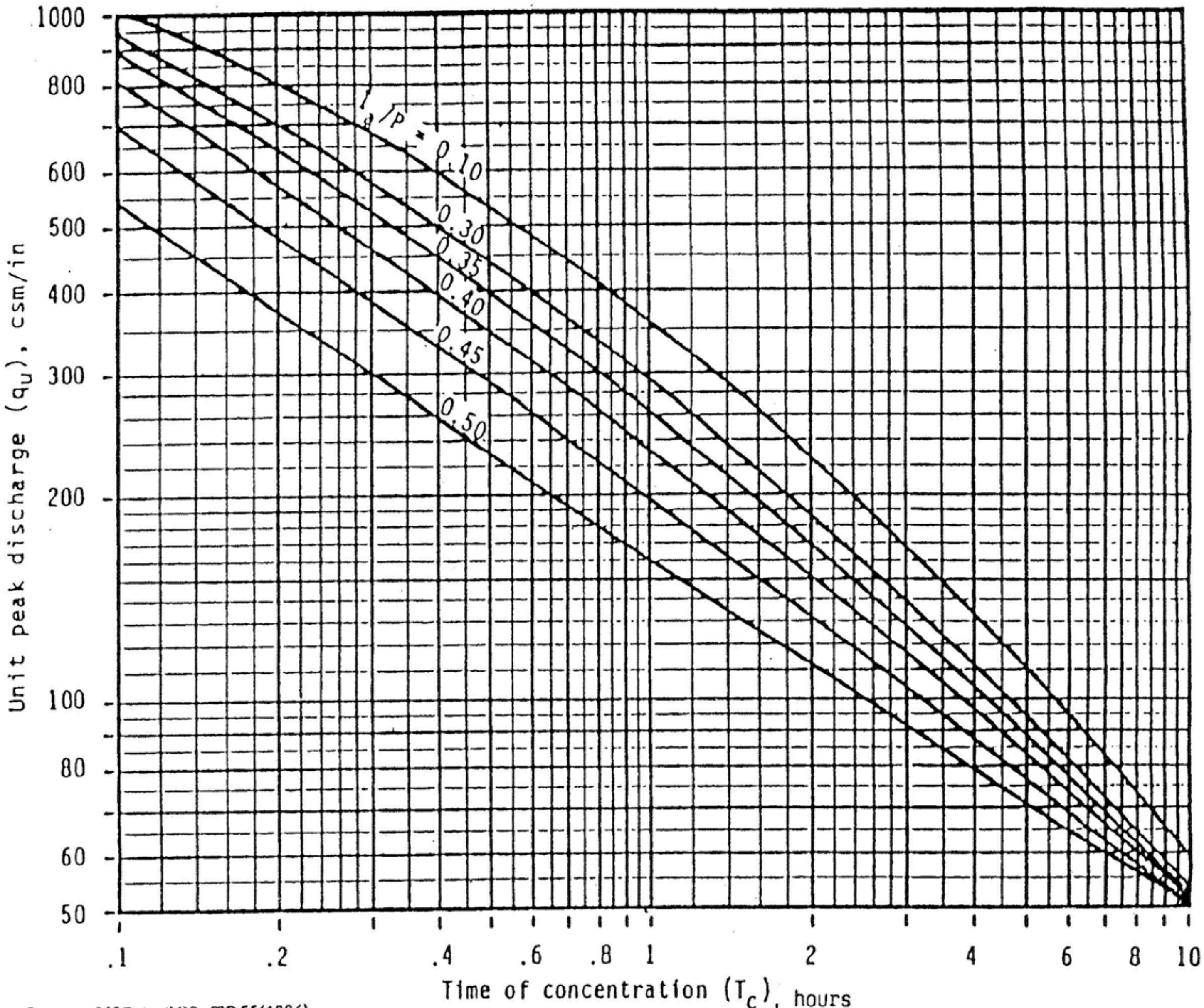
4. *The pond and swamp adjustment factor, F_p, applies only to areas located away from the main flow path.*

5. *Accuracy is reduced if the ratio I_a/P is outside the range given in Figure 3-8.*

6. *The weighted CN value must be greater than or equal to 40 and less than or equal to 98.*

7. *The same procedure should be used to estimate pre- and post- development time of concentration when computing pre- and post- development peak discharge.*

8. *The watershed time of concentration must be between 0.1 and 10 hours.*



Source: USDA, SCS, TR55(1986)

SCS Type II Unit Peak Discharge Graph
Figure 3 - 8

Table 3-15

I_a Values For Runoff Curve Numbers

Curve Numbers	I_a (in.)	Curve Number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Example Problem 3.10.5

Compute the 25-year peak discharge for a 50-acre wooded watershed which will be developed as follows:

1. Forest land – good cover (hydrologic soil group B) = 10 ac.
2. Forest land – good cover (hydrologic soil group C) = 10 ac.
3. Town house residential (hydrologic soil group B) = 20 ac.
4. Industrial development (hydrologic soil group C) = 10 ac.

Other data include: percentage of pond and swamp area = 0.

Example Problem (continued)

Computations

1. Calculate rainfall excess:

- *The 25-year, 24-hour rainfall is 5.76 inches.*

- *Composite weighted runoff coefficient is:*

Dev #	*Area*	*% Total*	*Cn*	*Composite Cn*
1	*10 ac.*	*0.20*	*55*	*11.0*
2	*10 ac.*	*0.20*	*70*	*14.0*
3	*20 ac.*	*0.40*	*85*	*34.0*
4	*10 ac.*	*0.20*	*91*	*18.2*
Total	*50 ac.*	*1.00*		*77.2 use 77*

- *From Figure 3-3, Q = 3.4 inches*

2.Calculate time of concentration

- *The hydrologic flow path for this watershed = 2,000 ft.*

Segment	*Type of Flow*	*Length*	*Slope (%)*
1	*Overland n = 0.45*	*150 ft.*	*6.0%*
2	*Shallow channel*	*750 ft.*	*1.7%*
3	*Main channel**	*1100 ft.*	*0.2%*

**For the main channel, n = 0.025, width = 10 feet, depth = 2 feet, rectangular channel.*

- *Segment 1 – Travel time form equation 3.17 with P_2 = 3.50 inches*

$T_t = [0.42(0.45 \text{ X } 150)^{0.8} / (3.50)^{1/2} (.06)^{0.4}$
$T_t = 20.1$ *minutes*

- *Segment 2 – Travel time from Figure 3-5 and equation 3.15*

V = 2.6 ft/sec (from Figure 3-5 or equation 3.24)

$T_t = 750 / 60 (2.6) = 4.8$ *minutes*

- *Segment 3 – Using equation 3.25*

$V = (1.49/.025) (1.43)^{0.67} (.002)^{1/2} = 3.4$ *ft/sec*

$T_t = 1100 / 60 (3.4) = 5.4$ *minutes*

$T_c = 20.1 + 4.8 + 5.4 = 30.3$ *minutes, use 30 minutes*

Example Problem (continued)

3. Calculate I_a/P for Cn = 77, I_a = .597

.597 / 5.76 = .10

4. Calculate unit discharge q_u from Figure 3-8 = 530 cfs

5. Calculate peak discharge with F_p = 1 (from equation 3.26)

Q_{25} = 530 (50/640) (3.3) (1) = 137 cfs.

Hydrograph Generation 3.10.6

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph. The Soil Conservation Service has developed a Tabular Hydrograph procedure which can be used to generate the hydrograph for small drainage areas. The Tabular Hydrograph uses unit discharge hydrographs which have been generated for a series of time of concentrations. (See example on next page.)

The tables in Appendix A at the end of this chapter give the unit discharges (csm/in) for different times of concentration which are applicable to the Town of Waxhaw area. The values that should be used are those with a travel time equal to zero. The other travel times indicate the unit hydrographs which would result if the hydrographs were routed through a channel system for a length of time equal to the travel time. Thus using these unit hydrographs would account for the effects of channel routing. Straight line interpolation can be used for time of concentrations and travel times between the values given in the appendix.

Composite Hydrograph 3.10.7

The procedures given in this chapter are for generation of a hydrograph from a homogeneous developed drainage area. For drainage areas which are not homogeneous where hydrographs need to be generated form sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55. The 1986 version can be obtained from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

Example Problem 3.10.8

For the example, problem in 3.10.5, calculate the entire hydrograph from the 50 acre development.

Using the chart in Appendix 4 with a time of concentration of .50 hours and Ia / P = 0.10 the following hydrograph can be generated.

The values given in the charts are in csm/in or cubic feet per second per square mile per inch of runoff. Thus for this example all values from the chart must be multiplied by (50 acres/640 acres) per square mile times 3.3 inches of runoff times 1 for the ponding factor.

(50/640)(3.3)(1) = .258

Example Problem (continued)

As an example, from the chart in Appendix A with T_c = .50 hours and I_a/P = 0.10, the unit discharge at time 12.1 hours is 170 cfs. Thus the ordinate on the hydrograph for this example would be 170(.258) = 44 cfs. This calculation must be done for all hydrograph values. The results for selected time values are given below.

*Hydrograph Time (hours)	Unit Discharge (csm/in)	Hydrograph (cfs)
11.0	17	4
11.3	23	6
11.6	32	8
11.9	57	15
12.0	94	24
12.1	170	44
12.2	308	79
12.3	467	120
12.4	529	136
12.5	507	131
12.6	402	104
12.7	297	77
12.8	226	58
13.0	140	36
13.2	96	25
13.4	74	19
13.6	61	16
13.8	53	14
14.0	47	12
14.3	41	11
14.6	36	9
15.0	32	8
15.5	29	7
16.0	26	7
16.5	23	6
17.0	21	5
18.0	19	5
19.0	16	4
20.0	14	4
22.0	12	3
26.0	0	0

*Note skips in time increments

3.11 SCS Step Function (Malcolm Spreadsheet)

Introduction 3.11.1

The SCS step function (Malcolm Spreadsheet) uses three aspects of hydrologic design to formulate a design hydrograph for use in storage design, peakk discharge, volume of runoff, and shape of hydrograph. The method has been revised from the original publication (Malcom) by including a fourth hydrologic aspect, time of start of runoff, to allow the method to be applied to the Town of Waxhaw area. Details of the original methodology can be found in Element of Urban Stormwater Design H. Rooney Malcom.

The Malcom spreadsheet includes the following basic steps:

1. *Determination of a peak discharge for the design storm by methods applicable for the drainage area of interest (Rational Method, SCS Simplified Method, etc.)*

2. *Estimation of the volume of runoff, the area under the hydrograph, with the SCS Curve Number method for a 6-hour runoff volume based on the total rainfall values listed in Table 3-8 and 3-9.*

3. *Application of the peak discharge and volume of runoff to a standard hydrograph shape as defined by a step-function approximation or exponential approximation of the SCS dimensionless unit hydrograph. The relation is used to compute time to peak.*

4. *Shifting the resulting hydrograph in time by computing the initial abstractions and corresponding time for the beginning of runoff when the precipitation event fills the initial abstractions.*

Limitations 3.11.2

The following are limitations of the SCS step function method of hydrograph generation:

1. *The watershed in question must contain less than 50 acres, and should have a homogenous land use.*

2. *The method cannot account for significant storage caused by channel, culvert, or detention basins and cannot be used in areas where storage with the watershed attenuates the resultant hydrograph.*

3. *The timing of the runoff hydrograph is not accurately modeled. Therefore, this method cannot be used where hydrologic timing of hydrographs is a concern.*

4. *The computational interval must be approximately equal to 10 percent of the time to peak (T_p).*

Peak Discharge 3.11.3

The peak discharge may be estimated by any of the approved methods listed in this publication under Section 3.1. Table 3.1 should be referenced to determine the ranges of applicability of each of these methods. However, a reasonableness check of the results should be performed with an alternative method to ensure that the results of the peak estimation are appropriate.

Runoff Volume 3.11.4

The volume of runoff should be estimated using the SCS Curve Number method for a 6-hour balanced storm distribution. The depth of precipitation for the 2-year and 10-year, 6-hour storm events may be obtained from Table 3-8, and 3-9, respectfully. The curve number may be determined by the methods outlined in Section 3.9 using hydrologic soil types and land uses of the sub-watershed in question. The equations for this computations have been listed in Section 3.9, and are repeated as follows:

$$Q = (P - I_a)^2 / (P - I_a) + S$$

where: Q = accumulated direct runoff

P = accumulated rainfall (potential maximum runoff) See Table 3-8, and 3.9.

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff.

S = potential maximum soil retention
$S = 1000 / CN -10$

The empirical relationship used in the SCS runoff equation for estimating Ia is:

$$I_a = 0.2S$$

The volume of runoff associated area under the hydrograph is the product of the runoff depth, Q, and the watershed area. Consistent units should be considered when computing the volume of runoff.

Hydrograph Shape 3.11.5

The shape of the hydrograph may be estimated using either of two equations. The first is referred to as the step function approximation of the SCS dimensionless unit hydrograph. The second is referred to as the exponential approximation of the SCS dimensionless unit hydrograph.

Step Function:

The following equations may be used to estimated the shape of the hydrograph:

For: $0 \leq t \leq 1.25\ T_p$

$$\mathbf{Q = (Q_p / 2)[1 - \cos(\pi t / T_p)]} \qquad \mathbf{(3.27)}$$

For: $t > 1.25\ T_p$

$$\mathbf{Q = 4.34\ Q_p \exp[-1.30\ (t / T_p)]} \qquad \mathbf{(3.28)}$$

Where: Q = *Discharge at the time of interest.*

Q_p = *Peak discharge of the design hydrograph (determined in step 1)*

t = *Time of interest at which the discharge is to be estimated*

T_p = *Time to peak of the design hydrograph measured from the time of significant rise of the rising limb of the hydrograph to the time at which the estimated peak occurs. This value is found by solving the following equation which was derived by integrating the above step function.*

$$\mathbf{Vol = 1.39\ Q_p\ T_p}$$

This relation can be solved fro Tp with the known values of Qp and Vol determined by the computations used in Step 1 and 2. Consistent units must be used for this relation.

The step function only approximates the SCS unit hydrograph shape. Therefore, equation (3.29) provides a result that is an approximation of the volume of the hydrograph. Solution of the actual SCS unit hydrograph results in a similar equation form with a coefficient of 1.33 instead of 1.39, as shown in equation (3.29).

Exponential Approximation:

The following single equation may be used to approximate the SCS Cuvilinear dimensionless unit hydrograph in lieu of the step function:

$$Q = Q_p\ [(t / T_p)\ e^{(1 - t/Tp)}]^{(n-1)}$$

Where: Q, Qp, Tp, and t are defined above

n = constant = 4.7 for the SCS curvilinear unit hydrograph shape

---

Start of Runoff
3.11.6

The time at which runoff beings is computed by calculating the initial abstractions and comparing the result to the accumulated precipitation of the design storm event. Initial abstractions have been previously computed in Step 2.

$$I_a = 0.2\ S$$

where: $S = 1000 / CN - 10$

Accumulated precipitation data is available in Appendix C for the 2-, 10-, and 50-year, 6-hour storm events.

Example Problem 3.11.7

Compute a 10-year, 6-hour storm event hydrograph given the following information:

Basin Area = 1.9 acres CN 81.7
CN = 81.7
Q_p = 7.7 cfs (computed by the HEC-1 model using SCS dimensionless unit hydrograph theory).

Potential maximum soil retention:
S = 1000 / CN – 10 = 1000 / 81.7 – 10 = 2.24 inches

Initial Abstraction:
I_a = 0.2 S = (0.2) (2.24) = 0.45 inches

Direct runoff:

$Q = [P - I_a]^2 / [P - I_a] + S = [3.72 - 0.45]^2 / \{ [3.72 - 0.45] + 2.24\}$
Q = 1.94 inches

Time to peak:

$T_p = Vol / 1.39\ Q_p$

$$T_p = \frac{(1.94 \text{ in.})(1 \text{ ft.}/12 \text{ in.})(1.9 \text{ acres})(43{,}560 \text{ sq. ft./acre})}{(1.39)(7.7 \text{ cfs})(60 \text{ sec/min})}$$

T_p = 20.8 min.

Start of Runoff:

Investigation of the 10-year storm event accumulated precipitation in Appendix C indicates that the initial abstractions of 0.45 inches of runoff are filled at time 124 minutes, therefore, runoff begins at the time 124 minutes.

Hydrograph generation:

Table 3-16 lists the results of computing the hydrograph ordinates using the steps function equations. The hydrograph has been shifted by the required time as computed to fill the initial abstractions. Column 3 of Table 3-16 lists the hydrograph ordinates resulting from a HEC-1 application of the SCS dimensionless unit hydrograph using the same input parameters. Figure 3-9 illustrates the resulting hydrographs using the 2 methods.

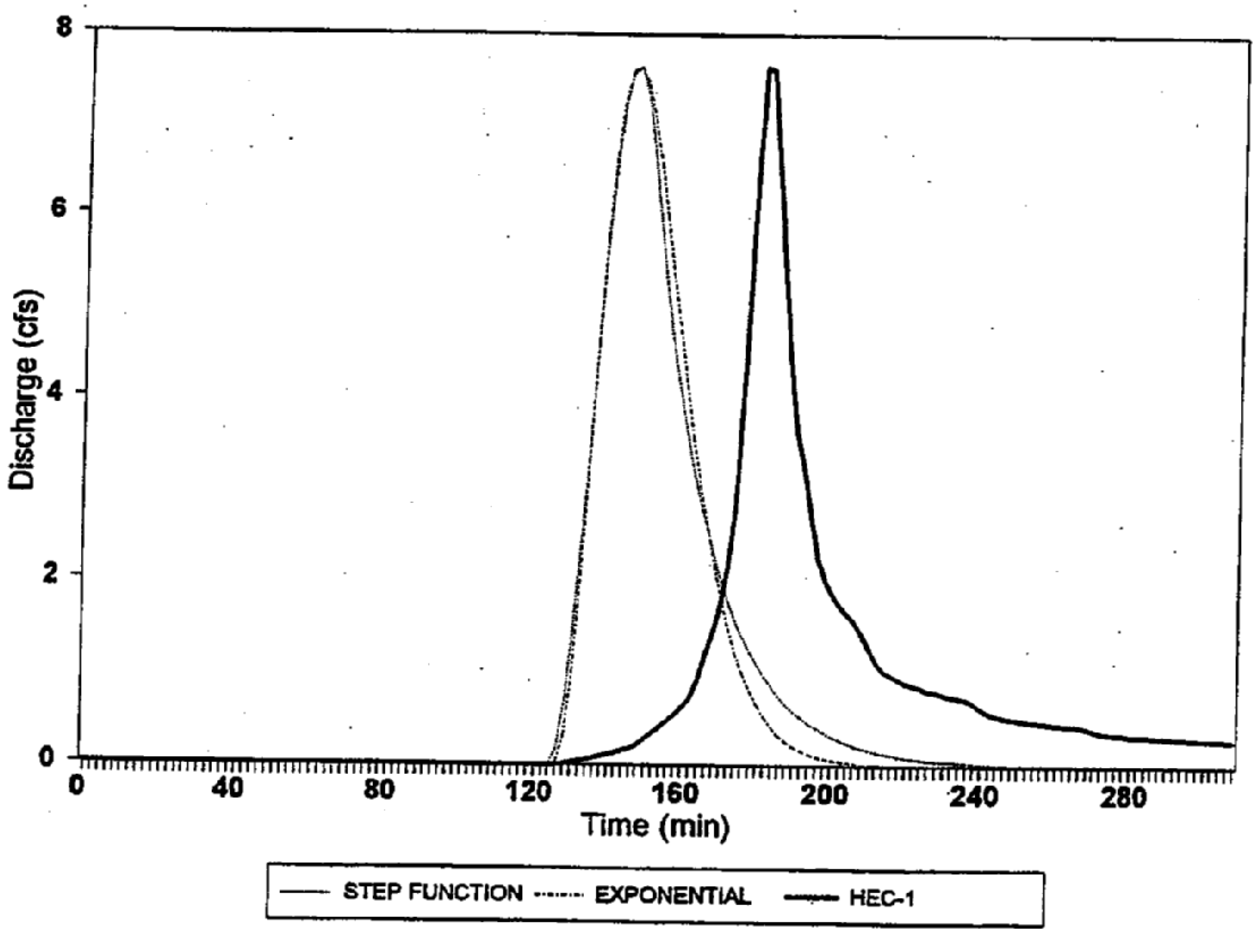
Table 3-16
Example Hydrograph Results

Time (min)	Step Function Hydrograph (cfs)	Exponential Approximation Hydrograph (cfs)	HEC-1 Unit Dimensionless Hydrograph (cfs)
0-122	0.000	0.000	0.000
126	0.165	0.035	0.006
128	0.647	0.318	0.016
130	1.404	1.006	0.029
132	2.371	2.061	0.043
134	3.463	3.324	0.059
136	4.588	4.611	0.075
138	5.646	5.763	0.093
140	6.548	6.673	0.112
142	7.215	7.289	0.133
144	7.590	7.605	0.155
146	7.640	7.645	0.181
148	7.360	7.453	0.221
150	6.776	7.080	0.283
152	6.019	6.580	0.351
154	5.328	6.001	0.415
156	4.715	5.383	0.477
158	4.174	4.760	0.549
160	3.694	4.155	0.632
162	3.270	3.586	0.717
164	2.894	3.063	0.929
166	2.561	2.592	1.212
168	2.267	2.175	1.445
170	2.006	1.812	1.729
172	1.776	1.498	2.210
174	1.572	1.231	2.910
176	1.391	1.006	4.456
178	1.231	0.817	6.306

Time (min)	Step Function Hydrograph (cfs)	Exponential Approximation Hydrograph (cfs)	HEC-1 Unit Dimensionless Hydrograph (cfs)
180	1.090	0.660	7.643
182	0.965	0.531	7.612
184	0.854	0.425	6.558
186	0.756	0.339	5.574
188	0.669	0.270	4.508
190	0.592	0.213	3.675
192	0.524	0.168	3.250
194	0.464	0.132	2.745
196	0.410	0.104	2.258
198	0.363	0.081	2.015
200	0.322	0.063	1.849
202	0.285	0.049	1.726
204	0.252	0.038	1.644
206	0.223	0.030	1.564
208	0.197	0.023	1.438
210	0.175	0.018	1.263
212	0.155	0.014	1.115
214	0.137	0.010	1.032
216	0.121	0.008	0.984
218	0.107	0.006	0.942
220	0.095	0.005	0.905
222	0.084	0.004	0.879
224	0.074	0.003	0.850
226	0.066	0.002	0.822
228	0.058	0.002	0.802
230	0.052	0.001	0.780
232	0.046	0.001	0.758
234	0.040	0.001	0.741
236	0.036	0.001	0.722

Time (min)	Step Function Hydrograph (cfs)	Exponential Approximation Hydrograph (cfs)	HEC-1 Unit Dimensionless Hydrograph (cfs)
238	0.032	0.000	0.686
240	0.028	0.000	0.628
242	0.025	0.000	0.578
244	0.022	0.000	0.550
246	0.019	0.000	0.535
248	0.017	0.000	0.522
250	0.015	0.000	0.509
252	0.013	0.000	0.501
254	0.012	0.000	0.490
256	0.011	0.000	0.479
258	0.009	0.000	0.471
260	0.008	0.000	0.462
262	0.007	0.000	0.454
264	0.006	0.000	0.447
266	0.006	0.000	0.439
268	0.005	0.000	0.423
270	0.004	0.000	0.395
272	0.004	0.000	0.371
274	0.004	0.000	0.359
276	0.003	0.000	0.352
278	0.003	0.000	0.346
280	0.002	0.000	0.339
282	0.002	0.000	0.334
284	0.002	0.000	0.329
286	0.002	0.000	0.325
288	0.001	0.000	0.319
290	0.001	0.000	0.314
292	0.001	0.000	0.311
294	0.001	0.000	0.307

Time (min)	Step Function Hydrograph (cfs)	Exponential Approximation Hydrograph (cfs)	HEC-1 Unit Dimensionless Hydrograph (cfs)
296	0.001	0.000	0.303
298	0.001	0.000	0.299
300	0.001	0.000	0.295
302	0.001	0.000	0.291
304	0.001	0.000	0.287
306	0.000	0.000	0.285
308	0.000	0.000	0.281



Resulting Hydrographs

Figure 3-9

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Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0
IA/P = 0.10														* * * TC = 0.1 HR * * *														IA/P = 0.10			
0.0	24	34	53	334	647	1010	623	217	147	123	104	86	76	66	57	51	46	42	38	34	32	29	26	23	21	20	19	18	15	13	12
.10	21	29	43	134	267	520	847	701	378	224	157	122	98	75	64	56	50	45	41	36	33	30	27	24	21	20	19	18	16	13	12
.20	18	25	35	61	110	215	418	704	702	486	312	209	151	94	73	62	54	49	44	38	34	31	28	25	22	21	19	18	16	14	12
.30	17	23	33	56	92	174	337	582	662	543	369	269	190	109	79	65	56	50	45	39	35	32	29	25	22	21	20	18	16	14	12
.40	15	20	28	41	51	73	142	272	478	601	563	447	328	172	104	76	63	55	49	42	37	33	29	26	23	21	20	19	17	14	12
.50	14	19	26	39	47	65	117	220	392	531	553	462	380	209	121	84	67	57	51	43	38	33	30	27	23	21	20	19	17	14	12
.75	12	15	21	29	33	33	49	73	126	224	343	432	464	385	252	156	103	76	62	50	43	36	31	28	25	22	21	19	17	15	12
1.0	9	12	15	21	23	26	29	33	40	55	86	148	238	406	434	317	205	130	89	62	50	41	34	30	27	24	22	20	18	16	12
1.5	7	8	10	14	15	16	18	20	22	25	29	34	43	101	220	339	373	320	234	131	80	53	40	34	30	27	24	21	19	17	12
2.0	4	6	7	9	9	10	11	12	13	15	16	18	20	25	37	72	150	252	336	312	216	109	58	42	34	30	27	24	20	18	13
2.5	3	4	5	6	7	7	8	8	9	10	11	12	13	16	19	25	39	75	142	262	305	229	108	58	41	34	30	27	22	19	14
3.0	1	2	3	4	4	5	5	6	6	7	7	8	8	10	12	14	17	22	31	76	169	288	236	122	64	43	35	30	24	20	16
IA/P = 0.30														* * * TC = 0.1 HR * * *														IA/P = 0.30			
0.0	0	0	0	154	568	936	524	217	172	149	126	107	97	86	76	69	63	58	53	48	46	42	38	34	31	30	28	27	24	20	19
.10	0	0	0	19	109	415	762	603	346	230	176	143	119	96	84	74	68	62	57	50	47	44	40	35	32	30	29	27	24	21	19
.20	0	0	0	0	13	77	302	609	605	432	297	217	167	115	94	81	73	66	60	53	48	45	41	37	33	31	29	28	25	21	19
.30	0	0	0	0	9	54	219	479	563	476	357	263	199	129	99	85	75	68	62	54	49	45	41	37	33	31	29	28	25	21	19
.40	0	0	0	0	0	6	38	159	372	500	484	399	309	183	123	96	82	73	66	58	51	46	42	38	34	31	30	28	25	22	19
.50	0	0	0	0	0	4	27	115	287	429	465	421	346	213	138	103	86	76	68	59	52	47	43	39	34	32	30	29	25	22	19
.75	0	0	0	0	0	0	1	10	46	132	246	335	381	341	243	165	119	94	80	67	58	50	45	41	37	33	31	29	26	23	19
1.0	0	0	0	0	0	0	0	1	4	22	69	149	241	357	331	246	170	122	96	76	64	54	47	42	38	34	32	30	27	24	19
1.5	0	0	0	0	0	0	0	0	0	0	0	1	4	41	142	258	310	285	224	142	97	71	55	47	43	39	35	32	29	25	20
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	10	49	130	221	279	255	182	108	70	55	47	42	38	34	30	27	20
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	52	119	224	256	193	107	70	55	47	42	38	32	28	22
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	52	141	240	199	117	74	56	48	43	35	30	24
IA/P = 0.50														* * * TC = 0.1 HR * * *														IA/P = 0.50			
0.0	0	0	0	0	70	539	377	196	171	154	134	117	108	99	89	83	77	72	67	61	59	56	51	46	43	42	40	38	34	30	28
.10	0	0	0	0	47	375	376	256	199	169	146	126	114	102	92	85	79	73	68	62	59	56	52	47	43	42	40	38	34	30	28
.20	0	0	0	0	0	31	260	338	283	227	189	160	138	112	99	90	83	77	72	64	60	57	53	48	44	42	41	39	35	30	29
.30	0	0	0	0	0	0	21	180	285	284	246	208	176	131	110	97	88	82	76	68	62	59	54	50	45	43	41	39	36	31	28
.40	0	0	0	0	0	0	14	125	232	266	253	223	162	142	115	100	91	83	77	69	63	59	55	50	45	43	41	40	36	31	28
.50	0	0	0	0	0	0	9	86	183	239	248	231	205	154	122	104	93	85	79	71	64	59	55	51	46	43	41	40	36	32	25
.75	0	0	0	0	0	0	3	31	87	147	190	211	213	164	147	121	103	92	84	75	67	61	57	52	47	44	42	40	37	32	28
1.0	0	0	0	0	0	0	0	0	1	13	45	92	141	205	197	165	134	112	98	84	75	65	57	55	50	46	43	41	38	34	28
1.5	0	0	0	0	0	0	0	0	0	0	0	2	9	51	118	170	183	167	143	111	92	77	65	59	54	50	45	43	39	35	28
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	15	51	103	148	168	156	127	96	76	65	58	54	49	45	41	37	29
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	31	69	131	159	140	101	78	66	59	54	50	43	39	31
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	45	101	151	134	99	77	65	59	54	45	41	33

RAINFALL TYPE = II   * * * TC = 0.1 HR * * *   SHEET 1 OF 10

continued: Tabular hydrograph unit discharges (csm/in) for type

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26
IA/P = 0.10														* * * TC =0.2 HR * * *														IA/P = 0.10				
0.0	23	31	47	209	403	739	800	481	250	166	128	102	86	70	61	54	49	44	40	35	33	30	27	24	21	20	19	18	16	13	12	
.10	19	26	39	86	168	325	601	733	565	355	229	161	122	83	69	59	53	47	43	37	34	31	28	25	22	21	19	18	16	14	12	
.20	17	23	32	49	74	136	262	488	652	594	435	298	207	115	81	67	58	51	46	40	35	32	29	26	23	21	20	19	16	14	12	
.30	16	22	30	46	64	112	212	396	566	585	485	360	258	139	90	71	60	53	48	41	36	32	29	26	23	21	20	19	16	14	12	
.40	14	19	25	37	43	57	94	173	322	485	551	507	409	227	129	87	68	58	52	44	38	33	30	27	24	21	20	19	17	14	12	
.50	13	18	24	35	40	52	80	142	262	410	504	506	441	269	153	98	73	61	53	45	39	34	30	27	24	22	20	19	17	15	12	
.75	10	13	17	23	26	30	34	40	55	86	150	247	349	438	360	240	151	101	75	57	47	39	33	29	26	23	21	20	18	15	12	
1.0	9	11	14	19	21	24	26	30	35	44	62	101	167	337	413	353	245	157	104	68	53	42	35	31	28	24	22	20	18	16	12	
1.5	6	8	10	13	14	15	17	19	21	23	26	30	37	73	166	288	356	337	264	154	91	57	42	35	30	27	24	22	19	17	13	
2.0	4	5	7	8	9	10	10	11	12	14	15	16	18	23	31	55	114	206	291	324	239	125	63	44	35	31	28	24	20	18	14	
2.5	3	4	5	6	6	7	7	8	9	9	10	11	12	15	18	22	32	58	111	227	298	246	122	63	43	35	31	27	22	19	15	
3.0	1	2	3	4	4	4	5	5	6	6	7	7	8	9	11	13	16	19	27	59	138	280	248	137	70	46	36	31	25	21	16	
IA/P = 0.30														* * * TC =0.2 HR * * *														IA/P = 0.30				
0.0	0	0	0	39	180	545	697	497	276	198	158	130	110	93	81	73	67	61	56	49	46	43	39	35	32	30	29	27	24	21	19	
.10	0	0	0	2	27	129	407	600	532	341	252	190	150	108	90	79	71	65	59	52	45	44	41	36	32	31	29	28	25	21	19	
.20	0	0	0	2	19	92	302	501	521	415	306	228	176	119	95	82	73	67	61	53	48	45	41	37	33	31	29	28	25	21	19	
.30	0	0	0	0	1	13	66	223	408	484	438	350	269	163	114	93	80	72	65	57	51	46	42	38	34	31	30	28	25	22	19	
.40	0	0	0	0	1	9	47	164	327	431	436	379	306	189	127	98	83	74	67	58	52	47	43	38	34	31	30	28	25	22	19	
.50	0	0	0	0	0	0	6	33	120	258	374	415	391	271	173	121	95	81	72	62	55	48	44	40	35	32	30	29	26	22	19	
.75	0	0	0	0	0	0	2	13	50	126	221	302	348	323	240	167	121	96	81	68	59	50	45	41	37	33	31	29	26	23	19	
1.0	0	0	0	0	0	0	0	0	1	6	24	69	139	285	331	280	204	145	109	82	68	56	48	43	39	35	32	30	27	24	19	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	16	79	186	271	288	247	165	110	76	58	49	44	40	35	32	29	26	20	
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	24	80	163	235	262	202	123	76	58	49	43	39	35	30	27	21	
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	28	77	179	242	207	120	75	57	48	43	39	32	29	22	
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	30	101	207	227	130	80	59	49	44	35	30	24	
IA/P = 0.50														* * * TC =0.2 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	7	98	371	322	221	182	158	137	120	104	94	86	80	74	69	62	60	57	52	47	44	42	40	39	35	30	29	
.10	0	0	0	0	4	67	270	305	249	204	174	149	130	108	97	88	82	76	71	64	60	57	53	48	44	42	41	39	35	30	28	
.20	0	0	0	0	0	3	45	195	268	255	221	189	163	125	106	95	87	80	75	67	62	58	54	49	45	43	41	39	35	31	28	
.30	0	0	0	0	0	2	31	140	226	245	229	203	176	134	111	98	89	82	76	68	62	59	55	50	45	43	41	39	36	31	28	
.40	0	0	0	0	0	1	21	101	184	225	228	211	183	144	117	101	91	84	78	69	63	59	55	50	45	43	41	40	36	31	28	
.50	0	0	0	0	0	0	1	14	72	146	199	218	213	175	137	113	99	89	82	73	66	60	56	52	47	43	42	40	36	32	28	
.75	0	0	0	0	0	0	0	5	28	71	121	162	186	193	161	133	112	98	88	78	70	62	57	53	48	44	42	41	37	33	28	
1.0	0	0	0	0	0	0	0	0	0	2	13	38	77	154	186	174	147	122	105	89	78	68	60	56	51	46	43	42	38	34	28	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	22	71	129	163	168	150	120	98	80	67	60	55	51	46	43	40	36	28	
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	25	65	112	146	157	134	103	79	67	60	55	50	46	41	38	29	
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	26	60	117	148	136	101	79	66	59	54	50	43	39	31	
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	40	90	142	130	99	78	66	59	54	45	41	33	

RAINFALL TYPE = II * * * TC = 0.2 HR * * * SHEET 2 OF 10

continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.
IA/P = 0.10														• • • TC = 0.3 HR • • •															IA/P = 0.10			
0.0	20	28	41	118	235	447	676	676	459	283	196	146	114	80	66	57	51	46	42	37	33	31	28	24	22	20	19	18	16	13	12	
.10	19	26	39	99	189	361	571	641	520	362	251	181	136	89	70	60	53	48	43	37	34	31	28	25	22	21	19	18	16	14	12	
.20	17	23	32	53	83	154	292	478	587	542	422	308	223	127	86	68	58	52	46	40	35	32	29	26	23	21	20	19	16	14	12	
.30	16	22	30	49	72	127	237	398	524	536	460	359	268	151	97	73	61	53	48	41	36	32	29	26	23	21	20	19	16	14	12	
.40	14	19	25	37	45	63	105	193	330	459	510	477	398	237	139	92	70	59	52	44	38	34	30	27	24	21	20	19	17	14	12	
.50	13	18	24	35	42	56	89	158	272	397	472	475	424	274	163	104	76	62	54	46	39	34	30	27	24	22	20	19	17	15	12	
.75	11	14	19	26	30	34	42	59	95	160	250	339	417	398	299	196	128	89	69	54	45	37	32	29	26	23	21	20	17	15	12	
1.0	9	11	14	19	21	24	27	30	36	46	68	109	174	328	396	346	248	163	109	70	54	43	35	31	28	24	22	20	13	16	12	
1.5	6	8	10	13	14	15	17	19	21	23	26	31	38	77	169	282	347	330	264	158	94	58	42	35	31	27	24	22	19	17	13	
2.0	4	5	7	8	9	10	10	11	12	14	15	16	18	23	32	57	116	205	285	317	239	128	64	44	36	31	28	25	20	18	14	
2.5	2	4	5	6	6	7	7	8	9	9	10	11	12	15	18	23	33	60	113	223	293	245	125	65	44	35	31	27	22	19	15	1
3.0	1	2	3	4	4	4	5	5	6	6	7	7	8	9	11	13	16	20	27	61	138	275	246	139	72	46	36	31	25	21	16	1
IA/P = 0.30														• • • TC = 0.3 HR • • •															IA/P = 0.30			
0.0	0	0	0	11	64	251	525	574	454	303	221	173	140	104	88	77	70	64	58	51	47	44	40	36	32	31	29	28	24	21	19	
.10	0	0	0	0	7	45	183	411	520	476	360	268	205	133	101	85	76	69	62	55	49	45	41	37	33	31	30	28	25	21	19	
.20	0	0	0	0	5	32	132	318	452	468	396	310	240	151	109	90	78	70	64	56	50	46	42	38	33	31	30	28	25	22	19	
.30	0	0	0	0	0	3	22	96	244	383	440	411	344	217	142	105	87	76	69	60	53	47	43	39	35	32	30	29	26	22	19	
.40	0	0	0	0	0	2	16	69	186	317	399	407	365	246	160	115	92	79	71	61	54	48	43	39	35	32	30	29	26	22	19	
.50	0	0	0	0	0	0	2	11	50	140	258	352	389	327	223	149	110	89	77	66	57	50	45	41	36	33	31	29	26	23	19	
.75	0	0	0	0	0	0	1	4	20	63	135	219	290	335	281	205	146	110	89	72	62	52	46	42	38	34	31	30	27	25	19	
1.0	0	0	0	0	0	0	0	0	0	2	9	32	78	216	320	306	243	176	128	90	72	59	49	44	40	36	33	31	28	24	19	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	2	20	84	185	264	281	246	168	112	77	58	49	44	40	36	32	29	26	20	
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	12	50	121	200	257	224	141	83	61	50	44	40	36	31	25	21	1
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	51	145	239	223	137	82	60	50	44	40	33	29	22	1
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	19	74	184	224	146	89	63	51	45	36	31	24	1
IA/P = 0.50														• • • TC = 0.3 HR • • •															IA/P = 0.50			
0.0	0	0	0	0	1	25	151	299	277	219	187	162	141	113	100	90	84	75	72	65	61	58	53	48	44	42	41	39	35	31	28	
.10	0	0	0	0	1	17	106	235	263	234	202	175	152	120	104	93	85	79	73	66	61	58	54	49	44	42	41	39	35	31	28	
.20	0	0	0	0	0	0	12	75	182	236	234	213	188	144	116	101	91	84	78	70	63	59	55	50	45	43	41	40	36	31	28	
.30	0	0	0	0	0	0	8	52	138	203	224	217	197	154	123	105	94	86	79	71	64	59	55	51	46	43	42	40	36	32	28	
.40	0	0	0	0	0	0	5	37	105	170	206	213	203	164	131	110	97	88	81	72	65	60	56	51	46	43	42	40	36	32	28	
.50	0	0	0	0	0	0	0	4	26	78	140	184	203	191	155	126	107	95	86	76	69	62	57	53	48	44	42	41	37	33	28	
.75	0	0	0	0	0	0	0	1	10	34	73	117	153	184	173	146	122	105	94	82	73	64	58	54	49	45	43	41	37	33	28	
1.0	0	0	0	0	0	0	0	0	0	0	4	17	42	114	168	178	159	134	114	94	82	70	61	57	52	47	44	42	39	35	28	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	44	98	144	163	157	130	105	84	69	61	56	52	47	44	40	36	29	
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	14	44	87	127	153	141	110	83	69	61	56	51	47	42	38	30	
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	16	42	97	138	145	107	82	68	60	55	51	43	40	32	
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	27	71	127	139	105	81	68	60	55	46	41	33	

RAINFALL TYPE = II   • • • TC = 0.3 HR • • •

HYDROGRAPH TIME (HOURS)

* * * TC = 0.4 HR * * * IA/P = 0.10

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	18	25	36	77	141	271	448	592	574	431	298	216	163	104	77	63	55	49	44	38	34	31	28	25	22	21	20	18	16	14	12	0
.10	18	24	34	67	116	219	385	523	557	473	357	263	196	119	84	67	57	51	46	39	35	32	29	25	22	21	20	19	16	14	12	0
.20	15	20	28	44	59	97	179	316	454	523	489	401	309	178	112	81	65	56	49	42	37	33	30	26	23	21	20	19	17	14	12	0
.30	15	20	27	41	53	82	147	260	389	478	486	429	349	210	129	89	69	58	51	43	38	33	30	27	24	21	20	19	17	14	12	0
.40	13	17	23	33	38	48	71	121	214	331	429	467	442	308	189	120	85	66	56	47	41	35	31	28	24	22	20	19	17	15	12	0
.50	12	16	22	31	36	44	62	102	176	279	379	438	440	339	218	137	94	71	59	49	42	35	31	28	25	22	21	19	17	15	12	0
.75	10	13	17	24	26	30	35	45	65	106	170	251	326	393	341	245	164	112	81	59	48	39	33	30	26	23	21	20	18	15	12	0
1.0	8	10	13	17	19	21	24	27	31	37	50	75	118	251	360	376	292	205	138	83	60	45	36	32	28	25	22	21	18	16	12	1
1.5	6	7	9	12	13	14	15	17	19	21	23	26	31	36	121	224	311	333	293	192	115	66	45	36	31	28	25	22	19	17	13	4
2.0	4	5	6	8	8	9	10	10	11	12	14	15	16	20	27	43	85	159	243	306	264	154	74	47	37	32	28	25	21	18	14	9
2.5	2	3	4	5	6	6	7	7	8	9	9	10	11	13	16	20	27	46	85	184	285	262	147	74	47	37	32	25	22	19	15	11
3.0	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	14	17	23	47	109	227	268	160	83	50	38	32	25	21	16	11

* * * TC = 0.4 HR * * * IA/P = 0.30

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	4	26	113	296	480	495	413	306	234	186	127	100	84	74	67	61	54	49	45	41	37	33	31	29	28	25	21	19	0
.10	0	0	0	0	2	18	81	224	395	462	430	347	272	172	121	96	82	73	66	57	51	46	42	38	34	31	30	28	25	22	19	0
.20	0	0	0	0	2	13	59	169	320	414	424	373	305	196	134	103	85	75	67	59	52	47	43	39	34	32	30	29	25	22	19	0
.30	0	0	0	0	0	1	9	42	127	255	361	403	383	274	181	127	99	83	73	63	55	48	44	40	36	32	30	29	26	23	19	0
.40	0	0	0	0	0	1	6	30	94	202	308	372	379	298	205	141	106	87	76	65	56	49	44	40	36	32	31	29	26	23	19	0
.50	0	0	0	0	0	0	0	4	21	70	158	258	334	364	270	187	133	102	85	70	60	51	46	41	37	33	31	30	26	23	19	0
.75	0	0	0	0	0	0	0	2	8	30	76	145	219	321	305	241	177	130	102	78	65	55	47	43	38	34	32	30	27	24	19	0
1.0	0	0	0	0	0	0	0	0	0	1	4	15	42	150	267	308	272	209	154	103	79	62	51	45	41	37	33	31	28	25	19	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	1	10	51	136	226	274	263	195	131	85	62	51	45	41	36	33	29	26	20	6
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	31	86	162	252	239	162	93	64	52	45	41	37	31	28	21	15
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	33	112	202	235	155	92	64	52	45	41	33	29	23	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	21	76	182	221	148	90	63	51	45	36	31	24	13

* * * TC = 0.4 HR * * * IA/P = 0.50

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
0.0	0	0	0	0	0	7	59	168	245	257	213	186	163	128	109	96	88	81	75	67	62	58	54	50	45	43	41	39	35	31	28	0
.10	0	0	0	0	0	0	5	41	125	205	240	222	198	154	123	106	94	86	79	71	64	60	56	51	46	43	42	40	36	32	28	0
.20	0	0	0	0	0	0	3	28	93	168	216	220	205	164	131	110	97	88	81	72	65	60	56	51	46	43	42	40	36	32	28	0
.30	0	0	0	0	0	0	0	2	20	69	135	189	209	192	155	126	107	95	86	77	69	62	57	53	48	44	42	41	37	33	28	0
.40	0	0	0	0	0	0	0	1	14	50	106	161	193	202	163	133	112	98	89	78	70	62	58	53	48	44	42	41	37	33	28	0
.50	0	0	0	0	0	0	0	1	9	37	83	135	174	194	171	140	117	102	91	80	71	63	58	54	49	45	43	41	37	33	28	0
.75	0	0	0	0	0	0	0	0	0	3	15	40	76	147	177	169	146	124	107	90	79	68	60	56	51	47	43	42	38	34	28	0
1.0	0	0	0	0	0	0	0	0	0	0	1	7	21	78	141	173	167	146	125	101	86	73	63	58	53	48	45	42	39	35	28	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	5	26	71	121	153	159	139	113	89	72	63	57	53	48	44	40	37	29	7
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	16	45	86	138	150	125	93	74	64	55	53	48	42	39	31	20
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	59	112	143	121	91	73	63	57	53	45	40	32	26
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	11	40	101	138	117	90	73	63	57	48	42	34	27

RAINFALL TYPE = II   * * * TC = 0.4 HR * * *   SHEET 4 OF 10

Source: USDA, SCS, TR55 (1986)

continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

HYDROGRAPH TIME (HOURS)

IA/P = 0.10 — * * * TC = 0.5 HR * * *

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.
0.0	17	23	32	57	94	170	308	467	529	507	402	297	226	140	96	74	61	53	47	41	36	32	29	26	23	21	20	19	16	14	12	
.10	16	22	30	51	80	140	252	395	484	499	434	343	265	162	105	80	65	55	49	42	36	33	29	26	23	21	20	19	16	14	12	
.20	14	19	25	38	47	69	116	207	332	434	477	449	378	238	149	101	77	62	53	45	39	34	30	27	24	22	20	19	17	14	12	
.30	13	18	24	35	43	60	97	170	274	382	446	448	401	270	171	114	83	66	56	46	40	34	31	27	24	22	20	19	17	15	12	
.40	12	15	21	29	33	40	53	83	141	233	332	408	434	361	243	157	107	79	64	51	43	36	32	28	25	22	21	20	17	15	12	
.50	11	15	20	28	31	37	48	71	118	194	286	367	412	378	271	178	119	86	68	53	44	37	32	29	25	23	21	20	17	15	12	
.75	9	11	14	19	21	24	27	31	37	49	74	116	182	319	374	328	244	169	117	76	56	43	35	31	28	25	22	21	18	16	12	
1.0	7	9	12	16	17	19	21	24	27	32	40	55	83	188	309	359	322	245	172	102	68	49	38	32	29	26	23	21	19	16	12	
1.5	5	7	8	11	12	13	14	15	17	19	21	23	27	43	89	175	269	322	309	225	140	77	49	38	32	29	25	23	20	17	13	
2.0	3	4	6	7	8	8	9	10	10	11	12	14	15	18	23	35	65	123	202	297	280	181	88	52	39	33	29	26	21	19	14	
2.5	2	3	4	5	5	6	6	7	7	8	9	9	10	12	15	18	24	36	66	150	244	278	171	87	52	39	33	29	23	20	15	
3.0	1	1	2	3	3	4	4	4	5	5	6	6	7	8	9	11	13	16	20	37	86	198	263	182	96	56	40	33	26	21	16	

IA/P = 0.30 — * * * TC = 0.5 HR * * *

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.
0.0	0	0	0	1	9	53	157	314	433	439	379	299	237	159	115	95	81	71	65	56	50	46	42	38	34	31	30	25	25	22	19	
.10	0	0	0	0	1	6	37	117	248	372	416	391	330	218	150	113	92	79	70	60	53	47	43	39	35	32	30	29	26	22	19	
.20	0	0	0	0	1	4	26	87	194	313	352	388	349	244	167	122	97	82	72	62	54	48	43	39	35	32	30	29	26	22	19	
.30	0	0	0	0	0	0	3	19	64	151	259	341	372	316	223	156	117	94	80	67	58	50	45	41	36	33	31	29	26	23	19	
.40	0	0	0	0	0	0	2	13	47	116	211	298	354	328	245	172	127	100	83	69	59	51	45	41	37	33	31	29	26	23	19	
.50	0	0	0	0	0	0	0	1	9	34	89	170	255	341	303	225	161	120	96	76	64	54	47	42	38	34	31	30	27	24	19	
.75	0	0	0	0	0	0	0	1	4	14	41	89	152	270	305	268	207	155	118	87	70	57	48	44	39	35	32	30	27	24	19	
1.0	0	0	0	0	0	0	0	0	0	0	2	7	22	98	212	295	285	237	181	120	88	67	53	46	42	38	34	31	28	25	19	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	5	30	95	153	249	265	217	152	96	66	53	46	41	37	34	30	26	20	
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	18	59	125	221	245	182	105	69	54	47	42	38	32	28	22	
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	21	84	174	230	172	103	69	54	46	42	34	30	23	
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	13	56	157	217	163	101	68	53	46	37	31	25	

IA/P = 0.50 — * * * TC = 0.5 HR * * *

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.
0.0	0	0	0	0	0	2	26	89	17)	217	229	200	179	144	119	104	93	85	78	70	64	59	55	51	46	43	41	40	36	32	28	
.10	0	0	0	0	0	0	1	18	65	135	190	216	205	170	137	115	101	91	83	74	67	61	56	52	47	44	42	40	36	32	28	
.20	0	0	0	0	0	0	1	12	47	106	162	198	203	178	145	121	105	94	85	76	68	61	57	52	48	44	42	40	37	32	28	
.30	0	0	0	0	0	0	0	1	8	34	82	135	177	194	168	139	117	102	92	80	71	63	58	54	49	45	43	41	37	33	28	
.40	0	0	0	0	0	0	0	0	6	25	63	111	155	189	174	146	122	106	94	82	73	64	58	54	50	45	43	41	37	33	28	
.50	0	0	0	0	0	0	0	0	4	18	48	90	133	184	177	152	128	110	97	84	74	65	59	55	50	45	43	41	38	33	25	
.75	0	0	0	0	0	0	0	0	1	7	22	47	80	142	169	164	144	124	108	91	79	68	61	56	51	47	44	42	38	34	28	
1.0	0	0	0	0	0	0	0	0	0	0	1	3	11	51	112	155	166	154	134	109	91	76	65	59	54	49	45	43	39	35	28	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	2	16	50	97	136	154	145	121	95	75	64	58	54	49	45	41	37	29	
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	47	86	134	146	125	94	75	64	58	53	49	42	39	31	
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	11	44	95	140	127	97	77	65	58	54	45	41	33	
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	29	86	135	122	95	76	65	58	49	43	35	

RAINFALL TYPE = II — * * * TC = 0.5 HR * * * — SHEET 5 OF 10

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC =0.75 HR * * *														IA/P = 0.10				
0.0	13	18	24	36	46	68	115	194	294	380	424	410	369	252	172	123	93	74	61	49	41	35	31	27	24	22	20	19	17	15	12	0
.10	13	17	23	34	42	59	97	162	250	337	395	405	381	279	191	135	100	79	65	51	42	36	31	28	25	22	21	19	17	15	12	0
.20	11	15	20	28	32	39	52	82	135	211	295	362	391	351	255	178	127	95	75	57	46	38	32	29	26	23	21	20	17	15	12	0
.30	11	14	19	26	30	36	47	70	113	179	256	326	379	360	277	196	140	103	80	60	48	38	33	29	26	23	21	20	18	15	12	0
.40	10	12	16	22	25	28	33	42	61	96	151	221	291	367	336	255	182	131	98	69	54	42	34	30	27	24	22	20	18	16	12	0
.50	9	12	16	21	24	27	31	39	53	82	128	190	258	358	343	274	200	144	106	74	56	43	35	30	27	24	22	20	15	16	12	0
.75	8	10	13	17	18	21	23	26	31	39	55	82	122	230	314	329	281	217	161	104	72	51	38	33	29	26	23	21	19	16	12	1
1.0	6	8	10	13	14	15	17	19	21	23	27	32	42	89	177	272	319	303	249	163	105	66	45	36	31	27	24	22	19	17	13	3
1.5	4	6	7	9	10	10	11	12	14	15	16	18	20	27	46	90	163	241	295	275	204	119	66	45	35	31	27	24	20	18	13	7
2.0	3	4	5	6	7	7	8	9	9	10	11	12	13	16	20	28	48	89	151	245	274	213	115	65	44	35	30	27	22	19	14	10
2.5	1	2	3	4	4	5	5	6	6	7	7	8	8	10	12	14	17	24	37	86	170	260	219	127	71	47	36	31	24	20	16	11
3.0	1	1	2	3	3	3	4	4	4	5	5	5	6	7	8	10	11	14	17	30	64	157	247	205	122	70	46	36	27	22	17	12
IA/P = 0.30														* * * TC =0.75 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	1	6	30	86	174	266	326	348	328	246	181	138	110	92	79	66	57	49	44	40	36	32	31	29	26	23	19	0
.10	0	0	0	0	0	1	4	22	65	137	223	292	329	303	228	170	131	106	89	73	61	52	46	41	37	33	31	29	26	23	19	0
.20	0	0	0	0	0	0	3	15	48	108	185	256	305	321	245	184	141	112	93	75	63	53	46	42	37	34	31	30	27	23	19	0
.30	0	0	0	0	0	0	2	11	36	84	151	221	277	308	260	199	152	120	98	78	65	54	47	42	38	34	31	30	27	23	19	0
.40	0	0	0	0	0	0	0	1	8	27	65	122	188	286	301	243	197	144	116	87	71	57	48	43	39	35	32	30	27	24	19	1
.50	0	0	0	0	0	0	0	1	6	20	50	98	158	263	292	254	200	155	122	91	74	59	49	44	40	35	32	30	27	24	19	1
.75	0	0	0	0	0	0	0	0	0	2	8	23	51	140	231	269	253	211	167	119	90	68	53	46	42	37	34	31	28	25	19	2
1.0	0	0	0	0	0	0	0	0	0	0	0	1	4	29	96	156	249	261	231	169	120	84	61	50	44	40	36	33	29	26	20	5
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	34	91	163	220	241	197	131	83	61	50	44	40	35	31	27	21	12
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	36	85	174	226	200	127	82	60	49	44	39	32	29	22	17
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	37	105	196	214	135	87	62	51	44	36	31	24	18
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	24	96	205	189	130	55	62	50	39	32	26	18
IA/P = 0.50														* * * TC =0.75 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	2	16	45	92	137	166	185	170	146	125	110	98	89	79	70	63	58	53	48	44	42	41	37	33	28	0
.10	0	0	0	0	0	0	0	1	11	34	73	115	149	180	163	141	122	107	96	84	74	65	59	54	50	45	43	41	38	33	28	0
.20	0	0	0	0	0	0	0	1	8	25	57	96	131	173	166	146	126	111	99	86	76	66	59	55	50	46	43	41	38	34	28	0
.30	0	0	0	0	0	0	0	0	1	5	16	44	79	143	170	160	141	122	108	92	81	69	61	56	52	47	44	42	38	34	28	1
.40	0	0	0	0	0	0	0	0	0	4	14	34	64	127	166	162	145	127	111	95	82	70	62	57	52	47	44	42	38	34	28	1
.50	0	0	0	0	0	0	0	0	0	0	2	10	26	82	133	162	157	140	123	103	88	75	64	58	53	49	45	43	39	35	28	2
.75	0	0	0	0	0	0	0	0	0	0	1	4	12	47	95	139	154	148	135	113	96	80	67	60	55	50	46	43	39	36	29	[illegible]
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	30	73	119	146	151	134	113	91	74	63	58	53	48	45	41	37	29	7
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	30	66	105	143	143	117	90	73	63	57	52	48	42	39	30	18
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	30	77	121	137	114	88	72	63	57	52	44	40	32	21
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	19	55	111	132	111	87	71	62	56	47	42	34	21
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	12	51	112	128	108	96	71	62	51	44	36	27

RAINFALL TYPE = II   * * * TC = 0.75 HR * * *   SHEET 6 OF 10

Source: USDA, SCS, TR55 (1986)

continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

HYDROGRAPH TIME (HOURS)

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0
IA/P = 0.10																* * * TC = 1.0 HR * * *												IA/P = 0.10			
0.0	11	15	20	29	35	47	72	112	168	231	289	329	357	313	239	175	133	103	83	63	50	40	33	29	26	23	21	20	17	15	12
.10	10	13	17	24	27	33	42	62	95	144	202	260	306	340	293	222	165	126	98	72	56	43	35	30	27	24	22	20	19	15	12
.20	10	13	17	23	26	30	38	54	82	123	176	232	281	332	303	238	179	136	105	76	59	45	35	30	27	24	22	20	18	16	12
.30	9	12	16	22	24	28	35	48	70	105	152	205	256	323	310	254	193	146	113	81	61	46	36	31	27	24	22	20	18	16	12
.40	8	11	14	19	21	23	27	32	42	61	91	132	181	276	318	294	237	181	138	95	70	51	39	32	28	25	23	21	18	16	12
.50	8	10	13	18	20	22	25	30	38	53	78	114	159	253	311	300	251	195	149	102	74	53	40	33	29	25	23	21	18	16	12
.75	7	8	11	14	16	17	19	21	25	30	38	53	76	146	228	284	293	256	208	143	99	66	46	36	31	27	24	22	19	17	13
1.0	5	7	8	11	12	13	14	16	17	19	22	25	31	57	111	188	256	286	272	208	144	90	56	41	33	29	26	23	20	17	13
1.5	4	5	6	8	8	9	10	11	12	13	14	15	17	22	33	59	107	171	231	268	235	157	88	56	41	33	29	25	21	18	14
2.0	2	3	4	5	5	6	6	7	7	8	9	9	10	12	15	19	27	44	78	157	231	252	167	96	59	42	34	29	23	20	15
2.5	1	2	2	3	4	4	4	5	5	6	6	7	7	8	10	12	15	19	27	58	120	214	241	159	94	59	42	34	26	21	16
3.0	0	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	10	12	14	22	44	113	214	231	152	91	58	42	29	23	17
IA/P = 0.30																* * * TC = 1.0 HR * * *												IA/P = 0.30			
0.0	0	0	0	0	1	4	16	42	83	137	195	243	271	292	227	178	143	117	98	79	66	55	47	42	38	34	31	30	27	23	19
.10	0	0	0	0	0	0	3	12	32	66	113	168	218	279	260	213	169	136	113	88	72	59	49	43	39	35	32	30	27	24	19
.20	0	0	0	0	0	0	2	9	24	52	93	143	193	271	271	225	180	145	119	92	75	60	50	44	39	35	32	30	27	24	19
.30	0	0	0	0	0	0	1	6	18	41	75	120	169	246	264	234	191	153	125	96	78	62	51	44	40	36	33	31	27	24	19
.40	0	0	0	0	0	0	0	1	4	14	32	61	100	190	251	259	222	181	146	109	86	67	53	46	41	37	33	31	28	25	19
.50	0	0	0	0	0	0	0	1	3	10	24	49	83	168	237	254	230	191	155	115	90	69	54	47	42	37	34	31	28	25	19
.75	0	0	0	0	0	0	0	0	0	1	4	12	25	76	150	213	239	228	198	149	112	82	61	50	44	39	35	32	29	26	20
1.0	0	0	0	0	0	0	0	0	0	0	0	1	2	15	51	113	182	226	234	197	150	104	72	56	47	42	38	34	30	27	20
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	18	51	104	162	220	210	158	102	71	56	47	42	37	31	23	22
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	121	187	209	152	100	70	55	47	41	34	29	23
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	32	87	171	199	146	98	69	54	46	37	31	24
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	62	158	192	151	103	73	56	41	34	26
IA/P = 0.50																* * * TC = 1.0 HR * * *												IA/P = 0.50			
0.0	0	0	0	0	0	0	1	7	21	42	71	101	126	160	154	138	123	110	100	87	77	67	60	55	50	46	43	41	38	34	28
.10	0	0	0	0	0	0	0	1	5	15	33	58	87	134	156	149	134	120	108	93	82	71	62	57	52	47	44	42	38	34	28
.20	0	0	0	0	0	0	0	1	4	12	26	48	74	123	153	153	137	123	111	95	84	72	63	57	52	47	44	42	38	34	28
.30	0	0	0	0	0	0	0	0	3	9	20	38	62	111	143	150	140	127	114	98	86	73	63	58	53	48	45	42	39	35	28
.40	0	0	0	0	0	0	0	0	0	2	6	16	31	75	120	145	148	137	123	106	91	77	66	59	54	49	45	43	39	35	29
.50	0	0	0	0	0	0	0	0	0	1	5	12	25	64	109	139	146	139	127	108	94	79	67	60	55	50	46	43	39	36	29
.75	0	0	0	0	0	0	0	0	0	0	2	5	12	39	78	115	136	140	134	117	101	84	70	62	56	51	47	44	40	36	29
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	26	59	96	125	139	133	117	97	78	66	59	54	49	46	41	37	29
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	9	26	54	86	123	133	119	95	77	66	59	54	49	43	39	31
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	25	64	104	129	116	93	76	65	58	53	45	41	33
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	10	34	84	125	117	96	78	66	59	49	43	35
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	32	89	122	114	94	77	66	53	45	37

RAINFALL TYPE = II * * * TC = 1.0 HR * * *

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC =1.25 HR * * *														IA/P = 0.10				
0.0	10	13	18	25	29	38	54	81	118	163	213	256	284	311	266	212	163	129	104	78	61	47	37	31	27	24	22	20	18	16	12	1
.10	10	13	17	23	27	34	47	69	102	143	189	234	267	297	274	226	175	138	111	82	64	48	38	31	27	24	22	20	18	16	12	1
.20	9	11	15	20	22	26	31	42	60	88	124	168	212	280	292	261	212	166	131	95	72	53	40	33	28	25	23	21	18	16	12	1
.30	8	11	14	19	21	24	29	38	53	76	108	148	190	263	288	268	224	177	140	101	76	55	41	34	29	25	23	21	18	16	12	2
.40	8	10	13	18	20	23	27	34	46	66	94	130	170	245	282	273	235	188	149	107	80	58	42	34	29	26	23	21	19	16	12	2
.50	7	9	12	16	17	19	22	25	31	41	58	82	114	190	256	279	262	222	178	127	93	65	46	36	31	27	24	22	19	17	13	2
.75	6	8	10	14	15	17	19	21	25	31	41	56	78	139	207	254	265	245	208	152	110	75	51	39	32	28	25	22	19	17	13	3
1.0	5	6	8	10	11	13	14	15	17	19	22	26	33	60	109	173	230	261	255	208	153	100	64	46	36	30	26	24	20	18	13	5
1.5	3	4	5	7	7	8	9	9	10	11	12	13	15	19	27	45	79	130	186	247	239	180	108	68	48	37	31	27	22	19	14	10
2.0	2	3	4	5	6	6	7	7	8	8	9	10	11	13	16	22	35	59	98	171	236	236	156	95	62	44	35	30	23	20	15	11
2.5	1	2	2	3	4	4	4	5	5	5	6	6	7	8	10	12	14	19	28	58	114	197	226	163	102	65	46	36	26	21	16	11
3.0	0	1	1	2	2	2	2	3	3	3	4	4	4	5	6	7	9	10	13	19	35	88	184	218	169	109	70	49	31	24	18	12
IA/P = 0.30														* * * TC =1.25 HR * * *														IA/P = 0,30				
0.0	0	0	0	0	0	2	9	25	50	86	130	174	208	253	235	201	164	136	115	92	76	61	51	44	39	35	32	30	27	24	19	1
.10	0	0	0	0	0	0	1	6	19	40	71	110	153	217	247	227	191	157	131	103	84	66	53	46	41	36	33	31	28	24	19	2
.20	0	0	0	0	0	0	1	4	14	31	58	93	133	202	239	231	199	165	138	108	87	68	55	47	41	37	33	31	28	25	19	2
.30	0	0	0	0	0	0	0	1	3	10	24	46	77	152	210	236	222	190	158	122	97	74	58	49	43	38	34	32	28	25	20	3
.40	0	0	0	0	0	0	0	0	2	8	19	37	64	134	196	232	225	198	166	127	101	77	59	50	43	38	35	32	28	25	20	3
.50	0	0	0	0	0	0	0	0	0	2	6	14	30	82	151	206	228	217	189	146	113	85	64	52	45	40	36	33	29	26	20	5
.75	0	0	0	0	0	0	0	0	0	1	2	7	15	49	105	164	205	218	205	166	129	95	69	55	47	41	37	33	29	26	20	6
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	9	32	77	134	185	214	203	166	120	83	63	52	45	39	35	30	27	21	10
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	33	72	121	184	203	171	117	82	62	51	44	39	32	29	22	15
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	67	132	194	174	123	86	64	52	45	35	31	24	18	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	13	46	121	187	166	119	84	63	52	39	32	25	18	
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	44	129	180	160	116	83	63	44	35	27	18	
IA/P = 0.50														* * * TC =1.25 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	1	5	13	26	44	68	91	125	142	142	128	117	107	94	83	72	63	57	52	47	44	42	38	34	28	2
.10	0	0	0	0	0	0	0	0	3	10	20	36	57	100	129	140	136	125	114	100	88	76	65	59	54	49	45	43	39	35	29	3
.20	0	0	0	0	0	0	0	0	2	7	16	30	48	90	122	139	139	127	117	102	90	77	66	60	54	49	45	43	39	35	29	3
.30	0	0	0	0	0	0	0	0	0	2	5	12	24	59	98	126	137	134	125	109	96	82	69	61	56	51	46	44	40	36	29	4
.40	0	0	0	0	0	0	0	0	0	1	4	10	19	51	89	119	134	136	127	112	98	83	70	62	56	51	47	44	40	36	29	5
.50	0	0	0	0	0	0	0	0	0	1	3	7	15	43	79	112	131	135	129	114	100	85	71	63	57	52	47	44	40	36	29	6
.75	0	0	0	0	0	0	0	0	0	0	1	3	15	39	71	102	123	130	125	112	94	78	67	60	54	49	46	41	37	29	9	
1.0	0	0	0	0	0	0	0	0	0	0	0	1	4	17	40	71	101	121	129	121	103	84	71	62	56	51	47	42	38	30	13	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	51	92	119	125	105	86	72	63	57	52	44	40	32	23
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	35	72	112	122	103	85	71	63	56	47	42	34	26
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	24	66	111	119	101	83	71	62	51	44	36	27
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	23	71	110	116	99	82	70	55	46	37	27

RAINFALL TYPE = II   * * * TC = 1.25 HR * * *   SHEET 8 OF 10

Source: USDA, SCS, TR55 (1986)

continued: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26
IA/P = 0.10														* * * TC =1.5 HR * * *														IA/P = 0.10				
0.0	9	11	15	21	25	31	41	58	82	112	147	184	215	255	275	236	198	159	129	98	76	57	43	35	30	25	23	21	18	16	12	
.10	9	10	13	18	20	23	28	37	51	72	98	131	166	226	265	254	226	187	151	113	86	63	46	37	31	26	23	21	19	16	13	
.20	8	10	13	17	19	22	26	33	45	63	87	116	149	212	259	259	233	197	160	119	90	66	48	38	32	27	24	22	19	16	13	
.30	7	9	12	16	18	21	24	30	40	55	76	103	134	197	244	255	238	206	169	125	95	68	49	38	32	27	24	22	19	17	13	
.40	7	8	11	14	15	17	19	23	28	36	49	67	91	151	208	247	252	230	196	146	109	77	54	41	34	29	25	22	19	17	13	
.50	6	8	10	13	15	16	18	21	26	33	43	59	80	136	194	238	249	235	204	154	115	81	56	42	34	29	25	23	20	17	13	
.75	5	7	8	11	12	13	14	16	18	21	25	32	42	76	125	179	222	240	233	193	148	102	67	48	38	32	27	24	20	18	13	
1.0	4	5	7	8	9	10	11	12	13	14	16	18	22	34	59	101	152	201	236	230	193	135	86	59	44	35	30	26	21	18	14	
1.5	3	4	5	6	6	7	8	8	9	10	11	12	13	16	22	34	58	95	141	203	226	197	131	84	58	43	35	29	23	20	15	1
2.0	1	2	3	4	4	5	5	6	6	7	7	8	9	10	12	16	22	34	56	110	172	218	187	126	82	57	43	34	25	21	16	1
2.5	1	1	2	2	3	3	3	4	4	4	5	5	6	7	8	9	11	14	18	34	69	141	210	190	133	87	60	44	30	23	17	1
3.0	0	0	1	1	2	2	2	2	3	3	3	3	4	5	5	6	8	9	11	16	27	66	149	204	181	128	85	58	35	25	18	1
IA/P = 0.30														* * * TC =1.5 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	0	1	6	15	31	53	80	112	144	193	225	208	156	157	134	103	89	70	56	48	42	37	34	31	28	25	20	
.10	0	0	0	0	0	0	1	4	12	25	43	68	97	157	198	219	203	178	151	120	98	77	60	50	44	38	35	32	28	25	20	
.20	0	0	0	0	0	0	0	1	3	9	19	35	57	114	168	201	213	196	171	135	108	84	64	53	46	40	36	33	29	26	20	
.30	0	0	0	0	0	0	0	1	2	7	15	29	48	100	155	193	210	200	177	140	113	87	66	54	46	41	36	33	29	26	20	
.40	0	0	0	0	0	0	0	0	2	5	12	23	39	87	141	184	207	202	182	146	117	89	68	55	47	41	36	33	29	26	20	
.50	0	0	0	0	0	0	0	0	0	1	4	9	18	51	101	153	190	205	197	164	131	99	73	58	49	43	38	34	30	26	20	
.75	0	0	0	0	0	0	0	0	0	0	2	4	9	30	68	116	160	189	197	179	147	110	80	62	52	45	39	35	30	27	21	
1.0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	20	49	92	138	175	195	178	137	97	72	57	48	42	37	31	28	21	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	21	47	55	145	187	178	133	95	71	57	48	42	34	29	23	1
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	13	45	97	162	180	138	99	74	58	49	38	32	25	1
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	31	89	161	174	133	97	72	58	42	34	26	1
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	29	98	160	169	129	95	71	48	37	28	1
IA/P = 0.50														* * * TC =1.5 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	0	3	8	16	27	42	59	92	115	128	130	121	112	100	90	78	67	60	55	50	46	43	39	35	29	
.10	0	0	0	0	0	0	0	2	6	12	22	35	51	84	110	125	128	123	114	102	91	79	68	61	55	50	46	43	39	35	29	
.20	0	0	0	0	0	0	0	0	1	4	10	18	29	60	91	114	126	128	120	108	97	83	71	63	57	52	47	44	40	36	29	
.30	0	0	0	0	0	0	0	0	1	3	8	14	24	52	83	108	123	126	122	110	98	85	72	63	57	52	48	44	40	36	29	
.40	0	0	0	0	0	0	0	0	0	1	2	6	12	31	60	90	112	124	126	116	104	90	75	66	59	54	49	45	41	37	29	
.50	0	0	0	0	0	0	0	0	0	0	2	4	9	26	53	83	106	121	125	118	106	91	77	67	60	54	49	46	41	37	29	
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	16	36	62	88	104	119	122	112	97	81	69	62	56	51	47	42	38	30	1
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	26	47	75	98	118	121	108	90	76	66	59	54	49	43	39	31	1
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	11	25	45	80	107	118	106	89	75	65	59	53	45	41	32	[illegible]
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	11	32	63	100	115	104	87	74	65	58	48	42	36	[illegible]	
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	48	94	113	105	89	76	66	53	45	36	[illegible]		
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	54	96	111	103	88	75	58	48	38	[illegible]			

RAINFALL TYPE = II * * * TC = 1.5 HR * * * SHEET 9 OF 10

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC =2.0 HR * * *														IA/P = 0.10				
0.0	7	9	12	16	18	21	27	36	49	64	82	104	127	171	201	226	208	193	171	132	105	79	58	45	36	30	26	23	20	17	13	3
.10	6	8	10	14	15	17	20	25	33	43	57	74	94	139	179	204	218	205	188	150	118	88	63	48	38	32	27	24	20	17	13	4
.20	6	8	10	13	14	16	19	23	29	39	51	66	84	128	169	198	213	207	192	157	123	91	65	49	39	33	28	24	20	17	13	4
.30	6	7	9	12	14	15	18	21	27	35	45	59	76	117	159	191	211	208	196	163	128	95	68	51	40	33	28	25	20	18	13	4
.40	5	6	8	11	12	13	15	17	20	24	31	41	53	87	128	167	197	209	205	180	145	106	75	55	43	35	30	26	21	18	14	5
.50	5	6	8	10	11	13	14	16	18	22	28	37	48	78	118	158	190	208	208	185	151	111	77	57	44	36	30	26	21	18	14	5
.75	4	6	7	9	10	11	12	13	15	18	22	27	35	58	91	129	164	191	202	194	167	125	87	63	48	38	32	27	22	18	14	6
1.0	3	4	6	7	8	8	9	10	11	12	14	16	18	28	46	74	110	147	178	201	193	156	108	76	56	43	35	30	23	19	14	8
1.5	2	3	3	5	5	5	6	6	7	8	8	9	10	12	16	23	36	57	86	137	178	195	160	113	79	58	45	36	26	21	16	11
2.0	1	2	2	3	3	4	4	4	5	5	6	6	7	8	10	12	16	23	35	67	112	169	190	154	110	78	57	44	30	23	17	11
2.5	0	1	1	2	2	2	3	3	3	4	4	4	5	6	7	8	9	12	16	28	52	105	170	185	149	107	76	56	35	26	18	12
3.0	0	0	1	1	1	1	1	2	2	2	2	3	3	3	4	5	6	7	8	12	18	41	99	161	180	152	112	80	45	30	19	12
IA/P = 0.30														* * * TC =2.0 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	0	1	3	8	15	25	38	54	74	115	148	168	185	170	159	131	110	89	70	57	49	42	38	34	29	26	20	5
.10	0	0	0	0	0	0	0	2	6	12	21	32	47	85	124	153	169	180	168	145	120	96	75	60	51	44	39	35	30	26	20	6
.20	0	0	0	0	0	0	0	2	4	10	17	27	41	75	114	146	165	175	170	149	124	99	76	62	52	45	39	35	30	27	21	6
.30	0	0	0	0	0	0	0	0	1	3	7	14	23	49	86	122	151	170	174	160	136	107	82	66	54	47	41	37	31	27	21	8
.40	0	0	0	0	0	0	0	0	1	2	6	11	19	43	77	113	144	165	173	163	140	111	85	67	55	47	41	37	31	27	21	8
.50	0	0	0	0	0	0	0	0	1	2	4	9	16	37	68	104	136	160	171	165	144	114	87	69	56	48	42	37	31	27	21	9
.75	0	0	0	0	0	0	0	0	0	0	1	2	5	15	34	62	96	127	152	167	160	132	100	77	62	52	45	40	32	28	22	11
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	10	24	48	79	111	150	166	153	118	90	71	58	49	43	34	29	23	14
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	24	45	58	130	161	148	115	88	70	57	48	37	31	24	17
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	32	68	122	157	143	113	87	68	56	42	34	26	18
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	16	51	114	153	144	116	89	70	49	38	27	19
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	15	59	118	150	140	113	88	57	42	29	19
IA/P = 0.50														* * * TC =2.0 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	0	1	4	8	13	20	28	51	73	92	104	111	112	106	97	86	75	66	60	54	49	46	41	37	30	7
.10	0	0	0	0	0	0	0	1	3	6	11	17	24	45	68	87	101	109	112	107	98	88	76	67	60	55	50	46	41	37	30	8
.20	0	0	0	0	0	0	0	1	2	5	9	14	21	40	62	82	98	107	111	108	100	89	77	68	61	55	50	47	41	37	30	8
.30	0	0	0	0	0	0	0	0	0	2	4	7	12	26	46	67	86	100	108	111	104	93	80	70	63	57	52	48	42	38	30	10
.40	0	0	0	0	0	0	0	0	0	1	3	6	10	22	41	62	81	96	106	110	105	94	81	71	63	57	52	48	42	38	30	11
.50	0	0	0	0	0	0	0	0	0	0	1	2	4	13	27	46	67	85	99	110	108	98	85	74	66	59	54	49	43	39	31	13
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	7	18	33	52	71	88	104	108	102	89	77	68	61	55	50	44	39	31	15
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	13	25	43	62	87	103	108	97	84	73	65	59	53	45	41	32	20
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	12	24	48	74	99	106	95	83	72	64	58	48	43	34	25
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	17	37	69	99	104	94	82	72	64	52	45	36	27
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	8	27	65	95	102	95	83	73	58	49	38	28
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	5	32	68	95	101	93	82	64	52	40	28

RAINFALL TYPE = II * * * TC = 2.0 HR * * * SHEET 10 OF 10

Source: USDA, SCS, TR55 (1986)

Appendix B – Impervious Area Calculations

Urban Modifications B.1

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, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Tables 3-11 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 3-11 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 3-11.

If all the impervious are is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 3-11 are not applicable, use Figure B-1 to compute a composite CN. For example, Table 3-11 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 B-1 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Composite CN With Connected Impervious Area

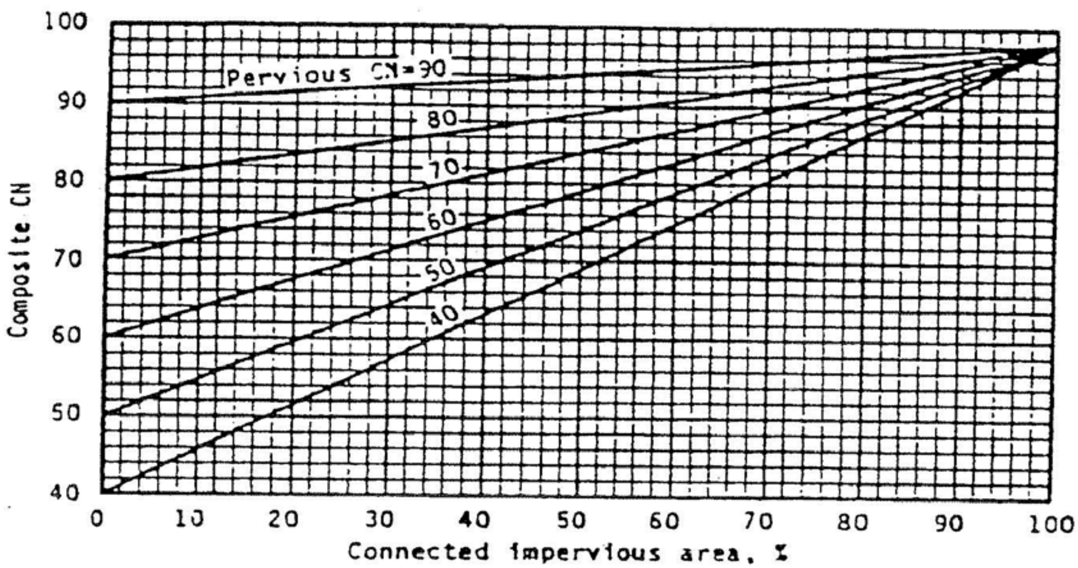
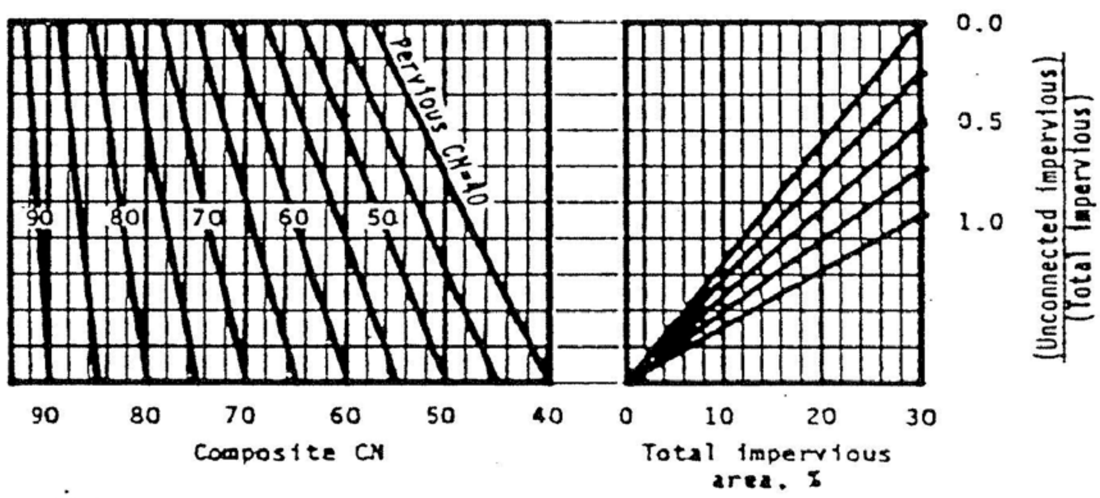


FIGURE B-1

Composite CN With Unconnected Impervious Area
(Total Impervious Area Less than 30%)



Source: USDA, SCS, TR55(1986)

FIGURE B-2

Urban Modifications (continued)

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 B-2 if total impervious area is less than 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of Figure B-2 with the percentage of total impervious area and the ratio of total unconnected impervious are 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 B-22 is 66. If all of the impervious area is connected, the resulting CN (from Figure B-1) would be 68.

Composite Curve Numbers B.2

When a drainage area has more than one land use, a 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 bust see 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 one presented in Table B-1.

Table B-1 Composite Curve Numbers Calculations

(1) Land Use	*(2) Curve Number*	*(3) Area*	*(4) % of Total Area*	*(5) Composite Curve Number (Col 2 x Col 4)*

The composite curve number for the total drainage area is then the sum of the composite curve numbers from column 5.

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 then'sub-basins should be developed and separate hydrographs developed and routed to the study point.

Appendix C – 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.)
0	0.0014	121	0.2678	242	2.0305
1	0.0029	122	0.2720	243	2.0340
2	0.0043	123	0.2763	244	2.0375
3	0.0058	124	0.2806	245	2.0410
4	0.0072	125	0.2850	246	2.0445
5	0.0087	126	0.2894	247	2.0479
6	0.0101	127	0.2939	248	2.0512
7	0.0116	128	0.2985	249	2.0546
8	0.0131	129	0.3031	250	2.0579
9	0.0146	130	0.3078	251	2.0611
10	0.0161	131	0.3125	252	2.0644
11	0.0176	132	0.3174	253	2.0676
12	0.0191	133	0.3223	254	2.0707
13	0.0206	134	0.3273	255	2.0739
14	0.0221	135	0.3323	256	2.0770
15	0.0237	136	0.3375	257	2.0800
16	0.0252	137	0.3427	258	2.0831
17	0.0267	138	0.3480	259	2.0861
18	0.0283	139	0.3534	260	2.0891
19	0.0299	140	0.3590	261	2.0928
20	0.0314	141	0.3646	262	2.0949
21	0.0330	142	0.3703	263	2.0979
22	0.0346	143	0.3761	264	2.1007
23	0.0362	144	0.3821	265	2.1036
24	0.0378	145	0.3882	266	2.1064
25	0.0394	146	0.3944	267	2.1092
26	0.0410	147	0.4007	268	2.1120
27	0.0426	148	0.4072	269	2.1147
28	0.0442	149	0.4138	270	2.1172
29	0.0459	150	0.4225	271	2.1196
30	0.0475	151	0.4313	272	2.1220

2-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
31	0.0492	152	0.4403	273	2.1244
32	0.0508	153	0.4495	274	2.1267
33	0.0525	154	0.4590	275	2.1291
34	0.0542	155	0.4687	276	2.1314
35	0.0559	156	0.4787	277	2.1337
36	0.0576	157	0.4889	278	2.1360
37	0.0594	158	0.4995	279	2.1382
38	0.0610	159	0.5104	280	2.1405
39	0.0627	160	0.5216	281	2.1427
40	0.0645	161	0.5332	282	2.1449
41	0.0662	162	0.5452	283	2.1471
42	0.0680	163	0.5577	284	2.1493
43	0.0698	164	0.5707	285	2.1515
44	0.0715	165	0.5879	286	2.1536
45	0.0733	166	0.6057	287	2.1558
46	0.0751	167	0.6243	288	2.1579
47	0.0769	168	0.6438	289	2.1600
48	0.0787	169	0.6642	290	2.1621
49	0.0806	170	0.6858	291	2.1641
50	0.0824	171	0.7086	292	2.1662
51	0.0843	172	0.7331	293	2.1682
52	0.0861	173	0.7655	294	2.1703
53	0.0880	174	0.8002	295	2.1723
54	0.0899	175	0.8444	296	2.1743
55	0.0918	176	0.8924	297	2.1763
56	0.0937	177	0.9458	298	2.1783
57	0.0956	178	1.0298	299	2.1802
58	0.0976	179	1.1138	300	2.1822
59	0.0995	180	1.1978	301	2.1841
60	0.1015	181	1.2818	302	2.1861
61	0.1034	182	1.3658	303	2.1880

2-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
62	0.1054	183	1.4162	304	2.1899
63	0.1074	184	1.4621	305	2.1918
64	0.1094	185	1.4983	306	2.1936
65	0.1114	186	1.5318	307	2.1955
66	0.1135	187	1.5631	308	2.1974
67	0.1155	188	1.5867	309	2.1992
68	0.1176	189	1.6089	310	2.2011
69	0.1197	190	1.6299	311	2.2029
70	0.1218	191	1.6498	312	2.2047
71	0.1239	192	1.6688	313	2.2065
72	0.1260	193	1.6870	314	2.2083
73	0.1282	194	1.7045	315	2.2101
74	0.1303	195	1.7178	316	2.2118
75	0.1325	196	1.7305	317	2.2136
76	0.1347	197	1.7428	318	2.2154
77	0.1369	198	1.7546	319	2.2171
78	0.1391	199	1.7660	320	2.2188
79	0.1413	200	1.7771	321	2.2206
80	0.1436	201	1.7878	322	2.2223
81	0.1459	202	1.7982	323	2.2240
82	0.1482	203	1.8083	324	2.2257
83	0.1505	204	1.8181	325	2.2274
84	0.1528	205	1.8273	326	2.2290
85	0.1552	206	1.8371	327	2.2307
86	0.1575	207	1.8462	328	2.2324
87	0.1599	208	1.8551	329	2.2340
88	0.1623	209	1.8638	330	2.2357
89	0.1647	210	1.8706	331	2.2373
90	0.1675	211	1.8771	332	2.2389
91	0.1703	212	1.8835	333	2.2405
92	0.1731	213	1.8898	334	2.2421

2-YEAR, 6-HOUR STORM
1-MINUTE TIME INCREMENT

TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
93	0.1759	214	1.8959	335	2.2437
94	0.1787	215	1.9020	336	2.2453
95	0.1816	216	1.9078	337	2.2469
96	0.1845	217	1.9136	338	2.2485
97	0.1874	218	1.9192	339	2.2501
98	0.1903	219	1.9249	340	2.2516
99	0.1933	220	1.9303	341	2.2532
100	0.1963	221	1.9357	342	2.2547
101	0.1993	222	1.9410	343	2.2563
102	0.2024	223	1.9462	344	2.2578
103	0.2055	224	1.9513	345	2.2593
104	0.2086	225	1.9563	346	2.2609
105	0.2117	226	1.9612	347	2.2624
106	0.2149	227	1.9661	348	2.2639
107	0.2181	228	1.9709	349	2.2654
108	0.2214	229	1.9756	350	2.2669
109	0.2246	230	1.9803	351	2.2684
110	0.2279	231	1.9849	352	2.2698
111	0.2313	232	1.9894	353	2.2713
112	0.2347	233	1.9939	354	2.2728
113	0.2381	234	1.9983	355	2.2742
114	0.2416	235	2.0026	356	2.5757
115	0.2451	236	2.0069	357	2.2771
116	0.2486	237	2.0112	358	2.2786
117	0.2522	238	2.0154	359	2.2800
118	0.2558	239	2.0195	360	2.2800
119	0.2595	240	2.0232		
120	0.2636	241	2.0269		

10-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
0	0.0022	121	0.4237	242	3.3271
1	0.0044	122	0.4311	243	3.3329
2	0.0066	123	0.4386	244	3.3387
3	0.0089	124	0.4461	245	3.3443
4	0.0111	125	0.4538	246	3.3499
5	0.0134	126	0.4615	247	3.3555
6	0.0156	127	0.4694	248	3.3610
7	0.0179	128	0.4774	249	3.3665
8	0.0202	129	0.4855	250	3.3718
9	0.0225	130	0.4936	251	3.3772
10	0.0248	131	0.5020	252	3.3824
11	0.0271	132	0.5104	253	3.3876
12	0.0294	133	0.5190	254	3.3928
13	0.0317	134	0.5276	255	3.3979
14	0.0341	135	0.5365	256	3.4030
15	0.0364	136	0.5454	257	3.4080
16	0.0388	137	0.5546	258	3.4129
17	0.0412	138	0.5638	259	3.4178
18	0.0436	139	0.5733	260	3.4227
19	0.0460	140	0.5829	261	3.4275
20	0.0484	141	0.5926	262	3.4323
21	0.0508	142	0.6026	263	3.4370
22	0.0533	143	0.6127	264	3.4417
23	0.0557	144	0.6230	265	3.4464
24	0.0582	145	0.6336	266	3.4510
25	0.0606	146	0.6444	267	3.4556
26	0.0631	147	0.6554	268	3.4601
27	0.0636	148	0.6666	269	3.4646
28	0.0681	149	0.6781	270	3.4684
29	0.0707	150	0.6936	271	3.4721
30	0.0732	151	0.7094	272	3.4759

10-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
31	0.0758	152	0.7255	273	3.4795
32	0.0783	153	0.7420	274	3.4832
33	0.0809	154	0.7589	275	3.4868
34	0.0835	155	0.7763	276	3.4904
35	0.0861	156	0.7940	277	3.4940
36	0.0887	157	0.8122	278	3.4975
37	0.0914	158	0.8310	279	3.5011
38	0.0940	159	0.8502	280	3.5045
39	0.0967	160	0.8701	281	3.5080
40	0.0994	161	0.8906	282	3.5114
41	0.1021	162	0.9118	283	3.5148
42	0.1048	163	0.9338	284	3.5182
43	0.1075	164	0.9566	285	3.5216
44	0.1102	165	0.9876	286	3.5249
45	0.1130	166	1.0198	287	3.5282
46	0.1158	167	1.0531	288	3.5315
47	0.1186	168	1.0878	289	3.5347
48	0.1214	169	1.1240	290	3.5380
49	0.1242	170	1.1620	291	3.5412
50	0.1271	171	1.2021	292	3.5444
51	0.1299	172	1.2445	293	3.5475
52	0.1328	173	1.2976	294	3.5507
53	0.1357	174	1.3542	295	3.5538
54	0.1386	175	1.4281	296	3.5569
55	0.1416	176	1.5067	297	3.5600
56	0.1445	177	1.5919	298	3.5630
57	0.1475	178	1.7099	299	3.5661
58	0.1505	179	1.8279	300	3.5691
59	0.1535	180	1.9459	301	3.5721
60	0.1565	181	2.0639	302	3.5751
61	0.1596	182	2.1819	303	3.5781

10-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
62	0.1627	183	2.2635	304	3.5810
63	0.1657	184	2.3395	305	3.5839
64	0.1689	185	2.3982	306	3.5868
65	0.1720	186	2.4529	307	3.5897
66	0.1751	187	2.5045	308	3.5926
67	0.1783	188	2.5458	309	3.5954
68	0.1815	189	2.5847	310	3.5983
69	0.1847	190	2.6218	311	3.6011
70	0.1880	191	2.6572	312	3.6039
71	0.1913	192	2.6912	313	3.6067
72	0.1945	193	2.7240	314	3.6094
73	0.1979	194	2.7556	315	3.6122
74	0.2012	195	2.7788	316	3.6149
75	0.2046	196	2.8012	317	3.6177
76	0.2080	197	2.8227	318	3.6204
77	0.2114	198	2.8436	319	3.6231
78	0.2148	199	2.8638	320	3.6257
79	0.2183	200	2.8833	321	3.6284
80	0.2218	201	2.9023	322	3.6310
81	0.2253	202	2.9208	323	3.6337
82	0.2289	203	2.9388	324	3.6363
83	0.2325	204	2.9563	325	3.6389
84	0.2361	205	2.9734	326	3.6415
85	0.2397	206	2.9901	327	3.6440
86	0.2434	207	3.0065	328	3.6466
87	0.2471	208	3.0224	329	3.6492
88	0.2508	209	3.0381	330	3.6517
89	0.2546	210	3.0497	331	3.6542
90	0.2591	211	3.0611	332	3.6567
91	0.2636	212	3.0722	333	3.6592
92	0.2681	213	3.0831	334	3.6617

10-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
93	0.2727	214	3.0937	335	3.6642
94	0.2774	215	3.1041	336	3.6666
95	0.2820	216	3.1144	337	3.6691
96	0.2868	217	3.1244	338	3.6715
97	0.2915	218	3.1343	339	3.6739
98	0.2963	219	3.1440	340	3.6763
99	0.3012	220	3.1535	341	3.6787
100	0.3060	221	3.1628	342	3.6811
101	0.3110	222	3.1720	343	3.6835
102	0.3160	223	3.1811	344	3.6858
103	0.3210	224	3.1900	345	3.6882
104	0.3261	225	3.1987	346	3.6905
105	0.3312	226	3.2073	347	3.6929
106	0.3364	227	3.2158	348	3.6952
107	0.3416	228	3.2242	349	3.6975
108	0.3469	229	3.2325	350	3.6998
109	0.3523	230	3.2406	351	3.7021
110	0.3577	231	3.2486	352	3.7044
111	0.3632	232	3.2565	353	3.7066
112	0.3687	233	3.2644	354	3.7089
113	0.3743	234	3.2721	355	3.7111
114	0.3799	235	3.2797	356	3.7134
115	0.3856	236	3.2872	357	3.7156
116	0.3914	237	3.2946	358	3.7178
117	0.3973	238	3.3020	359	3.72
118	0.4032	239	3.3092	360	3.72
119	0.4092	240	3.3152		
120	0.4164	241	3.3212		

10-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
0	0.0022	121	0.4237	242	3.3271
1	0.0044	122	0.4311	243	3.3329
2	0.0066	123	0.4386	244	3.3387
3	0.0089	124	0.4461	245	3.3443
4	0.0111	125	0.4538	246	3.3499
5	0.0134	126	0.4615	247	3.3555
6	0.0156	127	0.4694	248	3.3610
7	0.0179	128	0.4774	249	3.3665
8	0.0202	129	0.4855	250	3.3718
9	0.0225	130	0.4936	251	3.3772
10	0.0248	131	0.5020	252	3.3824
11	0.0271	132	0.5104	253	3.3876
12	0.0294	133	0.5190	254	3.3928
13	0.0317	134	0.5276	255	3.3979
14	0.0341	135	0.5365	256	3.4030
15	0.0364	136	0.5454	257	3.4080
16	0.0388	137	0.5546	258	3.4129
17	0.0412	138	0.5638	259	3.4178
18	0.0436	139	0.5733	260	3.4227
19	0.0460	140	0.5829	261	3.4275
20	0.0484	141	0.5926	262	3.4323
21	0.0508	142	0.6026	263	3.4370
22	0.0533	143	0.6127	264	3.4417
23	0.0557	144	0.6230	265	3.4464
24	0.0582	145	0.6336	266	3.4510
25	0.0606	146	0.6444	267	3.4556
26	0.0631	147	0.6554	268	3.4601
27	0.0636	148	0.6666	269	3.4646
28	0.0681	149	0.6781	270	3.4684
29	0.0707	150	0.6936	271	3.4721
30	0.0732	151	0.7094	272	3.4759

50-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
31	0.1074	152	0.9944	273	4.5804
32	0.1110	153	1.0159	274	4.5856
33	0.1147	154	1.0380	275	4.5907
34	0.1183	155	1.0606	276	4.5957
35	0.1220	156	1.0838	277	4.6008
36	0.1257	157	1.1076	278	4.6057
37	0.1295	158	1.1320	279	4.6107
38	0.1332	159	1.1571	280	4.6156
39	0.1370	160	1.1830	281	4.6204
40	0.1408	161	1.2098	282	4.6253
41	0.1446	162	1.2374	283	4.6301
42	0.1484	163	1.2660	284	4.6348
43	0.1523	164	1.2957	285	4.6395
44	0.1562	165	1.3363	286	4.6442
45	0.1601	166	1.3783	287	4.6489
46	0.1640	167	1.4219	288	4.6535
47	0.1679	168	1.4672	289	4.6581
48	0.1719	169	1.5145	290	4.6626
49	0.1759	170	1.5640	291	4.6671
50	0.1799	171	1.6161	292	4.6716
51	0.1839	172	1.6714	293	4.6761
52	0.1880	173	1.7400	294	4.6805
53	0.1921	174	1.8130	295	4.6849
54	0.1962	175	1.9086	296	4.6893
55	0.2004	176	2.0101	297	4.6936
56	0.2045	177	2.1198	298	4.6980
57	0.2087	178	2.2698	299	4.7022
58	0.2129	179	2.4198	300	4.7065
59	0.2172	180	2.5698	301	4.7107
60	0.2215	181	2.7198	302	4.7149
61	0.2258	182	2.8698	303	4.7191

50-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
62	0.2301	183	2.9751	304	4.7233
63	0.2344	184	3.0734	305	4.7274
64	0.2388	185	3.1490	306	4.7315
65	0.2432	186	3.2197	307	4.7356
66	0.2477	187	3.2864	308	4.7396
67	0.2522	188	3.3401	309	4.7437
68	0.2567	189	3.3908	310	4.7477
69	0.2612	190	3.4392	311	4.7516
70	0.2658	191	3.4854	312	4.7556
71	0.2704	192	3.5298	313	4.7595
72	0.2750	193	3.5726	314	4.7635
73	0.2797	194	3.6139	315	4.7673
74	0.2844	195	3.6442	316	4.7712
75	0.2891	196	3.6733	317	4.7751
76	0.2939	197	3.7014	318	4.7789
77	0.2987	198	3.7286	319	4.7827
78	0.3036	199	3.7549	320	4.7865
79	0.3084	200	3.7804	321	4.7902
80	0.3134	201	3.8052	322	4.7940
81	0.3183	202	3.8293	323	4.7977
82	0.3233	203	3.8528	324	4.8014
83	0.3283	204	3.8757	325	4.8051
84	0.3334	205	3.8980	326	4.8087
85	0.3385	206	3.9198	327	4.8124
86	0.3437	207	3.9411	328	4.8160
87	0.3489	208	3.9620	329	4.8196
88	0.3542	209	3.9824	330	4.8232
89	0.3594	210	3.9981	331	4.8268
90	0.3655	211	4.0134	332	4.8303
91	0.3717	212	4.0283	333	4.8338
92	0.3778	213	4.0430	334	4.8374

50-YEAR, 6-HOUR STORM 1-MINUTE TIME INCREMENT					
TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)	TIME (MIN.)	ACCUMULATED RAINFALL (IN.)
93	0.3841	214	4.0574	335	4.8409
94	0.3903	215	4.0714	336	4.8443
95	0.3967	216	4.0852	337	4.8478
96	0.4031	217	4.0988	338	4.8512
97	0.4095	218	4.1121	339	4.8547
98	0.4160	219	4.1252	340	4.8581
99	0.4226	220	4.1380	341	4.8615
100	0.4292	221	4.1507	342	4.8649
101	0.4359	222	4.1631	343	4.8682
102	0.4427	223	4.1753	344	4.8716
103	0.4495	224	4.1873	345	4.8749
104	0.4564	225	4.1992	346	4.8782
105	0.4634	226	4.2109	347	4.8815
106	0.4704	227	4.2224	348	4.8848
107	0.4775	228	4.2337	349	4.8881
108	0.4847	229	4.2449	350	4.8913
109	0.4919	230	4.2559	351	4.8946
110	0.4992	231	4.2668	352	4.8978
111	0.5066	232	4.2775	353	4.9010
112	0.5141	233	4.2881	354	4.9042
113	0.5217	234	4.2985	355	4.9074
114	0.5293	235	4.3089	356	4.9106
115	0.5370	236	4.3191	357	4.9137
116	0.5449	237	4.3291	358	4.9169
117	0.5528	238	4.3391	359	4.9200
118	0.5608	239	4.3489	360	4.9200
119	0.5689	240	4.3571		
120	0.5787	241	4.3651		

CHAPTER 4

OPEN CHANNEL HYDRAULICS

Chapter Table of Contents

4.1 Overview

Introduction 4.1.1

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.

Channel Linings 4.1.2

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.

Vegetative 4.1.2.1

Vegetation is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidated the soil mass of the bed, checks 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*
6. *Velocities*

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of a healthy growth 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 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.

Flexible 4.1.2.2

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 higher roughness values create larger cross sections.

Rigid
4.1.2.3

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 destroyed 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 conditions when a vegetative lining alone would be appropriate, a small concrete pilot channel could be used to handle the continuous low flows. Vegetation could then be maintained for handling larger flows.

4.2 Symbols and Definitions

Symbol Table

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in open channel publications.

Table 4-1
SYMBOLS AND DEFINITIONS

Symbol	Definition	Units
α	Energy Coefficient	-
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_g	Specific weight correction factor	-
D or d	Depth of flow	ft
d	Stone diameter	ft
Δd	Superelevation of the water surface profile due to a bend	ft
d_x	Diameter of stone for which x percent, by weight, of the gradation is finer	ft
E	Specific energy	ft
Fr	Froude Number	-
g	Acceleration due to gravity	ft/s^2
h_e	Eddy head loss	ft
h_L	Headloss	ft
K	Channel Conveyance	-
k_e	Eddy head loss coefficient	ft
K_T	Trapezoidal open channel conveyance factor	-
L	Length of channel	ft
L_P	Length of downstream protection	ft
M	Side slope, M to 1	-
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius of flow	ft
R_c	Mean radius of the bend	ft
S	Slope	ft/ft
S_f	Friction slope	ft/ft
S_o	Channel bottom slope	ft/ft
SW_s	Specific weight of stone	lbs/ft^3
T	Top width of water surface	ft
V or v	Velocity of flow	ft/s
W	Channel top width	ft
w	Stone weight	lbs
y	Depth of flow	ft
y_c	Critical depth	ft
y_n	Normal depth	ft
z	Critical flow section factor	-
Z	Vertical distance from datum	ft

4.3 Design Criteria

General Criteria 4.3.1

The following criteria shall be used for open channel design:

1. *Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1.*

2. *Channel side slopes shall be stable throughout the entire length and slope shall be a maximum of 2:1.*

3. *Superelevation of the water surface at horizontal curves shall be accounted for by increased freeboard.*

4. *Transition to 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 for superelevation of water surface at horizontal curves (see NC Erosion and Sediment Control Planning and Design Manual, Section 8.05.21).*

Return Period 4.3.2

Open channel drainage systems shall be designed to handle a 10-year design storm. The 100-year design storm shall be routed through the channel system to determine if a 100 + 1 flood study is required as described in Section 4.3.3.

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

100 + 1 Flood Analysis 4.3.3

All streams in the Town of Waxhaw and Union County which drain more than one square mile (640 acres) are regulated by Floodway Ordinances, which restricts development in those flood plains. However, streams 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. *For drainage systems which are expected to carry 150 cfs or more for the 100-year storm, the 100-year + 1 elevation and flood limits shall be shown on the recorded map of the area for residential property as further described in the Subdivision Ordinance.*

2. For portions of the drainage system, which are expected to carry between 50 and 150 cfs, the City/County Engineering Department can require that the 100-year + 1 elevation be shown on the recorded map if the engineering analysis indicates that or of the following conditions are present.

- The 100-year + 1 foot line would exceed the set-back limits
- The estimated runoff would create a hazard for adjacent properties or residents.
- The flood limits would be of such magnitude that adjacent residents should be Informed of these limits.

Velocity Limitations 4.3.4

The final design of artificial open channels should be consistent with the velocity limitations for the selected lining. For design information on channel linings, refer to the "North Carolina Erosion and Sediment Control Planning and Design Manual". Additional sources are listed in the bibliography for analysis and design criteria for the channel stabilization.

4.4 Hydraulic Terms

Energy of Flow 4.4.1

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 the 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 4-1 illustrates open channel energy concepts and equation 4.1 is the energy equation.

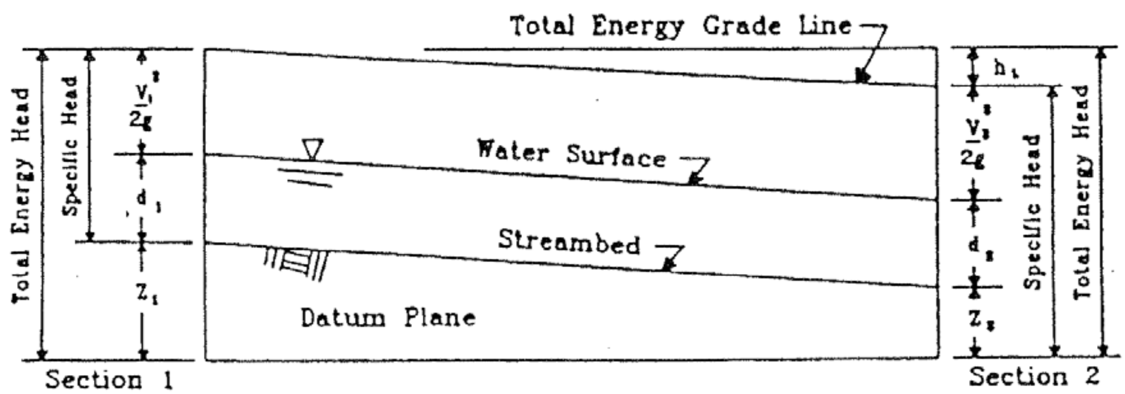


Figure 4-1 Energy In Open Channel Flow

$$d_1 + V_1^2/2g + Z_1 = d_2 + V_2^2/2g + Z_2 + h_L \qquad (4.1)$$

Where:

d = depth of flow above streambed (ft)
V = mean velocity of flow (ft/s)
Z = vertical distance form 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 (hL = S x L). Under uniform flow, the energy line is parallel to the water surface and stream bed.

Steady & Unsteady 4.4.2

Flow in open channels is classified as either 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.

Uniform and Non-Uniform 4.4.3

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.

Froude Number 4.4.4

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

$$\mathbf{Fr = v / (gD)^{1/2}} \qquad \mathbf{(4.2)}$$

Where: *v = mean velocity of flow (ft/s)*
g = acceleration due to gravity (32.2 ft/S2)
D = hydraulic depth (ft) – defined as the cross sectional area of water normal to the direction of channel flow divided by free surface width.

Critical Flow 4.4.5

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 4-2. 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.

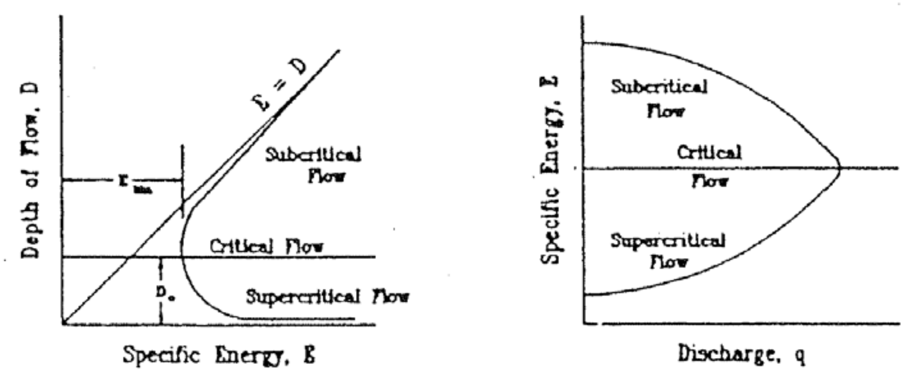


Figure 4-2 Definition Sketch of Specific Energy

Subcritical Flow 4.4.6

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

Supercritical Flow 4.4.7

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.

4.5 Manning's n Values

General Considerations 4.5.1

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 judgement must be exercised in the selection process.

Selection 4.5.2

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, type of vegetation, and how the vegetation affects the flow through the channel reach.*

3. *Channel shape 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.*

4. *A significant increase in the value of n is possible is sever 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 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 evaluation 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 freshly constructed channel are rarely appropriate as a basis for design calculations.

Manning's n Values 4.5.3

Recommended Manning's n values for artificial and natural channels are given i Table 4-2 shown below.

Table 4-2 Recommended Manning's n Values

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry; plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	.....	0.500

Table 4-2 (continued)

Type of channel and description	Minimum	Normal	Maximum
C. Excavated or Dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. Natural Streams			
D-1. 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

Table 4-2 (continued)

Type of channel and description	Minimum	Normal	Maximum
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
D-2. 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 crops	0.025	0.035	0.045
3. Mature field crops	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
D-3. Major streams (top width at flood stage >100 ft). The *n* value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	.....	0.060
b. Irregular and rough section	0.035	.....	0.100

Reference: Chow, V.T., ed. 1959, Open-Channel Hydraulics

4.6 Best Hydraulic Section

Introduction 4.6.1

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. A minimum freeboard of 6" must be provided.

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.

Equations 4.6.2

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

$$P = 4y\,(1 + M^2)^{1/2} - 2My \qquad (4.3)$$

(Figure 4-3 below shows a definition of variables)

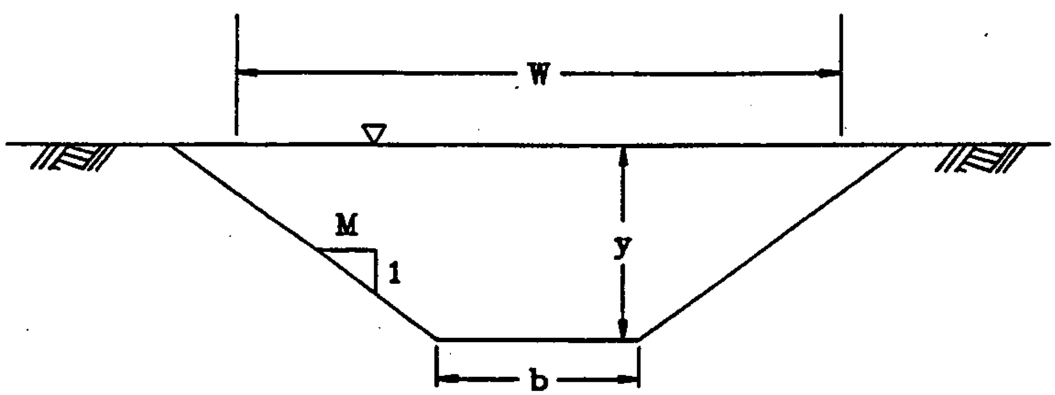


Figure 4-3 - Trapezoidal Channel - Definition of Variables

Equations (continued)

From the geometry of the channel cross-section and the Manning 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:

$$y = C_M \left(Qn/(S^{1/2})\right)^{3/8} \qquad (4.4)$$

Where:

$$C_M = \left[\frac{\{k + 2(M^2 + 1)^{1/2}\}^{2/3}}{1.49\ (k + M)^{5/3}}\right]^{3/8} \qquad (4.5)$$

The associated bottom width is:

$$b = ky \qquad (4.6)$$

The cross-sectional area of the resulting channel is:

$$A = by + My^2 \qquad (4.7)$$

Table 4-3 lists values of C_M and k for various values of M.

Table 4-3

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

4.7 Open Channel Flow Calculations

Design Charts 4.7.1

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration 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 Oven Channel Flow, Federal Highway Administration, Hydraulic Design Series No. 3, 1973.

Manning's Evaluation 4.7.2

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

$$v = (1.49/n)R^{2/3} S^{1/2} \qquad (4.8)$$

$$Q = (1.49/n)A\ R^{2/3} S^{1/2} \qquad (4.9)$$

$$S = [(Qn/1.49\ A\ R^{2/3})]^2 \qquad (4.10)$$

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)

P = wetted perimeter (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.

Geometric Relationships 4.7.3

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 difference 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.

Direct Solutions 4.7.4

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 4-4 and 4-5. Figure 4-4 provides a general solution for the velocity form of Manning's Equation, while Figure 4-5 provides a solution of Manning's Equation for trapezoidal channels.

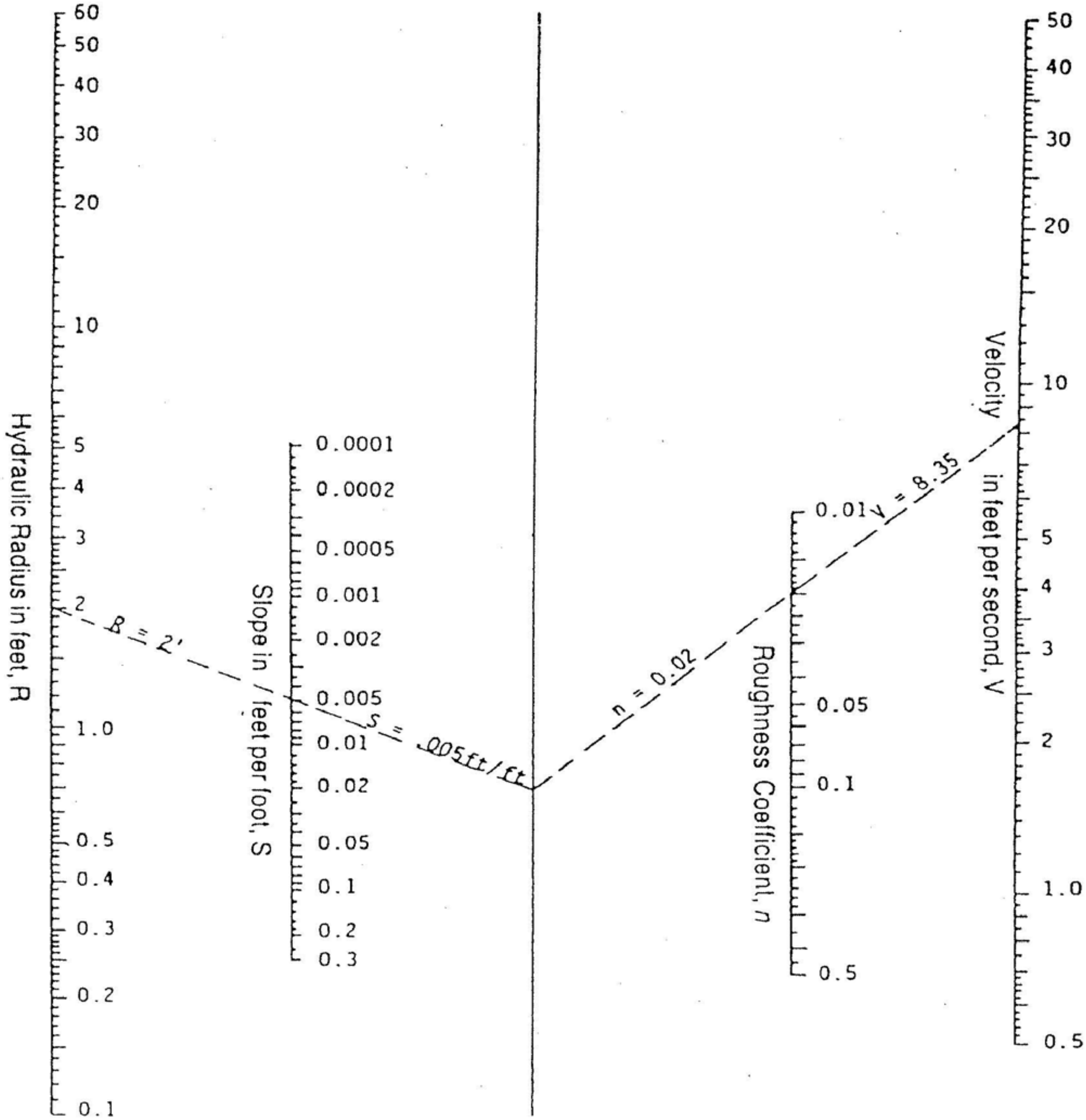


Figure 4-4
Nomograph for the Solution of Manning's Equation

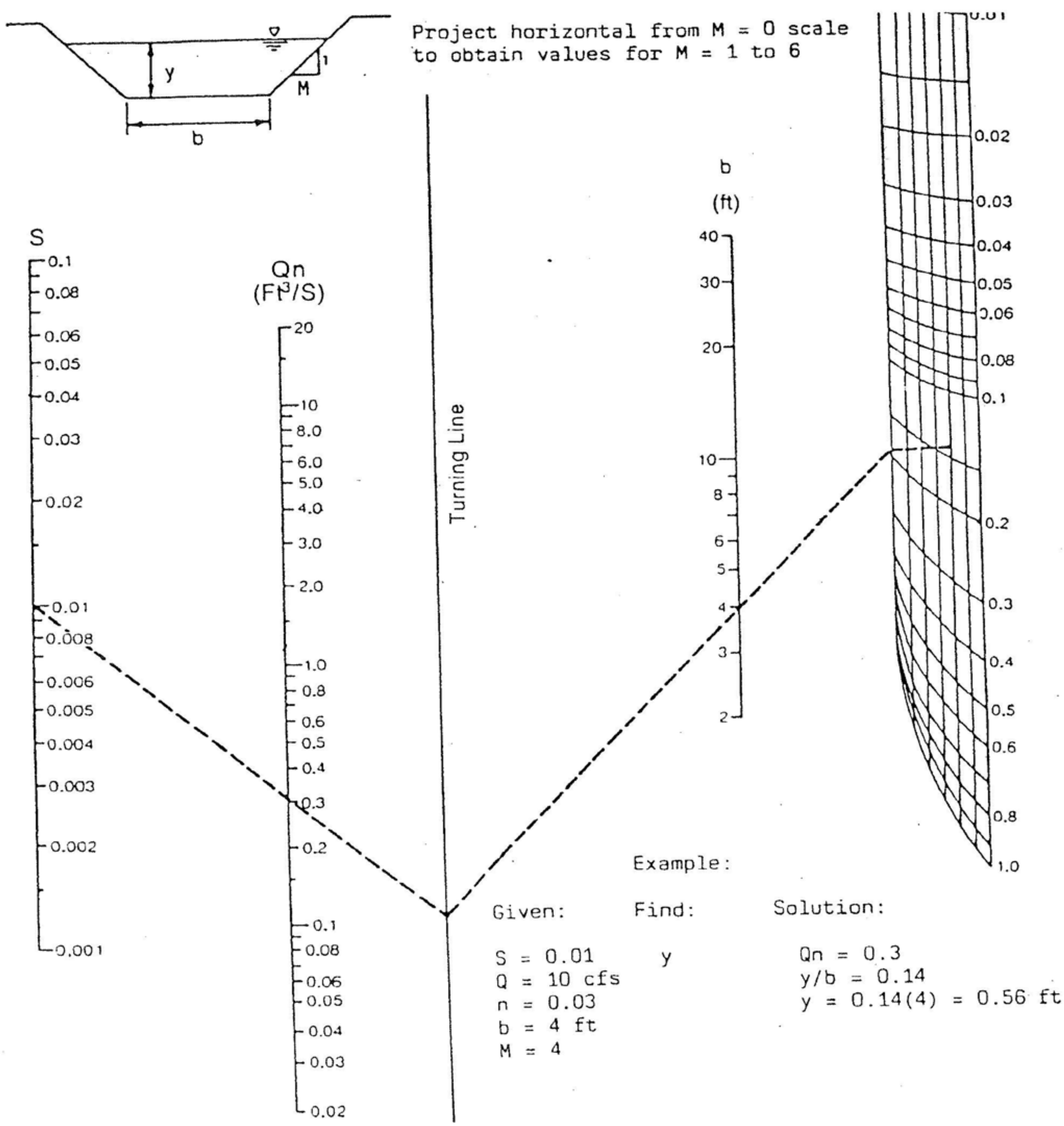


Figure 4-5

Solution of Manning's Equation for Trapezoidal Channels

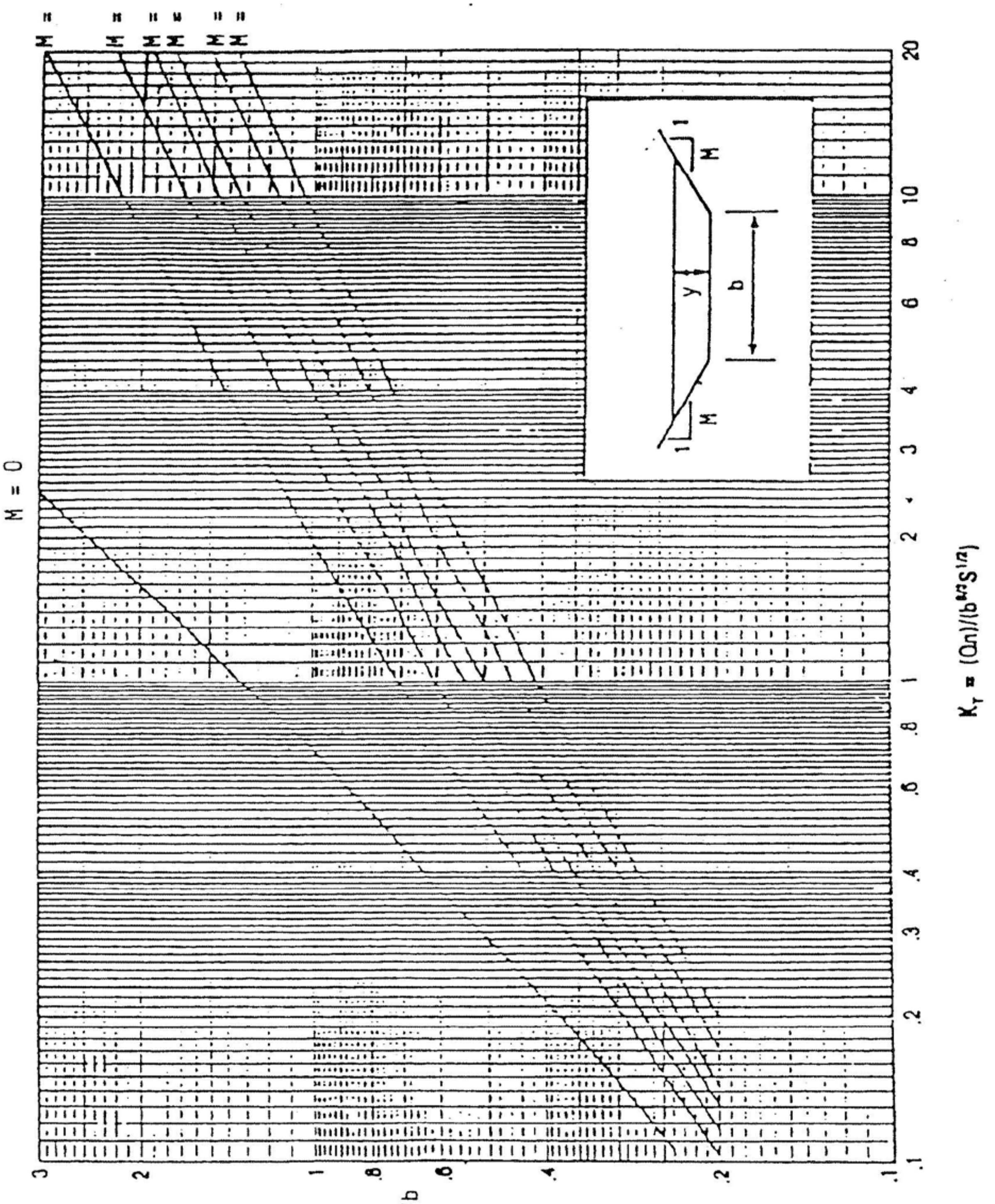


Figure 4-6

Trapezoidal Channel Capacity Chart

Normal Depth Solutions 4.7.5

A trial and error procedure for solving Manning's – Equation is used to complete 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:

$$AR^{2/3} = (Qn)/(1.49\ S^{1/2}) \qquad (4.11)$$

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 4.11 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 4-6 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:*

$$K_T = (On)/(b^{8/3}\ S^{1/2}) \qquad (4.12)$$

Where: KT = 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 4-6 with the value of KT calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriated m value from Step 1.*

Normal Depth Solutions (continued)

4. *From the point of intersection 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.*

4.8 Critical Flow Calculations

Background 4.8.1

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

$$Q^2/g = A^3 / T \qquad \textbf{(4.13)}$$

Where: Q = discharge rate for design conditions (cfs)

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

A = cross-sectional area (ft^2)

T = top width of water surface (ft)

A trial and error procedure is needed to solve equation 4-13.

Semi-Empirical Equations 4.8.2

Semi-empirical equations (as presented in Table 4-4) or section factors (as presented in Figure 4-7) 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:

$$Z = Q/(g^{1/2}) \qquad \textbf{(4.14)}$$

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 about 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.*

Table 4-4 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	$y_c = 0.81(Q^2/gM^{0.75}b^{1.25})^{0.27} - (b/30M)$	$0.1 < 0.5522(Q/b^{2.5}) < 0.1$, 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]	$A^3/T = Q^2/g$	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
- T = Top width of water surface, in feet

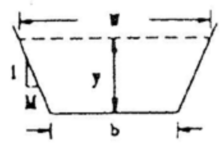
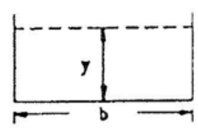
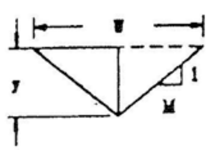
[a]See Figure 4-7 for channel sketches

[b]Assumes uniform flow with the kinetic energy coefficient eoual 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	$by + My^2$	$b + 2y\sqrt{M^2 + 1}$	$\frac{by + My^2}{\sqrt{b + 2y\ M^2 + 1}}$	$b + 2My$	$\frac{\left[(b + My)y\right]^{1.5}}{\sqrt{b + 2My}}$
 Rectangle	by	$b + 2y$	$\frac{by}{b + 2y}$	b	$by^{1.5}$
 Triangle	My^2	$2y\sqrt{M^2 + 1}$	$\frac{My}{2\sqrt{M^2 + 1}}$	$2My$	$\frac{\sqrt{2}}{2} My^{2.5}$

Open Channel Geometric Relationships for Various Cross Sections

Figure 4–7

Froude Number 4.8.3

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

$$Fr = v / (gA/T)^{1/2} \qquad \textbf{(4.15)}$$

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)

T = top width of flow (ft)

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

4.9 Open Channel Design

Procedures for designing open channels, both vegetative and riprap are found in Section 8.05 of the North Carolina Erosion and Sediment Control Planning and Design Manual.

4.10 Riprap Design

Assumptions 4.10.1

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap placement in both natural and prismatic channels and had 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.

Procedure 4.10.2

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 Section 4.7.5). Manning's n values for riprap can be calculated form the equation:*

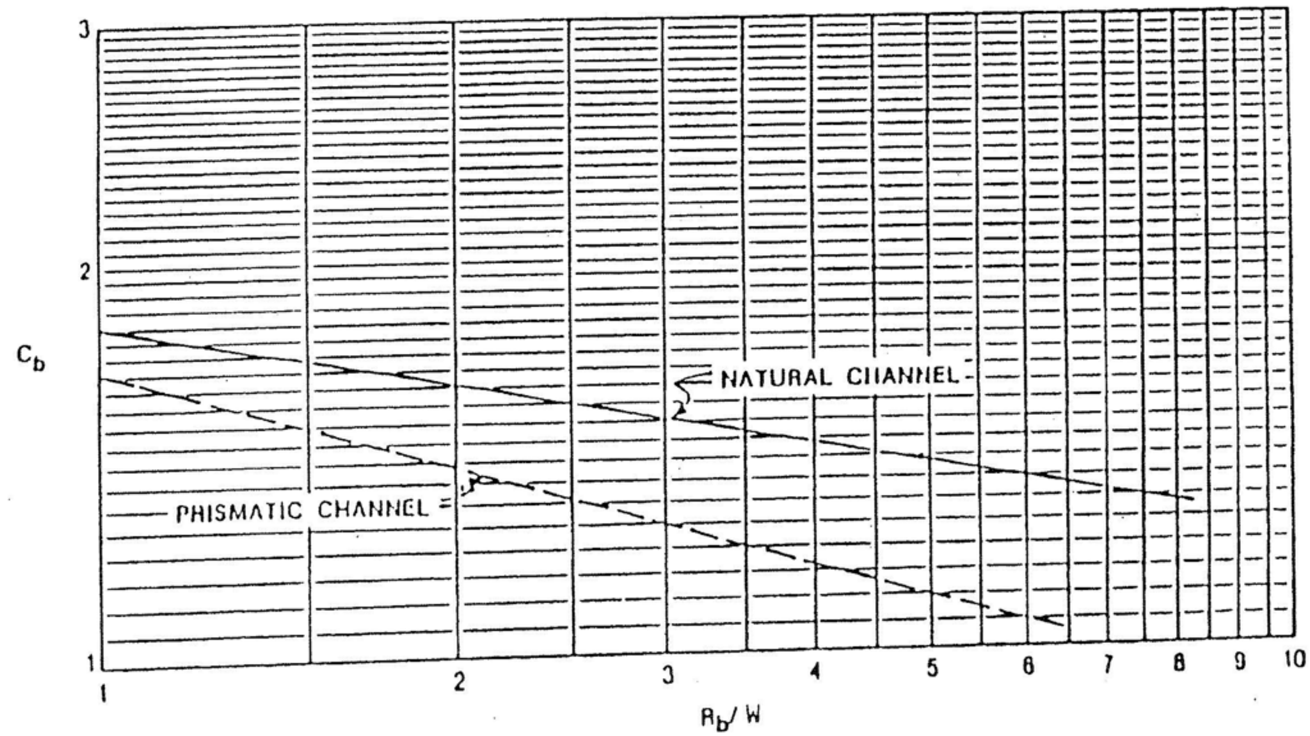
$$\mathbf{n = 0.0395\ (d_{50})^{1/6}} \qquad \mathbf{(4.17)}$$

2. *If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bed correction coefficient, C_b, given in Figure 4-8 for either a natural or prismatic channel. This requires determining the channel top width, W, just upstream from the bed 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 4-9.*

4. *Determine the required minimum d_{30} value from Figure 4-10 which is based on the equation:*

$$\mathbf{d_{30}/D = 0.193\ Fr^{2.5}} \qquad \mathbf{(4.18)}$$



To obtain effective velocity, multiply known velocity by C .

W = Channel Top Width

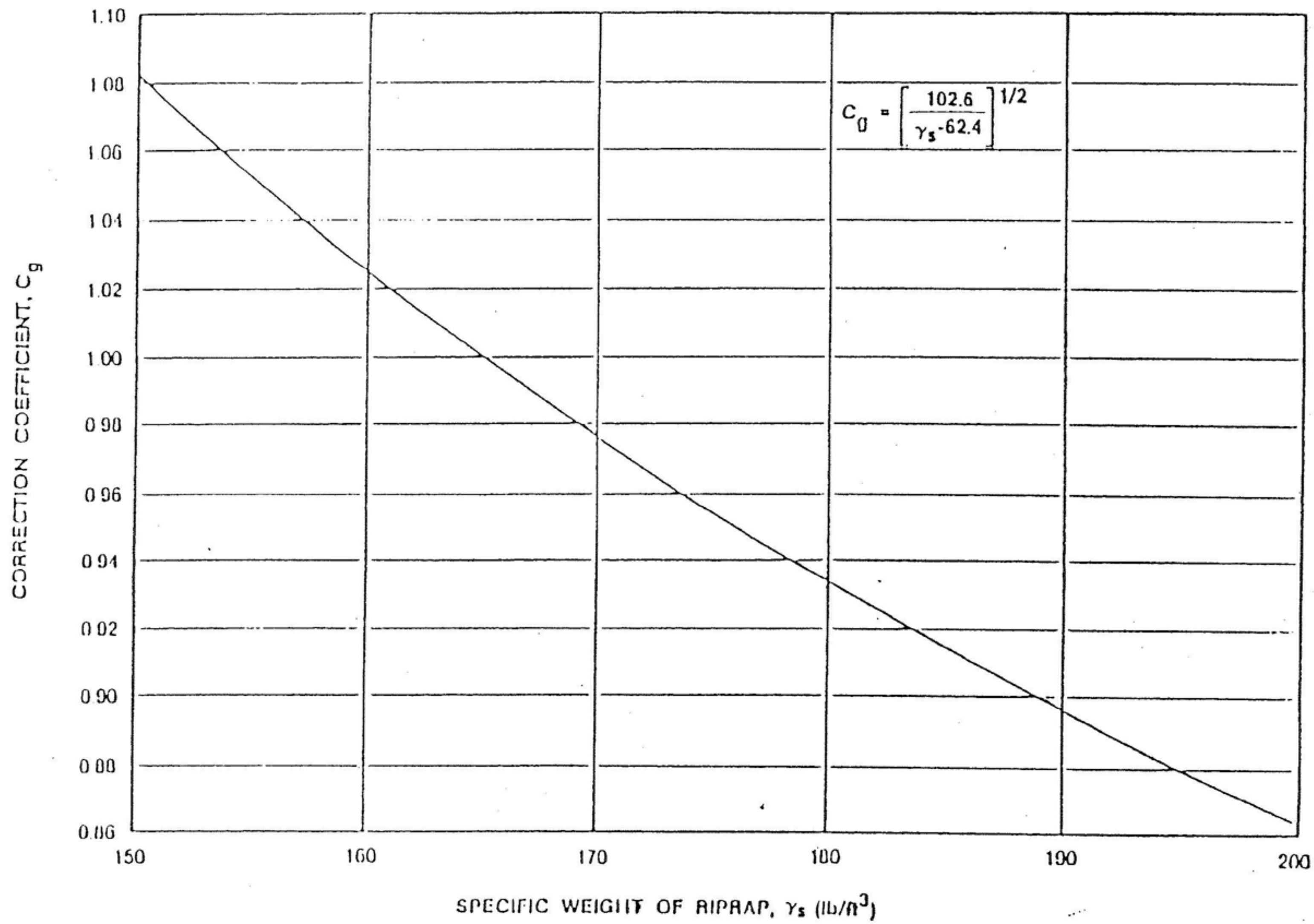
R_b= Centerline Bend Radius

C_b= Correction Coefficient

Referencce: Maynord (1987)

Figure 4-8

Riprap Lining Bend Correction Coeffic



C_g = Correction Coefficient

To obtain effective velocity, multiply known velocity by C_g.

Figure 4–9

Riprap Lining Specific Weight Correction Coefficie

Procedure

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 4.15), dimensionless

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

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

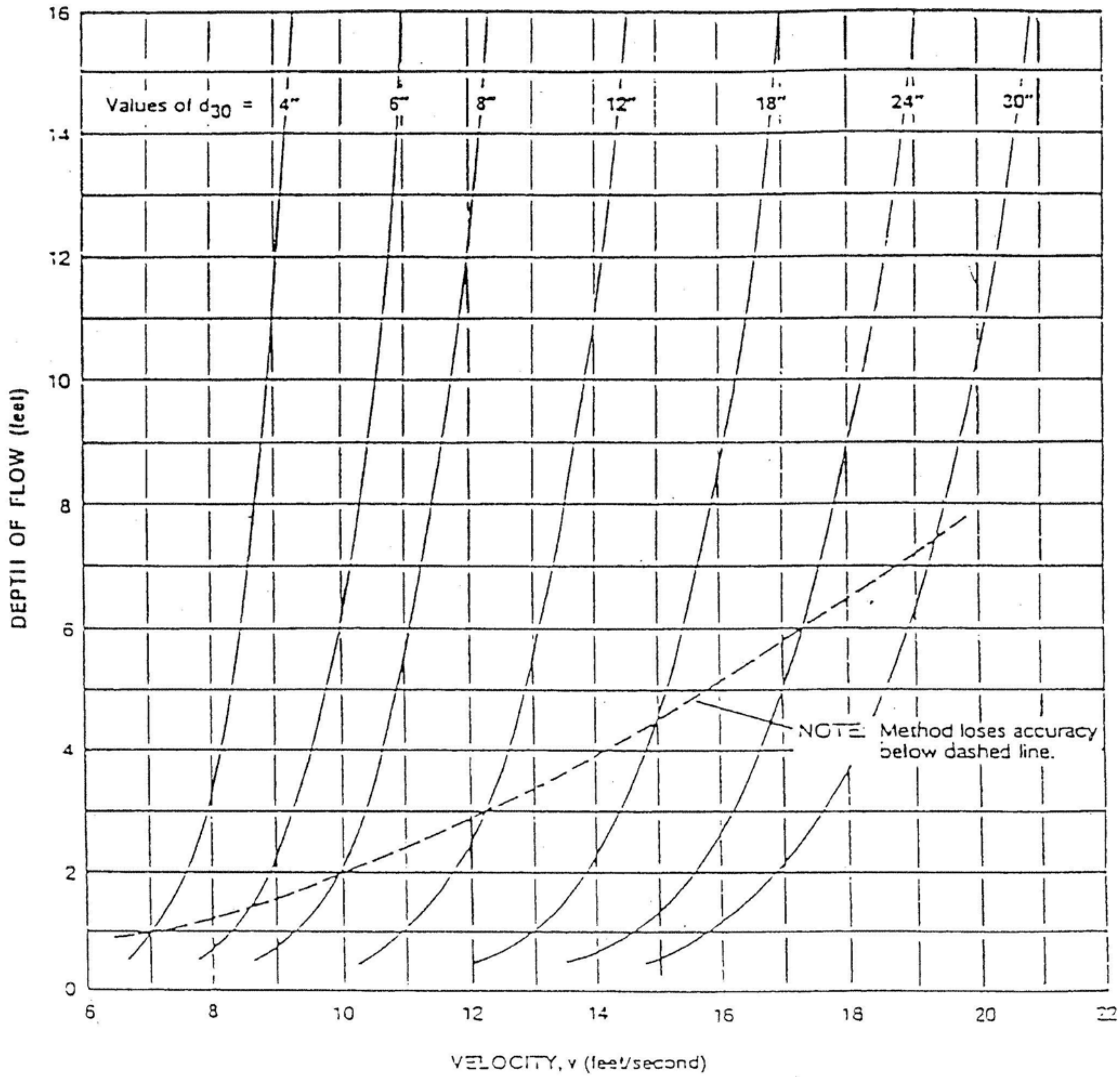
5. *Determine available riprap gradations. A well graded riprap is preferable to uniform size or gap graded. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50}, size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (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 equations:*

$$\mathbf{w = 0.5236\ SW_s\ d^3} \qquad \textbf{(4.19)}$$

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 d85 / d15 is between 2.0 and 2.3 and a smaller d30 size is desired, a thickness greater than d100 can be used to offset the smaller d30 size. Figure 4-13 can be used to make an approximate adjustment using the ratio of d30 sizes. Enter the y-axis with the ratio of the desired d30 size to the standard d30 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.*

6. *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.*



Reference: Reese (1983).

Figure 4–10

Riprap Lining d_{30} Stone Size as a Function of Mean Velocity and Depth

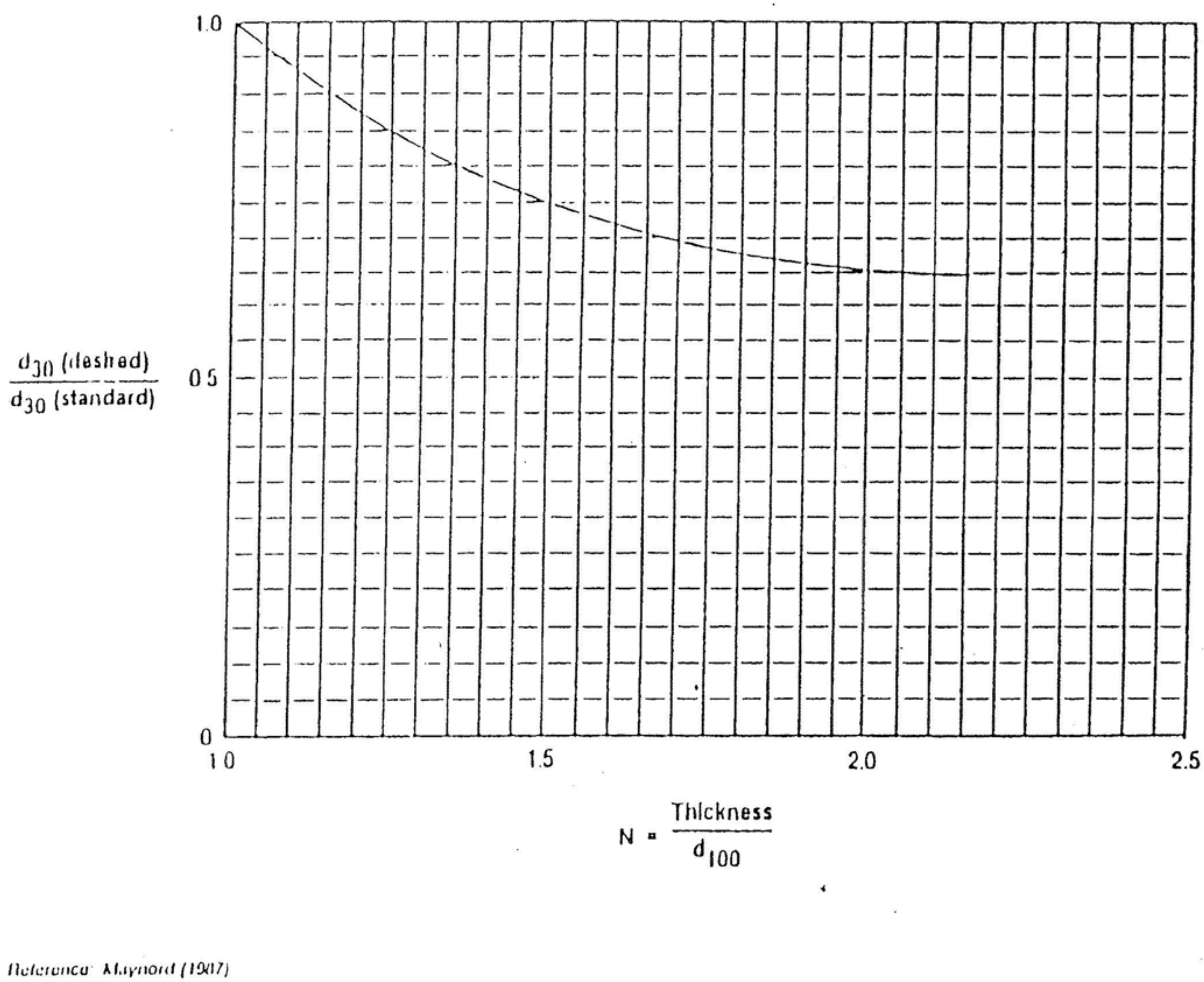


Figure 4-11

Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 2.0 to 2.

4.11 Gradually Varied Flow

Introduction 4.11.1

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constructions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used programs are, HEC-2, developed by the U.S. Army Corps of Engineers (1982) and Bridge Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the direct step method, as presented by Chow (1959). For an irregular nonuniform channel, the standard step method is recommended although it is more tedious and iterative process. The use of HEC-2 or WSPRO is recommended for standard step calculations.

Energy losses in transitions, junctions, and bends shall be accounted for as part of water surface profile calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity of the stream and flood plain. In general, a cross section should be obtained at each location where there are significant changes in stream width, shape, or vegetal patterns. Sections should usually be no more than 4 to 5 channel widths apart or 100 ft apart for ditches or streams and 500 ft apart for flood plains, unless the channel is very regular.

Direct Step Method 4.11.2

The direct step method is limited to prismatic channels. A form for recording the calculations described below is presented in Table 4-5 (Chow 1959).

1. *Record the following parameters across the top of Table 4-5:*

   Q = *design flow (cfs)*

   n = *manning's n value*

   S_o = *channel bottom slope (ft/ft)*

   α = *energy coefficient*

   y_c = *critical depth (ft)*

   y_n = *normal depth (ft)*

Location ____________

$Q =$ ________ $n =$ ________ $S_o =$ ________ $\alpha =$ ________ $y_c =$ ________ $y_n =$ ________

y (1)	A (2)	R (3)	v (4)	$\alpha v^2/2g$ (5)	E (6)	ΔE (7)	S_f (8)	$\bar{S}_f$ (9)	$S_o - \bar{S}_f$ (10)	Δx (11)	x (12)
1.											
2.											
3.											
4.											
5.											
6.											
7.											
8.											
9.											
10.											
11.											
12.											
13.											
14.											
15.											
16.											
17.											
18.											
19.											
20.											
21.											
22.											

(8) $S_f = \dfrac{n^2 v^2}{2.22\, R^{4/3}}$ (11) $\Delta x = \dfrac{\Delta E}{S_o - \bar{S}_f}$

Water Surface Profile Computation Form for the Direct Step Method

Table 4–5

Direct Step Method (continued)

2. *Using the desired range of flow depths, y, recorded in column 1, compute the cross sectional area, A, the hydraulic radius, R, and average velocity, v, and record the results in columns 2, 3, and 4, respectively.*

3. *Compute the velocity head, $\alpha v^2/2b$, in ft, and record the result in column 5.*

4. *Compute specific energy, ΔE, by summing the Method velocity head in column 5 and the depth of flow in column 1. Record the result in column 6.*

5. *Compute the change in specific energy, ΔE, between the current and previous flow depths and record the result in column 7 (not applicable for row 1).*

6. *Compute the friction slope using the equation:*

$$S_f = (n^2 v^2)/(2.22 R^{4/3}) \qquad (4.24)$$

Where: Sf = friction slope (ft/ft)

n = manning's n value

v = average velocity (ft/s)

R = hydraulic radius (ft)

Record the result in column 8.

7. *Determine the average of the friction slope between this depth and the previous depth (not applicable for row 1). Record the result in column 9.*

8. *Determine the difference between the bottom slope So, and the average friction slope Sf, from column 9 (not applicable for row 1). Record the result in column 10.*

9. *Compute the length of channel between consecutive rows or depths of flow using the equation:*

$$\Delta x = \Delta E/(S_o - S_f) = \text{Column 7/Column 10} \qquad (4.25)$$

Where: *Δx = length of channel between consecutive depths of flow (ft)*

ΔE = change in specific energy (ft)

S_o = bottom slope (ft/ft)

S_f = friction slope (ft/ft)

Record the result in column 11.

10. *Sum the distances from the starting point to give cumulative distances, x, for each depth in column 1 and record the result in column 12.*

Standard Step Method 4.11.3

The standard step method is a trial and error procedure applicable to both the natural and prismatic channels. The step computations are arranged in tabular form, as shown in Table 4-6 and described below (Chow, 1959):

1. *Record the following parameters across the top of Table 4-6:*

   Q = design flow (cfs)

   n = Manning's n value

   S_o = Channel bottom slope (ft/ft)

   α = energy coefficient

   k_c = eddy head loss coefficient (ft)

   y_c = critical depth (ft)

   y_n = normal depth (ft)

2. *Record the location of measured channel cross sections and the trial water surface elevation, z, for each section in column 1 and 2. The trial elevation will, be verified or rejected based on computations of the step method.*

3. *Determine the depth of flow, y, based on trial elevation and channel section data. Record the result in column 3.*

4. *Using the depth from Step 3 and section data, compute the cross-sectional area, A, in ft, and hydraulic radius, R, in ft. Record the results in columns 4 and 5.*

5. *Divide the design discharge by the cross-sectional area from Step 4 to compute the average velocity, v, in ft/s. Record the result in column 6.*

6. *Compute the velocity head, α v2 /2g, in ft, and record the result in column 7.*

7. *Compute the total head, H, in ft, by summing the water surface elevation, z, in column 2 and the velocity head in column 7. Record the result in column 8.*

8. *Compute the friction slope S_f, using equation 4.24 and record the result in column 9.*

9. *Determine the average friction slope, S_f, between the sections in each step (not applicable for row 1). Record the result in column 10.*

10. *Determine the distance between sections, Δ x, and record the result in column 11.*

11. *Multiply the average friction slope, S_f (column 10), by the reach length, Δ x (column 11), to give the friction loss in the reach, h_f. Record the result in column 12.*

Location ____________________

Q = ______ n = ______ S_o = ______ α = ______ k_e = ______ y_c = ______ y_n = ______

Station (1)	z (2)	y (3)	A (4)	R (5)	v (6)	$\alpha v^2/2g$ (7)	H (8)	S_f (9)	$\bar{S}_f$ (10)	Δx (11)	h_f (12)	h_e (13)	H (14)
1.													
2.													
3.													
4.													
5.													
6.													
7.													
8.													
9.													
10.													
11.													
12.													
13.													
14.													
15.													
16.													
17.													
18.													
19.													
20.													
21.													
22.													

(9) $S_f = \dfrac{n^2 v^2}{2.22\, R^{4/3}}$

(12) $h_f = \Delta x \times \bar{S}_f$

(13) $h_e = \dfrac{k_e v^2}{2g}$

Water Surface Profile Computation Form for the Standard Step Method

Table 4–6

Standard Step Method (continued)

12. *Compute the eddy loss using the equation:*

$$\mathbf{h_e = (k_e\ v^2)/2g} \qquad \textbf{(4.26)}$$

Where: h_e *= eddy head loss (ft)*

k_e *= eddy head loss coefficient (ft) (for prismatic and regular channels,* $k_e = 0$*; for gradually coverging and diverging channels,* k_e *= 0 to 0.2; for abrupt expansions and contractions,* $k_e = 0.5$*)*

v = average velocity (ft/s) (column 6)

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

13. *Compute the elevation of the total head, H, by adding the values of hf, and he (columns 12 and 13) to the elevation at the lower end of the reach, which is found in column 14 of the previous reach or row. Record the results in column 14.*

14. *If the value of H computed above does not agree closely with that entered in column 8, a new trial value of the water surface elevation is used in column 2 and calculations are repeated until agreement if obtained. The computation may then proceed to the next step or section reported in column 1.*

4.12 Gradually Varied Flow-Example Problems

Example 1
4.12.1

Direct, Step Method

Use the direct step method (Section 4.11.2) to compute a water surface profile for a trapezoidal channel using the following data:

Q = 400 cfs

B = 20 ft

M = 2

S = 0.0016 ft

n = 0.025

α = 1.10

A dam backs up water to a depth of 5 ft. immediately behind the dam. The upstream end of the profile is assumed to have a depth 1 percent greater than normal depth.

Results of calculations, as obtained from Chow (1959), are reported in Table 4-7. Values in each column of the table are briefly explained below.

1. *Depth of flow, in ft., arbitrarily assigned values ranging from 5 to 3.4 ft.*
2. *Water area in sq. ft. corresponding to the depth, y, in column 1.*
3. *Hydraulic radius, in ft, corresponding to y in column 1.*
4. *Mean velocity, in ft/s, obtained by dividing 400 cfs by the water area in column 2.*
5. *Velocity head, in ft, calculated using the mean velocity from column 4 and α = 1.1.*
6. *Specific energy, E, in ft obtained by adding the velocity head in column 5 to the depth of flow in column 1.*
7. *Change of specific energy, delta E, in ft, equal to the difference between the E value in column 6 and that of the previous step.*
8. *Friction slope, S_f, computed by equation 4.24, with n = 0.025, v as given in column 4 and R as given in column 3.*
9. *Average friction slope between the steps, S_f, equal to the arithmetic mean of the friction slope computed in column 8 and that of the previous step.*

y (1)	A by+My² (2)	R A/P (3)	V Q/A (4)	$\alpha V^2/2g$ (5)	E (1)+(5) (6)	ΔE (7)	S_f (8)	$\bar{S}_f$ (9)	$S_o-\bar{S}_f$ (10)	Δx (7)/(10) (11)	x (12)
5.00	150.00	3.54	2.667	0.1217	5.1217	--	0.000370	--	--	--	--
4.80	142.08	3.43	2.819	0.1356	4.9356	0.1861	0.000433	0.000402	0.001198	155	155
4.60	134.32	3.31	2.979	0.1517	4.7517	0.1839	0.000507	0.000470	0.001130	163	318
4.40	126.72	3.19	3.156	0.1706	4.5706	0.1811	0.000598	0.000553	0.001047	173	491
4.20	119.28	3.08	3.354	0.1925	4.3925	0.1781	0.000705	0.000652	0.000948	188	679
4.00	112.00	2.96	3.572	0.2184	4.2184	0.1741	0.000850	0.000778	0.000822	212	891
3.80	104.88	2.84	3.814	0.2490	4.0490	0.1694	0.001020	0.000935	0.000665	255	1146
3.70	101.38	2.77	3.948	0.2664	3.9664	0.0826	0.001132	0.001076	0.000524	158	1304
3.60	97.92	2.71	4.085	0.2856	3.8856	0.0808	0.001244	0.001188	0.000412	196	1500
3.55	96.21	2.68	4.158	0.2958	3.8458	0.0398	0.001310	0.001277	0.000323	123	1623
3.50	94.50	2.65	4.233	0.3067	3.8067	0.0391	0.001382	0.001346	0.000254	154	1777
3.47	93.48	2.63	4.278	0.3131	3.7831	0.0236	0.001427	0.001405	0.000195	121	1898
3.44	92.45	2.61	4.326	0.3202	3.7602	0.0229	0.001471	0.001449	0.000151	152	2050
3.42	91.80	2.60	4.357	0.3246	3.7446	0.0156	0.001500	0.001486	0.000114	137	2187
3.40	91.12	2.59	4.388	0.3292	3.7292	0.0154	0.001535	0.001518	0.000082	188	2375

Note: Q = 400 cfs, n = 0.025, $S_o = 0.0016$, $\alpha = 1.10$, $y_c = 2.22$ ft, $y_n = 3.36$ ft

Reference: Chow (1959)

Example 1 (continued)

10. *Difference between the bottom slope, S_o, 0.0016 and the average friction slope, S_f, in column 9.*

11. *Length of the reach, Δx, in ft, between the consecutive steps, computed by equation 4.25 or by dividing the value of Δ E in column 7 by the value of So – Sf in column 10.*

12. *Distance from the section under consideration to the dam site. This is equal to the cumulative sum of the values in column 11 computed for previous steps.*

Example 2 4.12.2

Standard Step Method

Use the standard step method (see Section 4.11.3) to compute a water surface profile for the channel data and stations considered in the previous example. Assume the elevation at the dam site is 600 ft.

Results of the calculations, as obtained from Chow (1959), are reported in Table 4-8 Values in each column of the table are briefly explained below:

1. *Section identified by station number such as "section 1 + 55". The location of the stations are fixed at the distances determined in the previous example to compare the procedure with that of the direct step method.*

2. *Water surface elevation, z, at the station. A trial value is first entered in this column; this will be verified or rejected on the basis of the computations made in the remaining columns of the table. For the first step, this elevation must be given or assumed. Since the elevation of the dam site is 600 ft and the height of the dam is 5 ft, the first entry is 605.00 ft. When the trial value in the second step has been verified, it becomes the basis for the verification of the trial value in the next step, and the process continues.*

3. *Depth of flow, y, in ft, corresponding to the water surface elevation in column 2. For instance, the depth of flow at station 1 + 55 is equal to the water surface elevation minus the elevation at the dam site minus the distance from the dam site times bed slope.*

   605.048 – 600.00 – (155) (0.0016) = 4.80 ft

4. *Water area, A, in square ft, corresponding to y in column 3.*

5. *Hydraulic radius, R, in ft, corresponding to y in column 3.*

6. *Mean velocity, v, equal to the given discharge 400 cfs divided by the water area in column 4.*

7. *Velocity head, in ft, corresponding to the velocity in column 6 and α = 1.1.*

8. *Total head, H, equal to the sum of z in column 2 and the velocity head in column 7.*

Station (1)	z (2)	y (3)	A $by+My^2$ (4)	R A/P (5)	v Q/A (6)	$\alpha v^2/2g$ (7)	H (2)+(7) (8)	S_f (9)	$\bar{S}_f$ (10)	Δx (11)	h_f (12)	h_e (13)	H (14)
0+00	605.000	5.00	150.00	3.54	2.667	0.1217	605.122	0.000370	--	--	--	--	605.122
1+55	605.048	4.80	142.08	3.43	2.819	0.1356	605.184	0.000433	0.000402	155	0.062	0	605.184
3+18	605.109	4.60	134.32	3.31	2.979	0.1517	605.261	0.000507	0.000470	163	0.077	0	605.261
4+91	605.186	4.40	126.72	3.19	3.156	0.1706	605.357	0.000598	0.000553	173	0.096	0	605.357
6+79	605.286	4.20	119.28	3.08	3.354	0.1925	605.479	0.000705	0.000652	188	0.122	0	605.479
8+91	605.426	4.00	112.00	2.96	3.572	0.2184	605.644	0.000850	0.000778	212	0.165	0	605.644
11+46	605.633	3.80	104.88	2.84	3.814	0.2490	605.882	0.001020	0.000935	255	0.238	0	605.882
13+04	605.786	3.70	101.38	2.77	3.948	0.2664	606.052	0.001132	0.001076	158	0.170	0	606.052
15+00	605.999	3.60	97.92	2.71	4.085	0.2856	606.285	0.001244	0.001188	196	0.233	0	606.285
16+23	606.146	3.55	96.21	2.68	4.158	0.2958	606.442	0.001310	0.001277	123	0.157	0	606.442
17+77	606.343	3.50	94.50	2.65	4.233	0.3067	606.650	0.001382	0.001346	154	0.208	0	606.650
18+98	606.507	3.47	93.48	2.63	4.278	0.3131	606.820	0.001427	0.001405	121	0.170	0	606.820
20+50	606.720	3.44	92.45	2.61	4.326	0.3202	607.040	0.001471	0.001449	152	0.220	0	607.040
21+87	606.919	3.42	91.80	2.60	4.357	0.3246	607.244	0.001500	0.001486	137	0.204	0	607.244
23+75	607.201	3.40	91.12	2.59	4.388	0.3292	607.530	0.001535	0.001518	188	0.286	0	607.530

Note: $Q = 400$ cfs, $n = 0.025$, $S_o = 0.0016$, $\alpha = 1.10$, $h_e = 0$, $y_c = 2.22$ ft, $y_n = 3.36$ ft

Reference: Chow (1959)

Example 2 (continued)

9. *Friction slope, S_f, computed by equation 4.24, with n = 0.025, v from column 6, and R from column 5.*

10. *Average friction slope through the reach, S_f, between, the (continued) sections in each step, approximately equal to the arithmetic mean of the friction slope just computed in column 9 and that of the previous step.*

11. *Length of the reach between the sections, Δx, equal to the difference in station numbers between the stations.*

12. *Friction loss in the reach, h_f, equal to the product of the values in column 10 and 11.*

13. *Eddy loss in the reach, h_e equal to zero.*

14. *Elevation of the total head, H, in ft, computed by adding the values of h_f and h_e, in columns 12 and 13 to the elevation at the lower end of the reach, which is found in column 14 of the previous reach. If the value obtained does not agree closely with that entered in column 8, a new trial value of the water surface elevation is assumed until agreement is obtained. The value that leads to agreement is the correct water surface elevation. The computation may then proceed to the next step.*

4.13 Approximate Flood Limits

Introduction
4.13.1

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. The requires a backwater analysis to determine the stream flow depth. Both HEC-2 and WSPRO methods are acceptable.

Floodline
Restrictions
4.13.2

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 the Hydrology Chapter.*

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.

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CHAPTER 5

STORM DRAINAGE SYSTEMS

Chapter Table of Contents

5.1 Overview

Introduction 5.1.1

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-1 2 (USDOT, FHWA, 1984). Storm drain design is based on the use of the rational formula.

Inlet Definition 5.1.2

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 sump. 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 "sump" condition exists when the inlet is located at a low point and water enters from both directions.

Criteria 5.1.3

The following criteria shall be used for drainage system design. These criteria are consistent with the North Carolina Department of Transportation practices.

Design Frequencies

	Roadway Ditch/Pipe	Cross Drain	Gutter
Thoroughfares	10 year storm	50 year storm	4 inches/hour
Minor Roadways	10 year storm	25 year storm	4 inches/hour

Spread Limits

- *Maximum spread of 6 feet in 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.*
- *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.*
- *In a sump condition (a vertical sag with no available overflow) inlet capacity and storm drains must be designed for the 25-year storm.*
- *Ponding at yard inlets outside the roadway should be limited to a maximum of one foot above a grated inlet for the 10-year storm.*

5.2 Symbols and Definitions

Symbol Table

To provide consistency within this chapter as well as through-out this manual the following symbols will be used. These symbols were selected because of their wide use in storm drainage publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

Table 5-1
SYMBOLS AND DEFINITIONS

Symbol	Definition	Units
A	Area of cross section	ft^2
d or D	Depth of gutter flow	ft
g	Acceleration due to gravity (32.2 ft/s^2)	ft/s^2
H	Head loss	ft
K	Loss coefficient	-
L	Pipe length	ft
n	Manning's roughness coefficient	
Q	Rate of discharge	cfs
R	Hydraulic radius	ft
S	Slope	ft/ft
Sf	Friction slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
V	Velocity of flow	ft/s
Z	T/d, reciprocal of the transverse slope	ft/ft

5.3 Concept Defintions

Following are definitions of concepts important in storm drain analysis and design as used it this chapter.

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.*
Curb-Opening Inlet	*A drainage inlet consisting of an opening in the roadway curb.*
Drop Inlet	*A drainage inlet with horizontal or nearly horizontal grade.*
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.*
Frontal Flow	*The portion of the gutter flow which passed over the upstream side of grate.*
Gutter	*That portion of the roadway section adjacent to the curb which is utilized to convey storm water runoff.*
Hydraulic Grade Line	*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 run.*
Inlet Efficiency	*The ratio of flow intercepted by an inlet to total flow in the gutter.*
Grate Perimeter	*The sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.*
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.*

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.*
Velocity Head	*A quantity proportional to the kinetic energy of flowing water expressed as a height or head of water.*
	For a more complete discussion of these concepts and others related to storm drain design, the reader is referred to – Drainage of Highway Pavements, Federal Highway Administration, Hydraulic Engineering Circular No. 12, March 1984.

5.4 Pavement Drainage

Introduction 5.4.1

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*

Storm Drain Location 5.4.2

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

Inlet Types and Spacing 5.4.3

Inlet types shall be selected from the Charlotte-Mecklenburg 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.

Longitudinal Slope 5.4.4

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 vary slightly to achieve .5 percent minimum gutter.

Cross Slope 5.4.5

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

Curb and 5.4.6

Gutter Curb and gutter installation shall be designed in accordance with the Charlotte-Mecklenburg Land Development Standards Manual. North Carolina Department of Transportation standards are also acceptable.

Median Ditches 5.4.7

Large median areas and inside shoulders should be sloped to a center swale, preventing drainage form 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.

Roadside Ditches 5.4.8

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 ditch shall be a minimum of 18 inches deep and shall provide the capacity designed for a 10 year storm. The 25 year storm should be checked to prevent inundation of the pavement. The minimum 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 4ft/sec. For grass lined channels with discharge up to 7 cft/sec, permanent matting may be approved on a case by case basis. For discharge greater than 7 cft/sec, a concrete lined ditch will 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 or roadside ditches, a design shall be provided for driveway culverts for each individual lot on the plan. The driveway and culvert shall be designed such that the flow from a 25 year storm shall not inundate the roadway pavement. The use of a small driveway culvert, 15 inches minimum, in conjunction with overtopping of the driveway itself will be allowed. Sizes for all driveway culverts shall be shown in a tabular form on the plans, and each culvert shall be designed for the highest ditch flow applicable for the lot.

Bridge Decks 5.4.9

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 man 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 tacks. 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.

Median Barriers 5.4.10

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.

5.5 Gutter Flow Calculations

Formula 5.5.1

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

$$\mathbf{Q = [0.56/n]S_T^{2/3}S_L^{1/2}T^{8/3}} \qquad \textbf{(5.1)}$$

Where: Q = Gutter flow rate (ft/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.

Procedure 5.5.2

Using Table 5-2, 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 intlet.

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 – $Q_{total} = Q_{sub} + \sum Q_{bypass}$

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

6. Trans. Slope – Transverse slope at "inlet #n" (in feet per foot). This 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 5-1.

Procedure (continued)

8. *Inlet Cap. – Inlet Capacity (CFS)*

$$Q = KD^{5/3}$$

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 =0.25 ft.

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 desination).*

Note: Computer programming for gutter flow analysis is acceptable. The computer printout should contain the same information that is shown in Table 5-2.

INLET CAPACITY CHART

PROJECT__________ COMPUTED BY __________ DATE______

LOCATION__________ CHECKED BY __________

RAINFALL INTENSITY = 4.0 IN/HR

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

Table 5 - 2

5.6 Grate Inlet Design

Grate Inlets on Grade 5.6.1

Following is a discussion of the procedure for the design of grate inlets on grade. Figure 5-1 is used for the design of grate inlets on grade using type 'F' and 'G' grates.

Design Steps

1. *Determine the following input data:*

- *Gutter flow rate – Q (cfs)*
- *Longitudinal gutter slope – S_L (ft/ft)*
- *Transverse gutter slope – ST (ft/ft)*
- *Roughness coefficient – 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 Equation:*

$$\mathbf{Q = (0.56/n)ZD^{8/3}\,S_L^{1/2}} \qquad \mathbf{(5.2)}$$

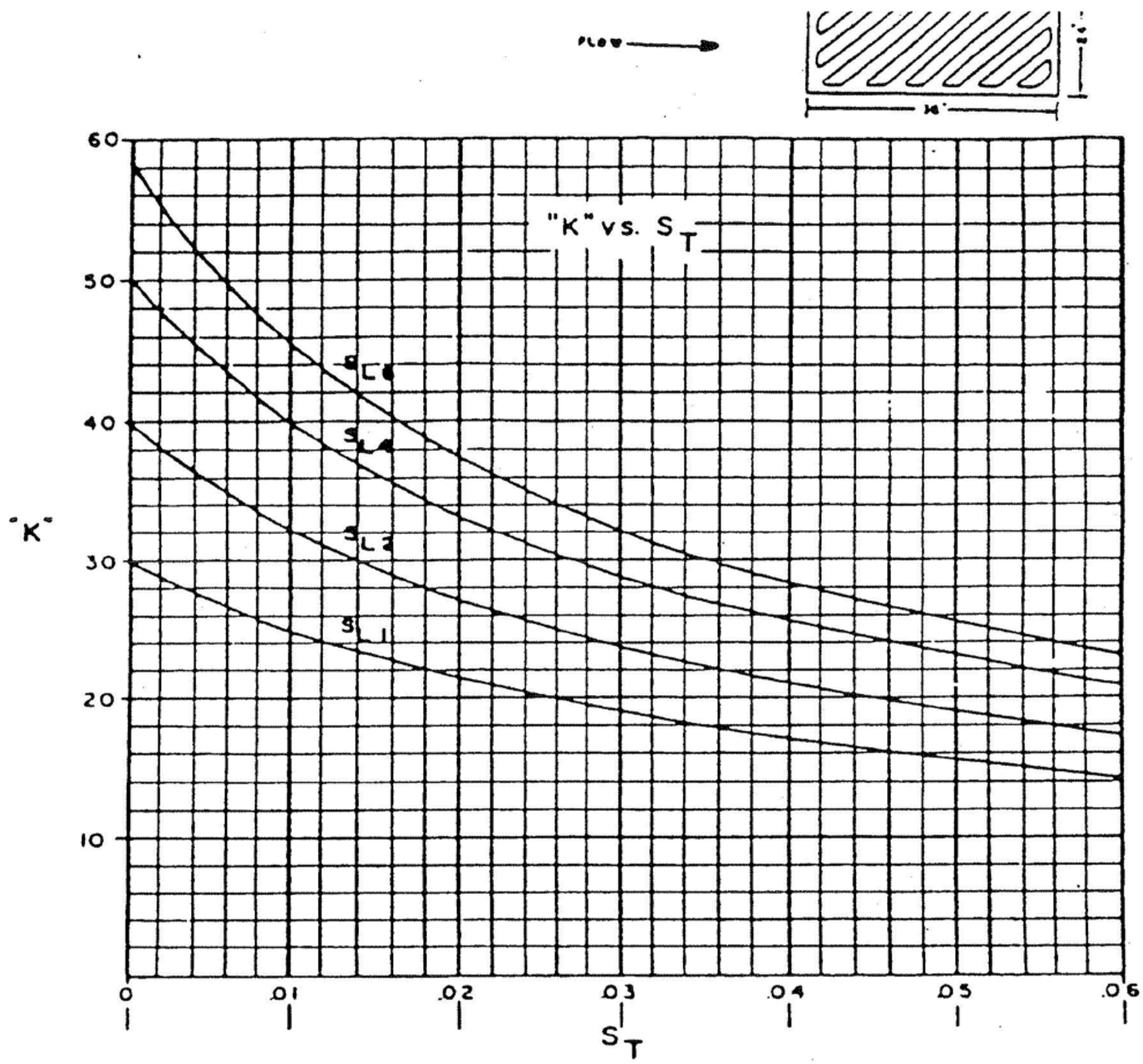
Since $\mathbf{D = T(S_L)}$ **(5.3)**

and $\mathbf{S_T = 1/Z}$ **(5.4)**

T can be derived:

$$\mathbf{T = [Qn(Z^{5/3})/0.56(S_L^{1/2})]^{3/8}} \qquad \mathbf{(5.5)}$$

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)
S_L = longitudinal street slope (ft/ft)



S_T = TRANSVERSE GUTTER SLOPE

S_L = LONGITUDINAL GUTTER SLOPE

K = GRATE INLET COEFFICIENT

$Q = KD^{5/3}$

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

Q = discharge intercepted by grate (cfs)

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

Figure 5-1 Grate Inlet Coefficient - On Grade

Design Steps (continued)

4. *Once T is calculated and determined to be within it imposed limits, D can be calculated as follows:*

$$\mathbf{D = T/Z} \qquad \mathbf{(5.6)}$$

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

5. *The inlet capacity of the grate, Q, can then be determined by using Figure 5-1. If 100% interception is not achieved, the overflow must be included in the next system.*

Example 1
5.6.1.2

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

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

T = 7.1 ft (which is less than 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 5-1, K = 28.5*

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.

Grate Inlets in Sump 5.6.2

Because a grate inlet in a sump condition is subject to clogging, a curb opening is required as a supplemental inlet. The capacity of a grate in a sump depends upon the area of the openings and the depth of water at the grate. Figure 5-2 can be used to calculate the head or flow for Type 'E' grates in sump or depressed conditions, and Figure 5-3 for standard drop inlet grates.

Type 'E' Grate 5.6.2.1

For a type 'E' grate, the weir equation will control to a depth (D) of 0.69 feet. Refer to Figure 5-2. 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:*

$$\mathbf{D = \left(\frac{Surface\ Q_{total}}{3.3P} \right)^{2/3}} \qquad \mathbf{(5.8)}$$

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)

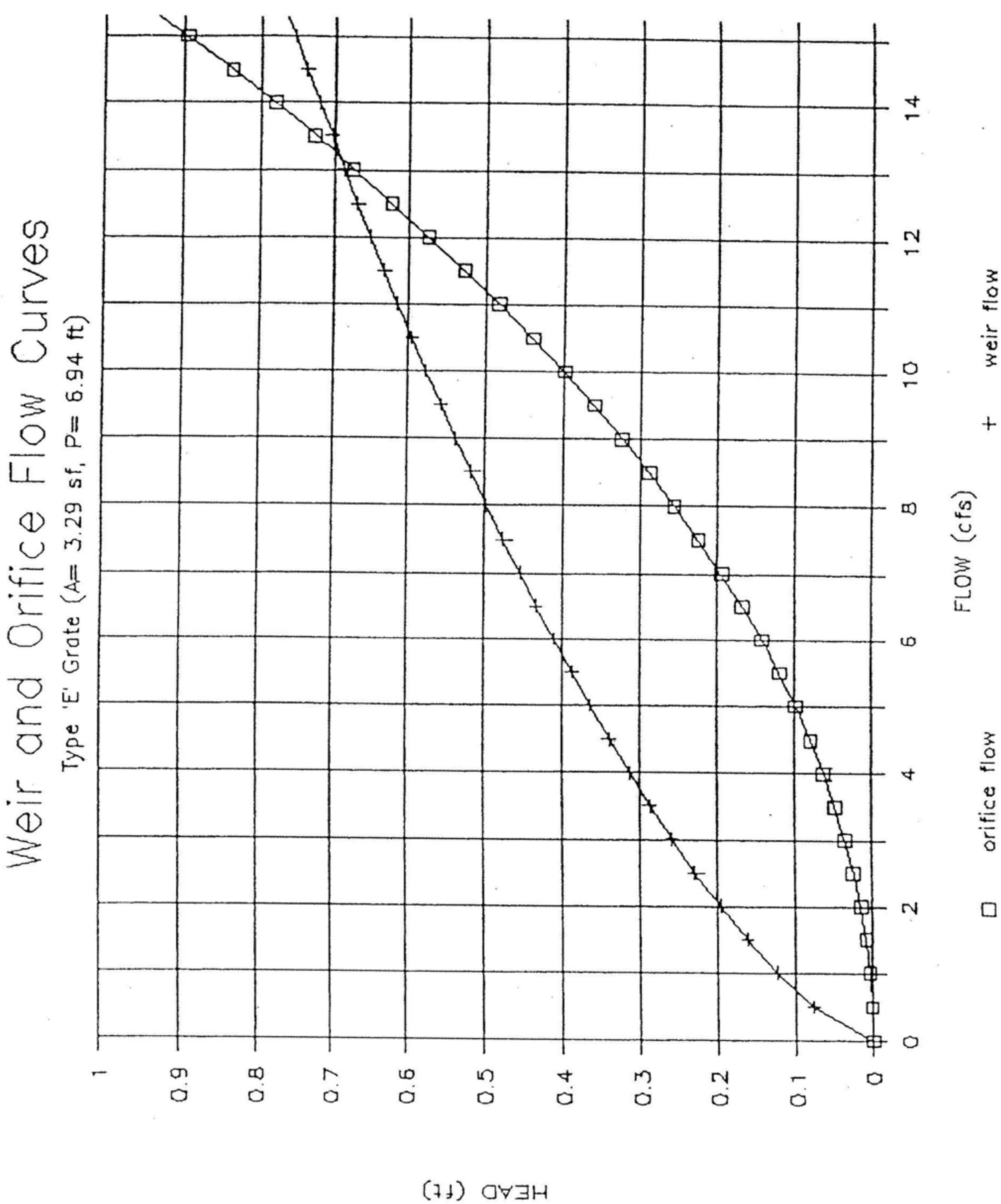


Figure 5-2 Type 'E' Grate - Standard 20.02

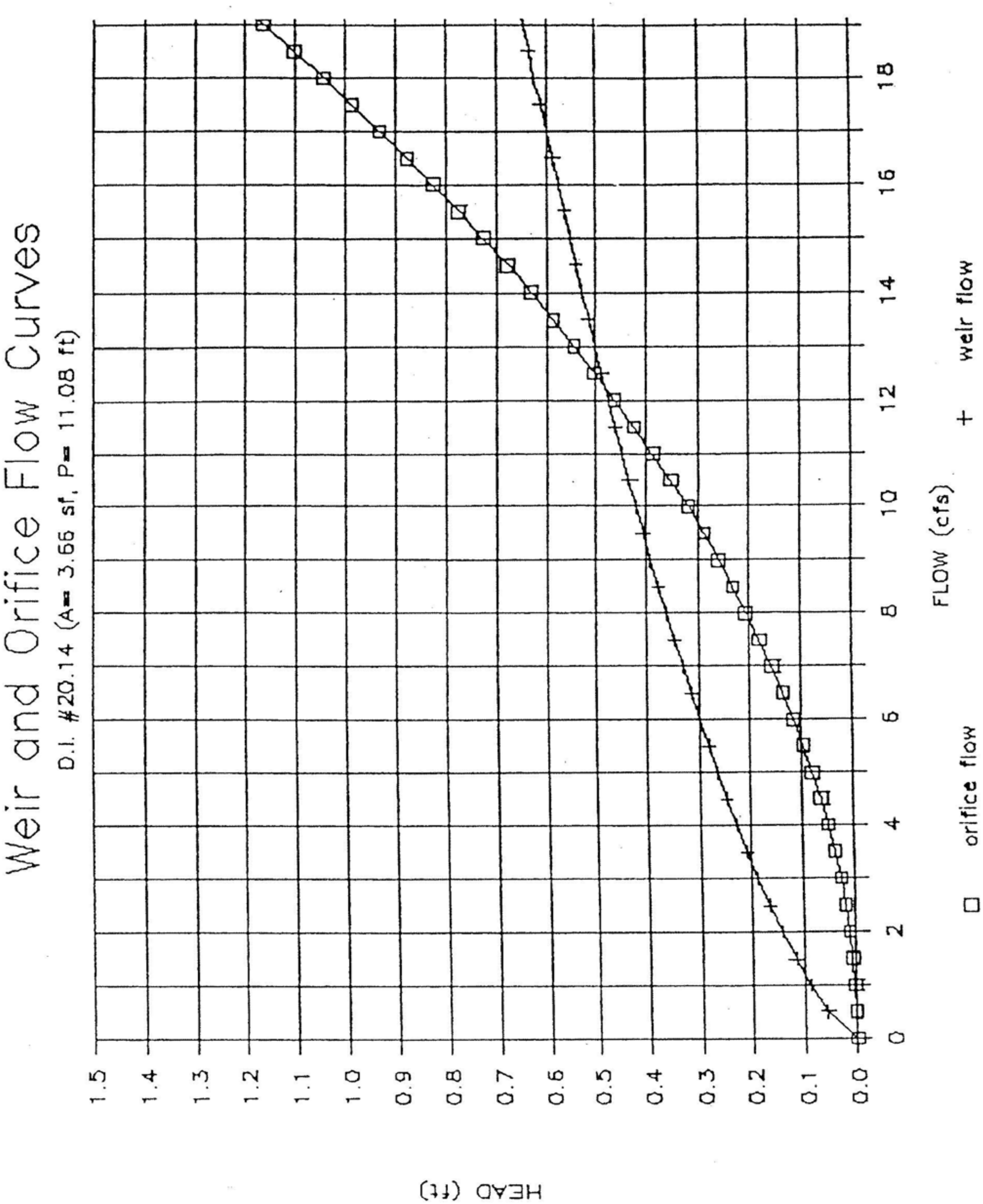


Figure 5-3 Standard Drop Inlet Grate - Standard 20.14

Drop Inlet (Std.20.14) 5.6.2.2

For drop inlet STD.20.14, both the weir and orifice equation should be analyzed:

(orifice)

$$\mathbf{D} = \left[\frac{\text{Surface } Q_{total}}{(0.6)\ A\ (64.4)^{1/2}}\right]^2 \qquad \mathbf{(5.9)}$$

A = 3.66 sq. ft. (open area of standard drop inlet grate)

(weir)

$$\mathbf{D} = \left[\frac{\text{Surface } Q_{total}}{3.3P}\right]^{2/3} \qquad \mathbf{(5.8)}$$

Solve both equations 5.8 and 5.9 for D, or refer to Figure 5-3. The larger D controls.

Example 2 5.6.2.3

Using the data given for Example 1 in 5.6.1.2, calculate the flow intercepted by type 'E' grate in a sag location.

Given: *Same data given for Example 1 in 5.6.1.2*

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

Solution:

1. *From Example 5.6.1.2, Surface Q_{total} = 4 cfs*

2. *Solve for D using Figure 5-2 or Equation 5-8*

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

3. *Check the spread*

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

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.*

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

Check the spread

$$T = \frac{D}{S_t} = \frac{0.25\ ft}{0.03125\ ft/ft} = 7.9\ ft$$

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

5.7 Combination Inlets

Combination Inlets On Grade 5.7.1

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 design procedures should be followed based on the sue of Figures 5-1, 5-2, and 5-3.

5.8 Hydraulic Gradient

Friction Losses 5.8.1

Energy losses from pipe friction may be determined by rewriting the Manning Equation.

$$\mathbf{S_t = [Qn/1.486\ A(R)^{2/3}]^2} \quad \textbf{(5.8)}$$

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

$$\mathbf{H_f = S_f\ L} \quad \textbf{(5.9)}$$

Where: H_f *= friction head loss (ft)*
S_f *= friction slope (ft/ft)*
L = length of outflow pipe (ft)

Velocity Head Losses 5.8.2

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.

$$\mathbf{H = KV^2\ /2g} \quad \textbf{(5.10)}$$

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)

Entrance Losses 5.8.3

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

$$\mathbf{H_{tm} = V^2\ /2g} \quad \textbf{(5.11)}$$

$$\mathbf{H_e = KV^2\ /2g} \quad \textbf{(5.12)}$$

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.)

Junction Losses 5.8.4

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.*

$$\mathbf{H_{j1} = (V_3^2)\ (outflow)/2g} \quad \textbf{(5.13)}$$

Junction Losses (continued)

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 bed (approaching 90°) the more sever the energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber by the formula:

$$\mathbf{H_b = K(V^2)\ (outlet)/2g} \qquad \mathbf{(5.14)}$$

Where: H_b = bend head loss (ft)
K = junction loss coefficient

Table 5.3 below lists the values of K for various junction angles.

Table 5-3

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 intersection is as follows:

Junction Losses (continued)

The following equation can be used to calculate these losses:

$$H_{j2} = [(Q_4 V_4^2)-(Q_1 V_1^2)-(Q_2 V_2^2) + (KQ_1 V_1^2)]/(2gQ_4) \quad (5.15)$$

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 compound 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.

Summary 5.8.5

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.

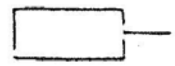
In Figure 5-4 on page 5-22 is a summary of energy losses which should be considered. Following this in Figure 5-5 is a sketch showing the proper and improper use of energy losses in developing a storm drain system.

SUMMARY OF ENERGY LOSSES

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

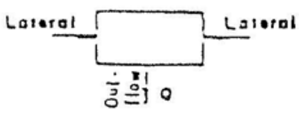
TERMINAL JUNCTION LOSSES
(at beginning of run)

Where g = gravitational constant, 32.2 feet per second per second.



$$H_e = 0.5\frac{V^2}{2g}$$

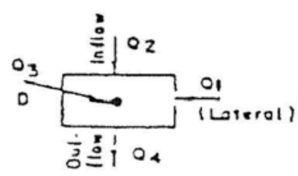
ENTRANCE LOSSES
(for structure at beginning of run)
Assuming square-edge



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

JUNCTION LOSSES

Use only where flows are identical to above, otherwise use H_{j2} Equation.

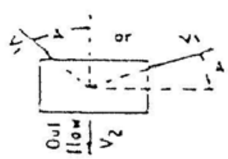


$$H_{j2} = \frac{Q_4V_4^2 - Q_1V_1^2 - Q_2V_2^2 + KQ_1V_1^2}{2gQ_4}$$

JUNCTION LOSSES
(After FHWA)

Total losses to include H_{j2} plus losses for changes in direction of less than 90° (H_b).

Where K = Bend loss factor (Figure 5-6 page 5-38)
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 in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSS (H_f)

$H_f = S_f \times L$

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

$$S_f = \left(\frac{Qn}{1.486\,AR^{\frac{2}{3}}}\right)^2$$

Where Q = discharge of conduit
n = Mannings coefficient of roughness (use 0.013 for R.C. Pipes)
A = area of conduit
R = hydraulic radius of conduit (D/4 for round pipe)

TOTAL ENERGY LOSSES AT EACH JUNCTION

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

Figure 5-4 Summary of Energy Losses

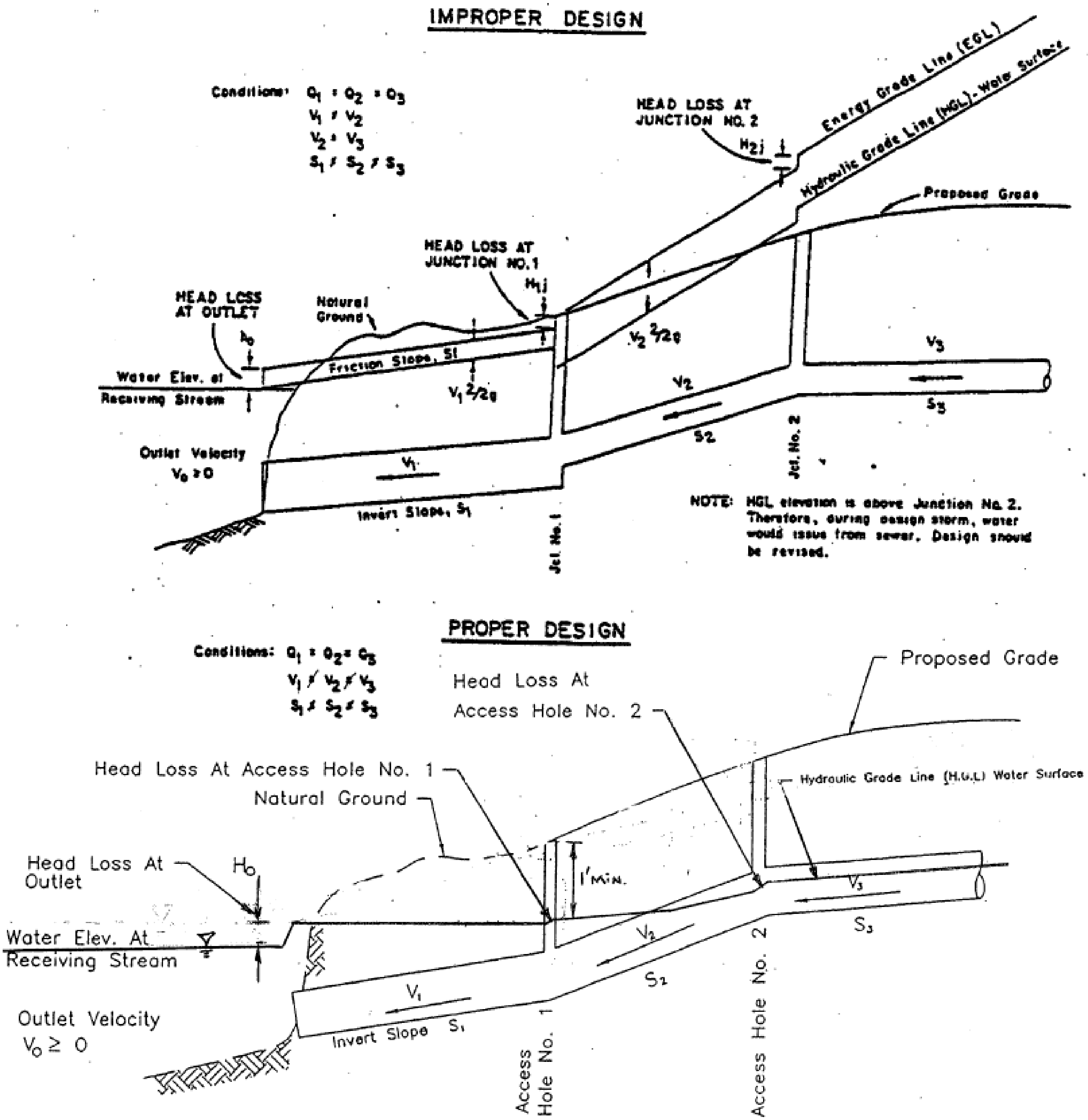


Figure 5-5 Energy and Hydraulic Grade Lines for Storm Sewer Under Constant Discharge

5.9 Storm Drains

Introduction 5.9.1

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 of 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 Formula is recommended for capacity calculations.

Design Criteria 5.9.2

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

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

2. *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 sewer is flowing full, whichever is greater.*

Systems should be designed for non-pressure conditions, such that the Hydraulic Grade Line is lower than the crown of the pipe. When non-pressure conditions cannot be attained and 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 foot below surface inlet elevation. As a general rule, minor losses should be considered when the 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. However, pressure flow condition requires special treatment of joints. Pipes that are in pressure flow and have been approved by the Town Engineer must use O-Ring ASTM C-443 for pipes that are surcharged.

Capacity 5.9.3

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 Formula and it is expressed by the following equation:

$$V = [1.486\ R^{2/3}\ S^{1/2}]/n \qquad (5.16)$$

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

In terms of discharge, the above formula becomes:

$$Q = [1.486\ AR^{2/3}\ S^{1/2}]/n \qquad (5.17)$$

Where: Q = rate of flow (cfs)
A = cross sectional area of flow (ft^2)

For pipes flowing full, the above equations become:

$$V = [0.590\ D^{2/3}\ S^{1/2}]/n \qquad (5.18)$$

$$Q = [0.463\ D^{2/3}\ S^{1/2}]/n \qquad (5.19)$$

Where: D = diameter of pipe (ft)

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

$$H_f = (2.87\ n^2V^2L/S^{4/3} \qquad (5.20)$$

$$H_f = (29\ n^2\ LV^2)/(R^{4/3})(2g) \qquad (5.21)$$

Where: Hf = total head loss due to friction (ft)
L = length of pipe (ft)
g = acceleration due to gravity = 32.2 ft/sec^2

Nomographs and Table 5.9.4

The nomographs solution of Manning's formula for full flow in circular storm drain pipes is shown of Figures 5-6, 5-7, and 5-8. Figure 5-9 has been provided to solve the Manning's equation for part full flow in storm drains.

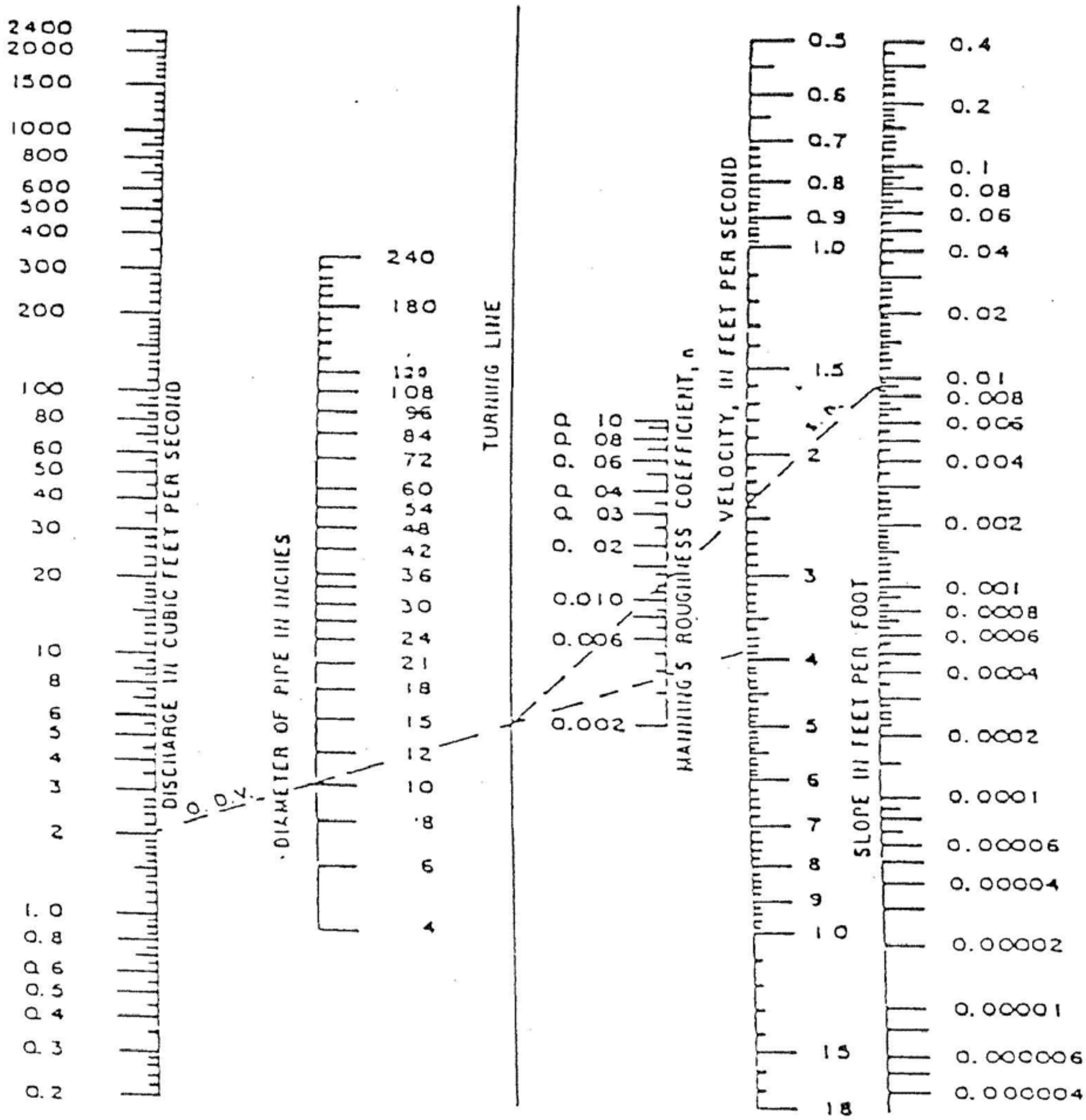


Figure 5-6 Nomograph for Solution of Manning's Formula for Flow in Storm Sewers

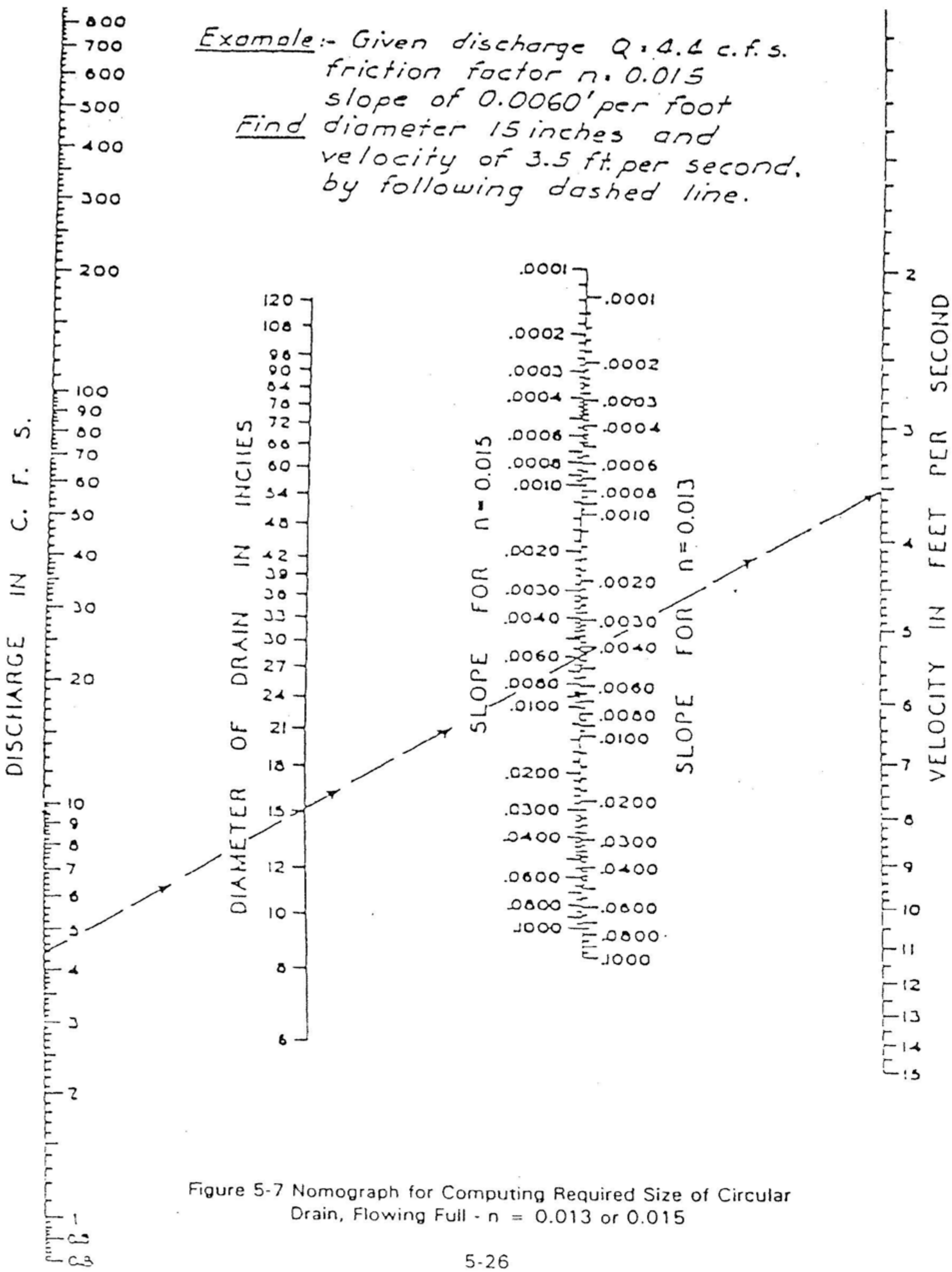


Figure 5-7 Nomograph for Computing Required Size of Circular Drain, Flowing Full - n = 0.013 or 0.015

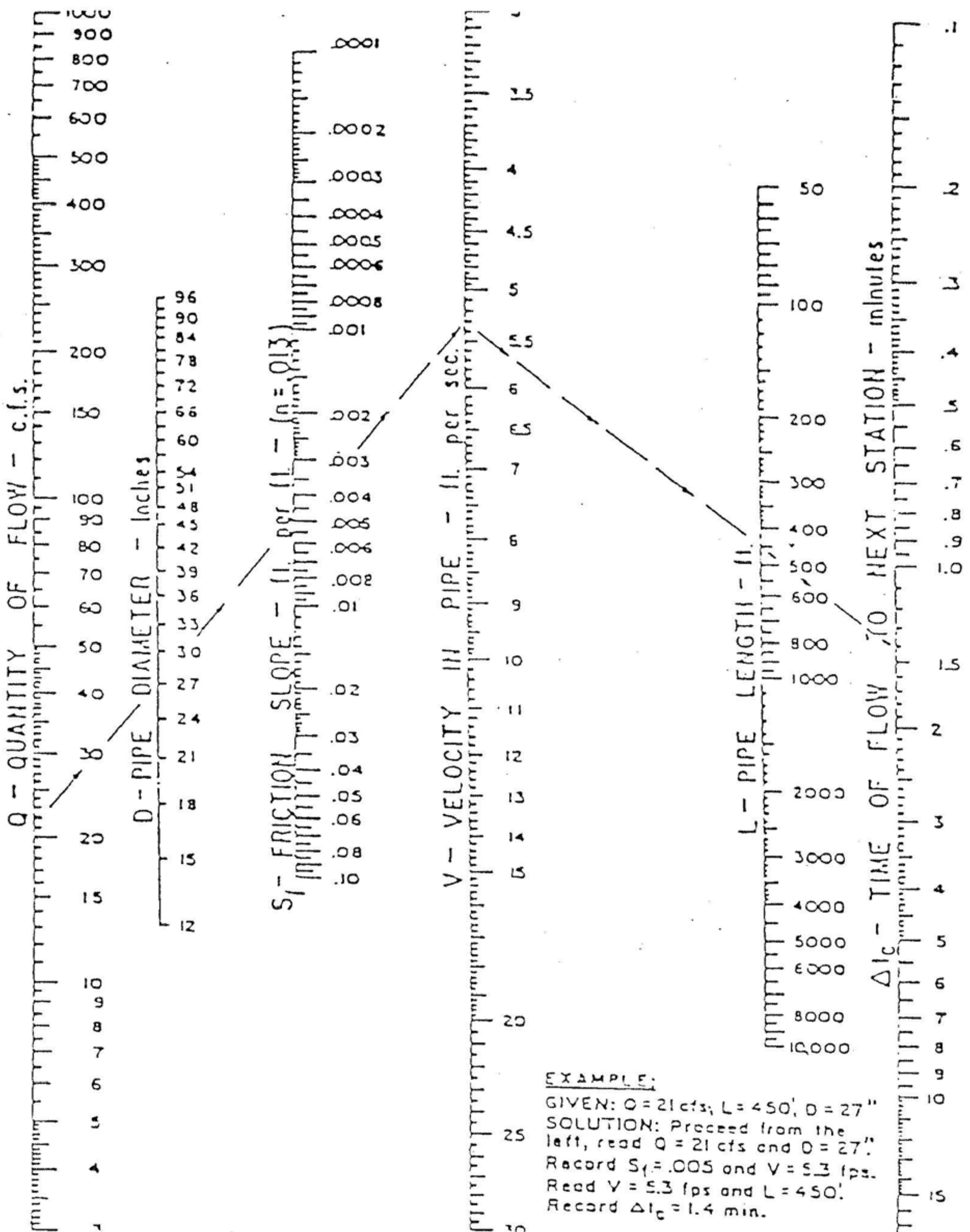
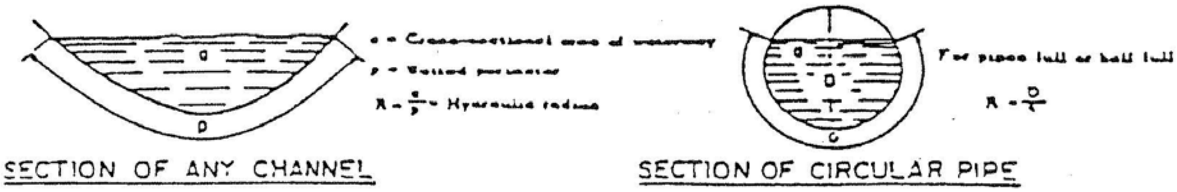
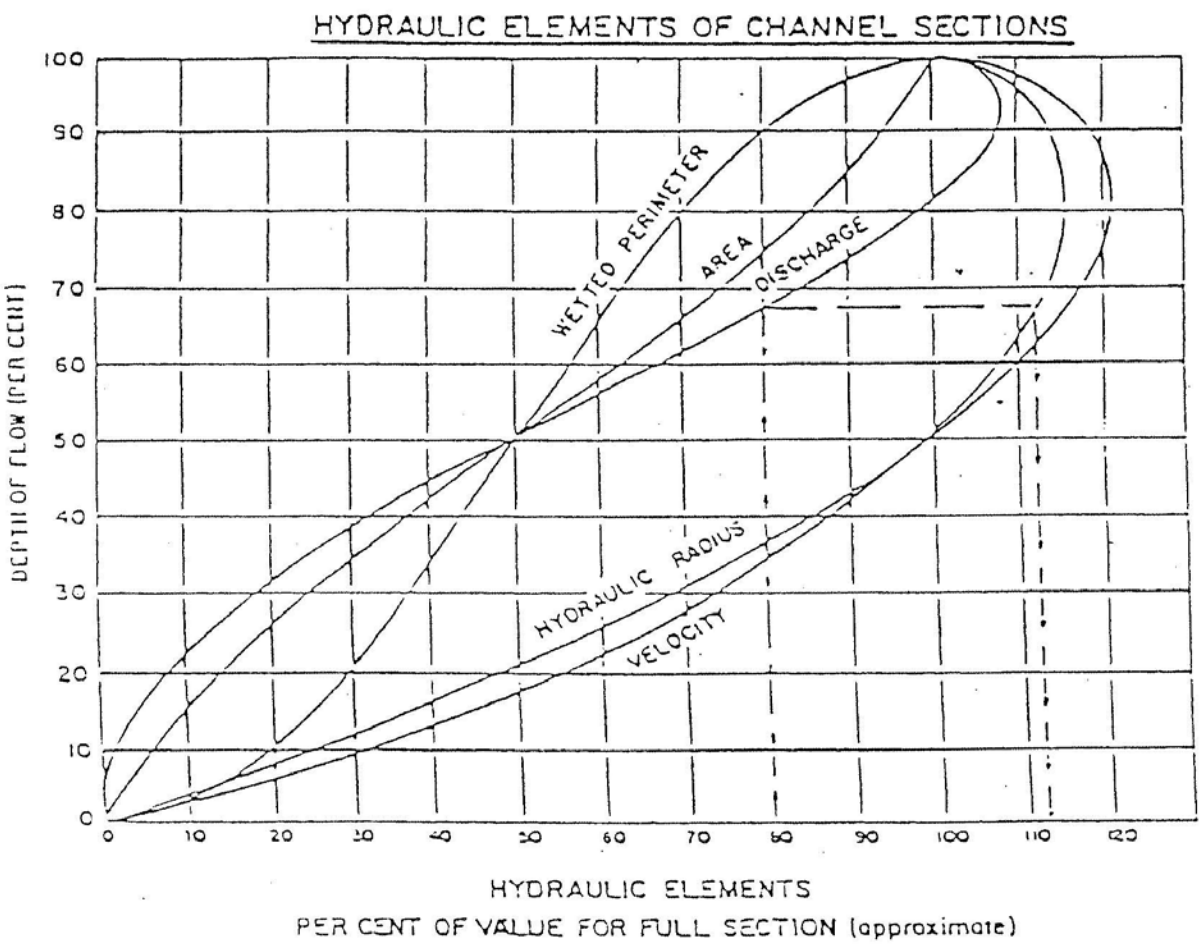


Figure 5-8 Concrete Pipe Flow Nomograph





V = Average of mean velocity in feet per second
Q = Discharge of pipe or channel in cubic feet per second
S = Slope of hydraulic grade line

Figure 5-9 Values of Various Elements of Circular Section for Various Depths of Flow

Hydraulic Grade Lines 5.9.5

In calculating the hydraulic grade line within a closed storm sewer 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, whichever is higher, is the outlet hydraulic grade line. Hydraulic grade lines will be required only as requested on a case by case basis.

Design Procedure 5.9.5.1

The head losses are calculated beginning from the control point to the first junction and the procedure is repeated from the next junction. The computation for outlet control may be tabulated using Figure 5-10 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:*

$$S_f = Q^2n/1.486AR^{2/3} \qquad \textbf{(5.22)}$$

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:*

$$H_o = 0.25\ (V_o^2)/2g \qquad \textbf{(5.23)}$$

   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.*

Design Procedures (continued)

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

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

13. *Enter in Column 13 the controlling expansion loss (H_j) using the formula $H_j = 0.35 (V_j^2) / 2g$.*

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 ($H\Delta$) calculated by using the formula:*

$$H\Delta = KV_j^2/2g \qquad (5.24)$$

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_j), and Column 15 ($H\Delta$).*

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.*

Minimum Grade 5.9.6

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

The minimum slopes are calculated by the modified Manning formula:

$$\mathbf{S = (nV^2)/(2.208\ R^{4/3})} \qquad \mathbf{(5.25)}$$

Design Procedures 5.9.7

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 sewer drainage system establishing the following design data:*
    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.*

4. *The storm drain design computation sheet (Figure 5-11) can be used to summarize the design computations.*

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	$1.3\, H_t$	$0.5\, H_t$			
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
70° K = 0.64	30° K = 0.35
60° K = 0.56	15° K = 0.19

Figure 5-10 Hydraulic Grade Line Computation Form

Storm Sewer Computation Sheet

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 5-11 Storm Sewer Computation Form

Rational Method Example 5.9.8

The following example will illustrate the hydrologic calculations needed for storm drain design using the rational formula (see Hydrology for Rational Method description and procedures). Figure 5-12 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 5-4
Hydrologic Data

Inlet[a]	Drainage Area (acres)	Time of Concentration (minutes)	Rainfall Intensity (Inches/hr)	Runoff Coefficient	Inlet FlowRate[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 5-12.*
[b] *Calculated using the Rational Equation (see Chapter 3, 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 5-5

Storm Drain System Calculations

Storm Drain Segment	Tributary Area (acres)	Time of Concentration (minutes)	Rainfall Intensity (inches/hr)	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
$1_3 – M^2$	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

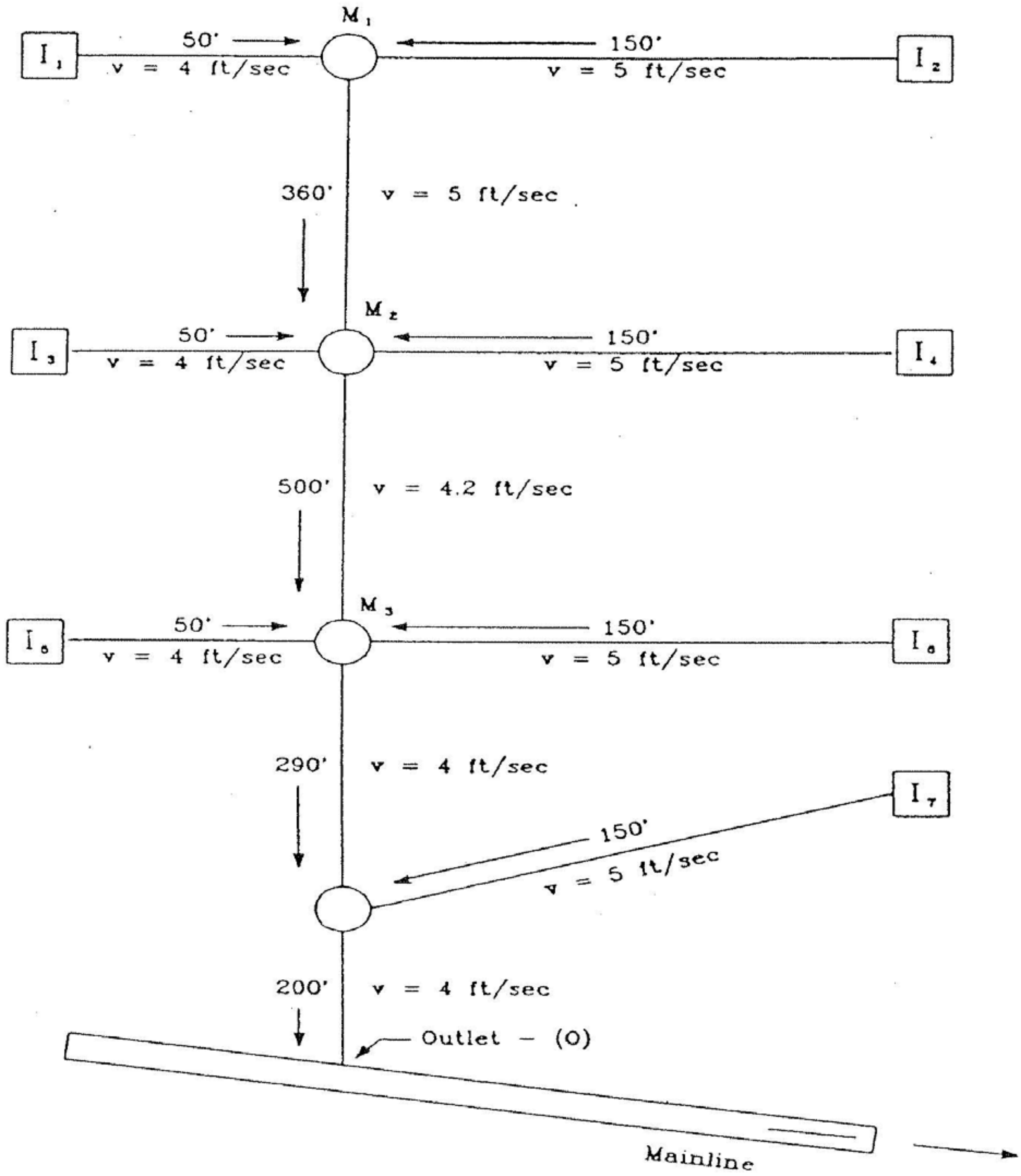


Figure 5-12 Hypothetical Storm Drain System Layout

5.10 Computer Programs

To assist with storm drain system design a microcomputer software model has been developed for the computation of hydraulic gradeline. The computer model has been attached to the program HYDRA, which ahs been adopted by the Federal Highway Administration organized Pooled Fund Study on Integrated Drainage System, as the program for storm drain design and analysis. The model developed in this study, called HYGRD, would allow a user to check design adequacy and also to analyze the performance of a storm drain system under assumed inflow conditions.

For more information about the HYGRD computer program the following publication is available from the National Technical Information Service, Springfield, VA 22161:

Microcomputers Software for Storm Drain Hydraulic Gradeline Computation

by Shaw L. Yu and James Y. Li, Virginia Transportation Research Council

This document describes the concepts and equations in the model, implementation of the model, example applications, and typical model output.

The HYDRA computer program is integrated into the Federal Highway Administration's HYDRAIN computer model system which is available from McTrans; Software, University of Florida, 512 Weil Hall, Gainesville, Florida 32611.

Flood Study 5.11

In compliance with the Federal Emergency Management Agency National Flood Insurance Program and Regulations, the applicant shall:

A. *When development impacts an existing floodplain and causes the water elevation to rise one (1) or more feet during the one hundred (100) year storm, submit plans and appropriate flood studies for any proposed construction activity in the floodway to the Federal Emergency Management Agency for approval prior to submission to the Town of Waxhaw for consideration;*

B. *Ensure that every computer program used to perform hydrologic and hydraulic analysis shall be made using the hydraulic computer model used to develop the base flood elevations shown on the effective Flood Insurance Rate Map;*

C. *Be responsible for obtaining from the Federal Emergency Management Agency a letter of Map Revision (LOMAR), as applicable and be responsible for payment of all fees associated with said map revisions.*

Prior to issuing a zoning permit for any structure located or appearing to be located within a floodplain, the Administrator shall require the applicant to submit a plot plan which shows the location of the 100 year floodplain contour [as mapped by the Federal Emergency Management Agency (FEMA)] or a statement that the entire lot lies within a floodplain. The plot plan must be prepared by or under the direct supervision of a registered land surveyor or professional engineer and certified by the same and show the location of the floodway as identified by FEMA.

Where base flood elevation data is provided in accordance with Section 6.5.3.1, the application for a development permit within the Zone A on the Flood Insurance Rate Map shall show:

A. *The elevation (in relation to mean sea level) of the lowest floor (including basement) of all new and substantially improved structures, and*

B. *If the structure has been floodproofed6.5.3.1, the elevation (in relation to mean sea level to which the structure was floodproofed).*

Where the base flood elevation data is not provided, the application for a development permit must show construction of the lowest floor at least two feet above the highest adjacent grade or provide a localized 100+1 flood study certified by a professional engineer. Floodplain elevations (i.e., F.R.Es) shall be specified for each adjacent lot affected by the established flood line and elevations shall be at least one (1) foot above the base flood elevation. For culvert or roadway crossings, the floodplain elevation for lots located adjacent the culvert, shall be one (1) foot above the roadway crest (i.e., low point of the road) or one (1) foot above the base flood elevation, whichever is more stringent.

100+1 flood studies shall be conducted for all drainage basins that carry 50+ cf/s (cubic feet per second) during the 100-year storm and do not have established FEMA base flood elevations. Cross sections of the basin shall be certified by a registered land surveyor and be at a maximum of two-hundred (200) foot intervals and can be generated from a field run topographic map.

Where any watercourse will be altered or relocated as a result of proposed development, the application for a zoning permit shall include or be accompanied by: a description of the extent of watercourse alteration or relocation; an engineering report on the effects of the proposed project on the flood-carrying capacity of the watercourse and the effects to properties located both upstream and downstream; and a map showing the location of the proposed watercourse alteration or relocation.

When a structure is floodproofed, the application shall be accompanied by a certificate from a registered professional engineer or architect stating that the non-residential floodproofed structure meets floodproofing criteria.

A floor elevation or floodproofing certification is required after <u>the lowest floor is completed</u>, or in instances where the structure is subject to the regulations applicable to Coastal High Hazard Areas, after placement of the horizontal structural members of the lowest floor. Within twenty-one (21) calendar days of establishment of the lowest floor elevation, or floodproofing by whatever construction means, or upon placement of the horizontal structural members of the lowest floor, whichever is applicable, it shall be the duty of the permit holder to submit to the Administrator a certification of the elevation of the lowest floor, floodproofed elevation, or the elevation of the bottom of the horizontal structural members of the lowest floor, whichever is applicable, as built, in relation to mean sea level. Said certification shall be prepared by or under the direct supervision of a registered land surveyor or professional engineer and certified by the same. When floodproofing is utilized for a particular building, said certification shall be prepared by or under the direct supervision of a registered land surveyor or professional engineer and certified by the same. Any work done within the twenty-one (21) day calendar period and prior to submission of the certification shall be at the permit holder's risk. The Administrator shall review the floor elevation surveyed as submitted. Deficiencies detected by such review shall be corrected by the permit holder immediately and prior to further progressive work being permitted to proceed. Failure to submit the survey or failure to make said corrections required hereby, shall be cause to issue a stop-work order for the project.

References

U.S. Department of Transportation, Federal Highway Administration, 1984. Drainage of Highway Pavements. Hydraulic Engineering Circular No.12.

CHAPTER 6

DESIGN OF CULVERTS

Chapter Table of Contents

6.1 Overview

Definitions 6.1.1

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

Performance Curves 6.1.2

Performance curves should be developed for all culverts for evaluating the hydraulic capacity of a culvert for various headwaters. These curves will display the consequence 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 estimated design discharge will have on the culvert design.

6.2 Symbols and Definitions

Symbol Table

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of wide use in many culvert design publications.

Table 6-1

SYMBOLS AND DEFINITIONS

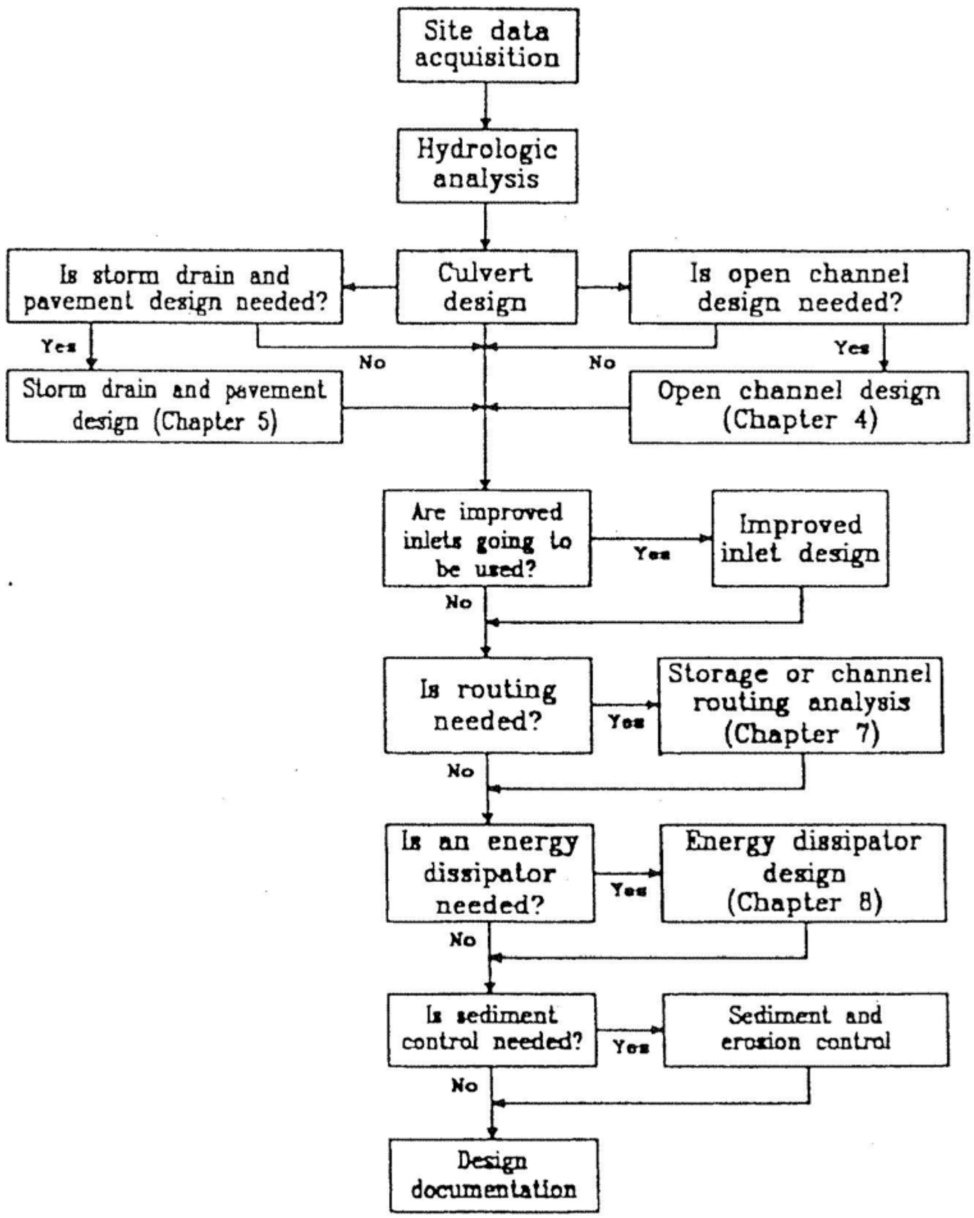
Symbol	Definition	Units
A	Area of cross section of flow	sq. ft.
B	Barrel width	ft
C_d	Overtopping discharge coefficient	--
D	Culvert diameter or barrel depth	in/ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration due to gravity	ft/s^2
H	Total energy loss	ft
H_e	Entrance head loss	ft
H_f	Friction headloss	ft
H_o	Height of hydraulic grade line above outlet invert	ft
HW	Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)	ft
K_e	Inlet loss coefficient	--
L	Length of culvert	ft
P	Empirical approximation of equivalent hydraulic grade line	ft
Q	Rate of discharge	cfs
S	Slope of culvert	ft/f
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
Vc	Critical Velocity	ft/s

6.3 Culvert Design Procedure Flowchart

Purpose And Use 6.3.1

The procedure 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.

Design Flowchart 6.3.2



6.4 Concept Definitions

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. Appendix A at the end of this chapter gives a series of critical depth charts for the different shapes encountered in culvert design.*
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. This type of flow will exist in a culvert operating on a steep slope provided the culvert is sufficiently long.*
Freeboard	*Freeboard is an additional depth 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.*
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.*
Submerged	*Submerged inlets are those inlets having a headwater greater than (1.5D).*
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 inet and thus increases the capacity of culverts at the inlet.*
Invert	*Invert refers to the flowline of the culvert (inside bottom).*
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.*

6.5 Engineering Design Criteria

Introduction 6.5.1	*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.*

Criteria 6.5.2

Engineering Aspects	*• Flood frequency* *• Velocity limitations*
Site Criteria	*• Length and slope* *• Debris control*
Design Limitations	*• Headwater* *• Tailwater conditions* *• Ground cover* *• Utility Conflicts* *• Regulated Floodway Requirements*
Design Options	*• Culvert inlets* *• Inlets and headwalls* *• Wingwalls and aprons* *• Improved inlets* *• Material selection* *• Culvert skews* *• Culvert sizes*
Related Designs	*• 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 sitting and design.

Flood Frequency 6.5.3

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 upstream (backwater) or downstream (redirection of floodwaters) property. On*

Culverts must be designed to accommodate the following minimum flood frequencies.

- *Cross-drainage on thoroughfare classified roadways – 50 year frequency*
- *Cross-drainage on minor roadways – 25 year frequency*
- *Culverts over regulated floodways – 100 year frequency*

In addition, the 100-year frequency storm shall be routed through all culverts as required by the 100 + 1 criteria.

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.

Velocity Limitations 6.5.4

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. It velocities exceed permissible velocities for the various types of nonstructural outlet lining material available, the installation of structural energy dissipators is required. The Maximum allowable velocity within corrugated metal pipe is 10 fps.

There is no specified 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.

Debris Control 6.5.5

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.

Headwater Limitations 6.5.6

The allowable headwater elevation is determined from an evaluation 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 25-year storm):

* *The allowable headwater for design frequency conditions should allow for the following Up stream 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 set to produce acceptable velocities, or stabilization or energy dissipation should be provided.*

* *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.

Tailwater Considerations 6.5.7

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.

Tailwater Considerations (continued)

If the culvert outlet is operation 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 4, 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.

Culvert Inlets 6.5.8

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 6-2 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 450 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*
Round 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)	
Squared-edged at crown	*0.7*
Side- or slope-tapered inlet	*0.2*

**Note: End Sections conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance.*

Inlets With Headwalls 6.5.9

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) shorten the length of the required structure.

Inlets With Headwalls (continued)

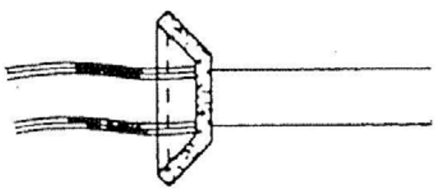
The relative efficiency of the inlet depends on the conduit. Headwalls are required for all metal culverts. The figure below illustrates the use of headwalls and wingwalls. Corrugated metal pipe in a headwall is essentially square-edged with an inlet coefficient of about 0.5.

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

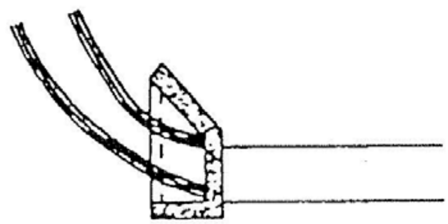
Wingwalls And Aprons 6.5.10

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.

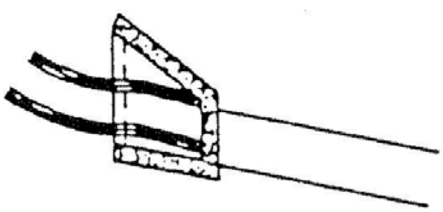
Little increase in hydraulic efficiency is realized with the use of wingwalls, regardless of the pipe material used and, therefore, the use should be justified for other reasons. Wingwalls can be used to increase hydraulic efficiency if designed as a side-tapered inlet (See Section 6.10.6.2 for more information on the design of side-tapered inlets). The figure shown below illustrates several uses of wingwalls.



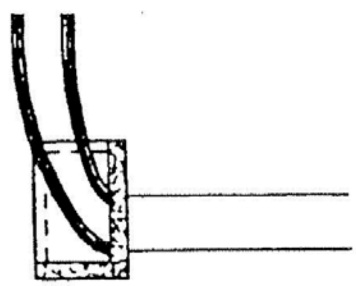
Flow Normal To Embankment – Preferred



Flow Skewed To Embankment – Undesirable



Flow And Culvert Skewed To Embankment – Preferred



Flow Parallel To Embankment – Undesirable

Wingwalls and Aprons (continued)	*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 as least the barrel depth upstream from The entrance, and the top of the apron should not protrude above the normal streambed elevation.*
Improved Inlets 6.5.11	*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 the section 6.10 which describes the design of improved inlets.*
Material Selection 6.5.12	*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, bituminous coated corrugated steel pipe or aluminum pipe is allowed if it has a concrete poured invert.*

Table 6-3 Manning's n Values

Type of Conduit	*Wall & Joint Description*	*Manning's n*
Concrete Pipe	*Good joints, smooth walls*	*0.012*
	Good joints, rough walls	*0.015*
	Poor joints, rough walls	*0.016*
Concrete Box	*Good joints, smooth finished walls*	*0.012*
	Poor joints, rough, unfinished walls	*0.016*
Corrugated Metal Pipe and Boxes Annular Corrugations	*2/3 by 1/2 inch corrugations*	*0.024*
	6 by 1 inch corrugations	*0.025*
	5 by 1 inch corrugations	*0.026*
	3 by 1 inch corruglations	*0.027*
	6 by 2 inch structural plate	*0.034*
	9 by 2 1/2 structural plate	*0.035*
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	*2/3 by 1/2 inch corrugated 24 inch plate width*	*0.018*
Spiral Rib Metal Pipe	*3/4 by 3/4 in recesses at 12 inch spacing, good joints*	*0.018*

Note: For further information, consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page 163.

Outlet Protection 6.5-13

See Chapter 8, Energy Dissipation, for information on the design of outlet protection.

Environmental Considerations 6.5.14

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.

Sites that will cause the least impact on streams and wetlands should be selected. This selection must consider the entire site, including any necessary lead channels, and the materials used for the bottom on the culvert.

6.6 Culvert Flow Controls and Equations

Introduction 6.6.1

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.

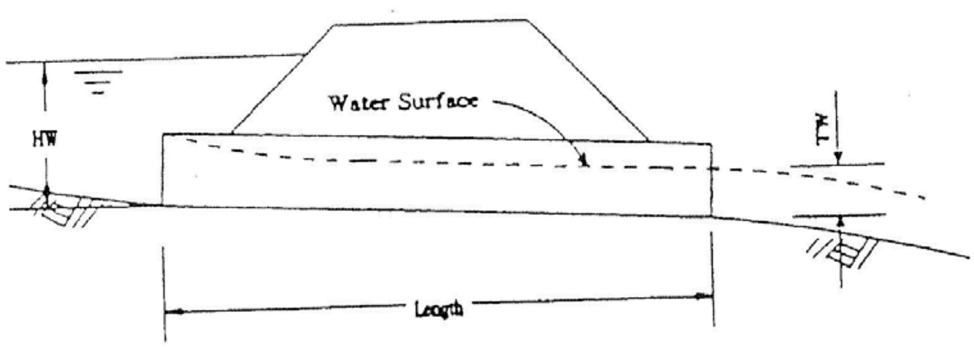
Inlet And Outlet Control 6.6.2

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 Culvert, HDS-5, 1985.

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



Equations 6.6.3

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.

Mild Slope 6.6.3.1

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

$$\mathbf{HW = d_c + V_c^2 /(2g) + H_e + H_f - SL} \qquad \mathbf{(6.1)}$$

Where: HW = headwater depth
d_c = critical depth
V_c = critical velocity
$g = 32.2$ ft/sec^2
H_e = entrance headloss
H_f = friction headloss
S = slope of culvert
L = length of culvert

Tailwater Depth – Outlet Control- The entrance is unsubmerged ($HW \leq 1.5D$), 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.

$$\mathbf{HW = TW + V^2 /(2g) + H_a + H_f - SL} \qquad \mathbf{(6.2)}$$

Where: HW = headwater depth
TW = tailwater at the outlet
V = velocity based on tailwater depth
$g = 32.2$ ft/sec^2
He = entrance headloss
Hf = friction headloss
S = slope of culvert
L = length of culvert

Tailwater Detph > 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.

Mild Slope (continued)

$$HW = H + TW - SL \qquad (6.3)$$

Where: HW = headwater depth
H = total head loss of discharge through culvert
TW = tailwater depth
SL = culvert slope times length of culver

<u>Tailwater < Barrel – Outlet Control</u>- 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 6.4. 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.

$$HW = H + P - SL \qquad (6.4)$$

Where: HW = headwater depth
H = total head loss of discharge through culvert
P = empirical approximation of equivalent hydraulic grade line. P = (d_c + D)/2 if TW depth is less than critical then P = TW.
SL = culvert slope times length of culvert

Steep Slope 6.6.3.2

<u>Tailwater Insignificant – Inlet Control</u>- 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 form 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.

"Slug" Flow 6.6.3.3

<u>Inlet or Outlet Control</u>- 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 hill 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.

"Slug" Flow (continuted)

and 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 6.3 or 6.4 or the outlet control nomographs.

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

6.7 Design Procedures

Procedures

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

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 effect and outlet scours.

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

Tailwater Elevations

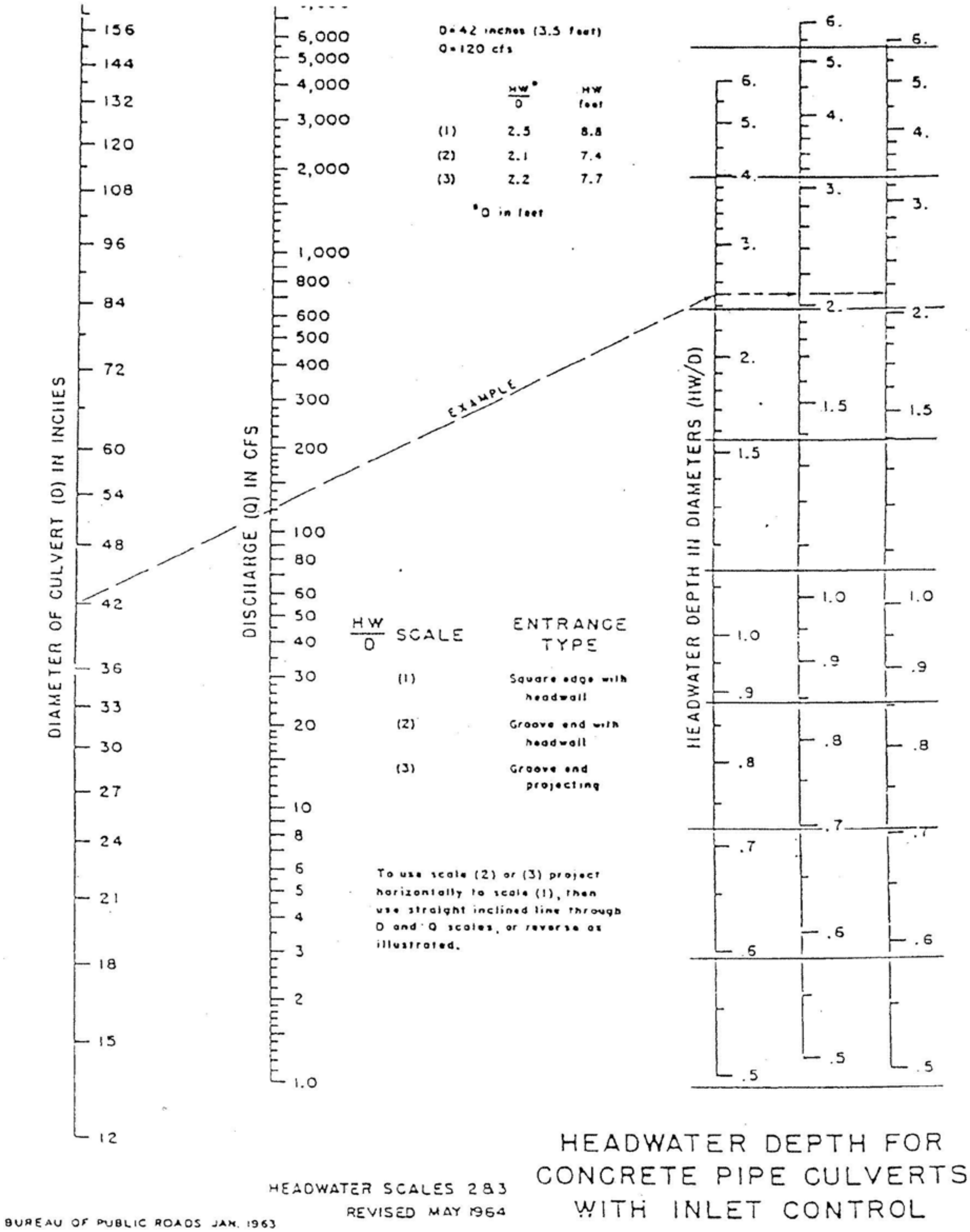
In some cases culverts fail to perform as intended because of tailwater elevations high enough to create backwater. The problem is more severe in areas where gradients are very flat, and in some cases in areas with moderate slopes. Thus, as part of the design process, the normal depth of flow in the 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 elevation 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 than the determination of a cross section downstream where normal flow can be assumed, and a Manning equation calculation. (See Chapter 4, 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.

Nomographs

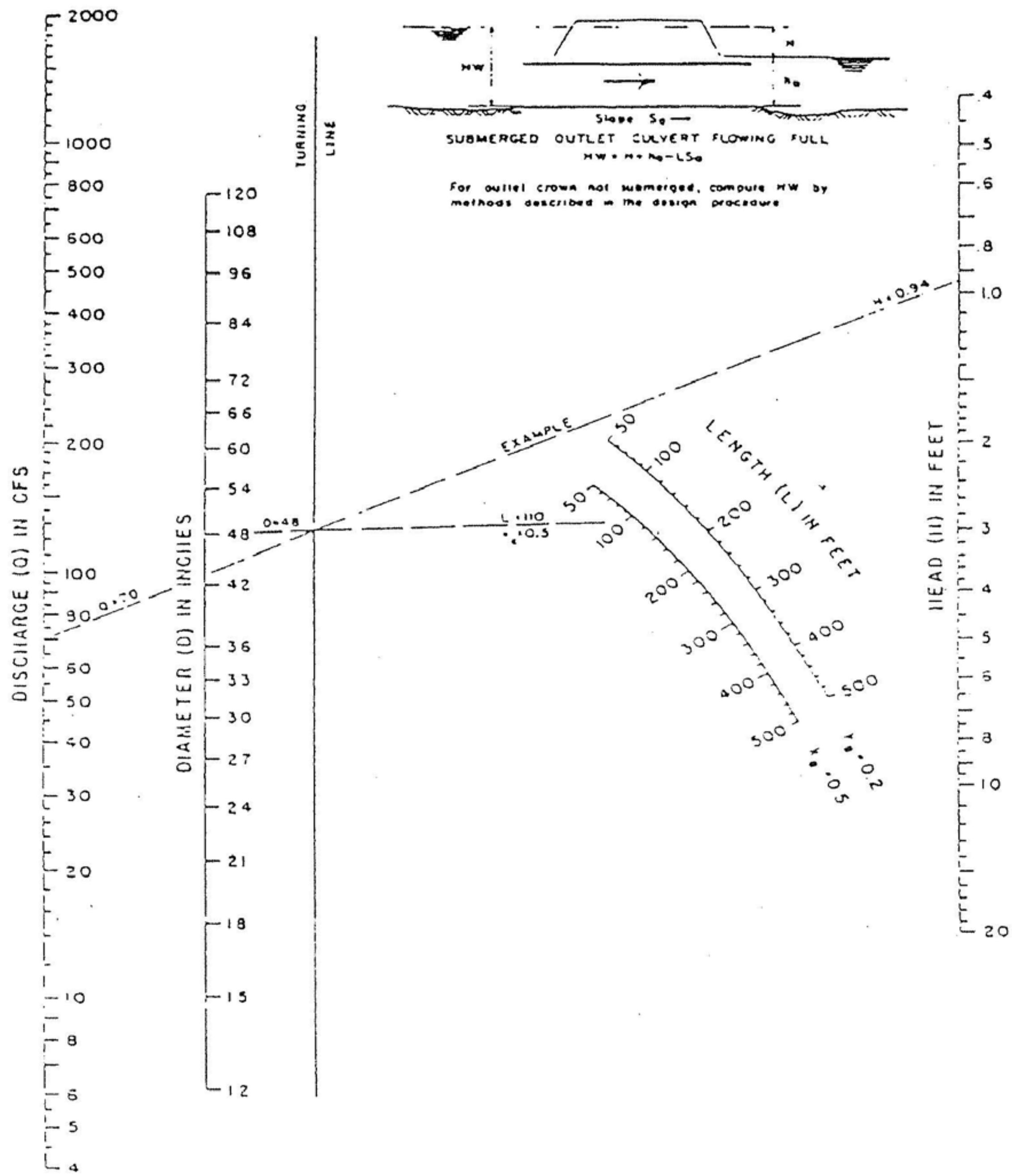
The use of nomographs require a trial and error solution. The solution is quite easy and provides reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained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



D=42 inches (3.5 feet)
Q=120 cfs
HW*/D HW feet
(1) 2.5 8.8
(2) 2.1 7.4
(3) 2.2 7.7
*D in feet
EXAMPLE
DIAMETER OF CULVERT (D) IN INCHES
DISCHARGE (Q) IN CFS
HEADWATER DEPTH IN DIAMETERS (HW/D)
HW/D SCALE
ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove end with headwall
(3) Groove end projecting
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER SCALES 2&3 REVISED MAY 1964
BUREAU OF PUBLIC ROADS JAN. 1963
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Figure 6-1



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

Figure 6-2

Steps In Design Procedure 6.7.4

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

Step	*Action*
(1)	*List of 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)* *Ke = 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 th turning line, and draw a straight line form, the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation.*

$$\mathbf{HW = H + h_o - LS} \qquad \textbf{(6.5)}$$

Where: ho = ½ (critical depth + D), or tailwater depth, whichever is greater.

(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 trial 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.*

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 vs. 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.

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, an 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 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) Combine the inlet and outlet control performance curves to define a sing 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 6.6 to calculate flow rates across the roadway.

$$\mathbf{Q = C_d\ LHW_r^{1.5}} \qquad \mathbf{(6.6)}$$

Where: Q = overtopping flow rate in ft3/s
C_d = overtopping discharge coefficient
L = length of roadway in 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 pages 39-42 in 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.

Storage Routing

A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert site to determine the outflow hydrograph and stage behind the culvert. The procedures for doing this routing are outlined in the publication Hydraulic Design of Highway Culverts, Section V-Storage Routing, HDS No. 5, Federal Highway Administration

6.8 Culvert Design Example

Introduction

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

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.

Data

Input Data

Discharge for 10-yr flood = 70 cfs
Discharge for 100-yr flood = 176 cfs
Allowable HW for 10-yr discharge = 4.5 ft
Allowable HW for 100-yr 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-yr discharge = 3.0 ft
Tailwater depth for 100-yr discharge = 4.0 ft
Tailwater depth is the normal depth in downstream channel
Entrance type = Groove end with headwall

Computations

Steps *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 sq. ft. (for the 10-yr 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: $D = [(14 \text{ sq. ft. x } 4/3.14)^{0.5} \text{ x } 12 \text{ in/ft})]$

   $D = 50.7$ *in*

3. *A grooved end culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 6-1), with a pipe diameter of 48 in. and a discharge of 70 cfs; read a HW/D value of 0.93.*

3. *The depth of headwater (HW) is (0.93) x (4) = 3.72 ft which is less than the allowable headwater of 4.5 ft.*

Computations (continued)

5. *The culvert is checked for outlet control by using Figure 6-2.*

   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 1/2 (critical depth in culvert + D) which ever is greater.

   h_o = 3.0 ft or ho = 1/2 (2.55 + 4.0) = 3.28 ft

   The headwater depth for outlet control is:

   $HW = H + ho – LS$
   $HW = 0.77 + 3.28 – (100) \times (0.0015) = 3.90$ 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-yr 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 <u>100-yr</u> discharge.*

   The allowable headwater for a 100-yr discharge is 7 ft; critical depth in the 48 in. diameter culvert for the 100-yr 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 <u>larger size culvert must be selected.</u>

8. *A 54 in. diameter culvert is tried and found to have a maximum headwater depth of 3.74 ft for the 10-yr discharge and of 6.97 ft for the 100-yr discharge. These values are acceptable for the design conditions.*

9. *Estimate outlet exit velocity. Since this culvert is on 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-yr 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: $V = 176 / (4.5^2/4) = 11.8$ 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 8, Energy Dissipation, for design procedures related to energy dissipators.

10. *The design engineer should check minimum velocities for low frequency flows if the larger storm event (100-year) controls culvert design.*

Figure 6-3 on the next page provides a convenient form to organize culvert design calculations.

PROJECT: ______________________

STATION: ______________ **SHEET** ______ **OF** ______

CULVERT DESIGN FORM

DESIGNER / DATE: ________ / ________

REVIEWER / DATE: ________ / ________

HYDROLOGICAL DATA

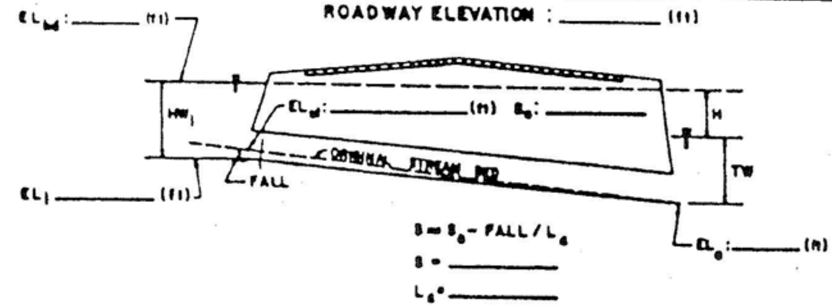
SEE ADD'L SHTS

- ☐ METHOD: ______________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
- ☐ CHANNEL SHAPE: ______________
- ☐ ROUTING: ________ ☐ OTHER: ________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: ________ (ft)



$S = S_o - \text{FALL} / L_a$

S = ________

L_a = ________

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS — INLET CONTROL: HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL: TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL $= HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L) / R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______________

SHAPE: ______________

MATERIAL: ______________ n

ENTRANCE: ______________

Reference: USDOT, FHWA, HDS-5 (1985)

Figure 6-3

6.9 Long Span Culverts

Introduction 6.9.1

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 structures: (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 ft to 40 ft.

Structural Aspects 6.9.2

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 back-filling.

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.

Hydraulic Considerations 6.9.3

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.

6.10 Design of Improved Inlets

Introduction 6.10.1

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.

Outlet Control 6.10.2

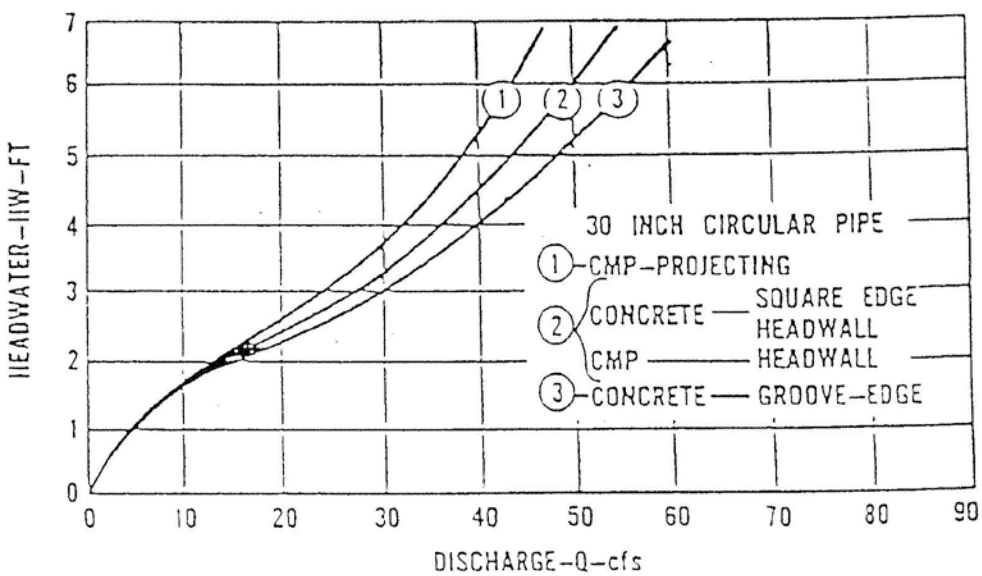
The flow condition for outlet control may be full or partly full for all or part of the culvert length. The design discharge usually results in full flow. Inlet improvements in these culverts reduce the entrance losses, which are only a small portion of the total headwater requirements. Therefore, only minor modifications of the inlet geometry (which result in little additional cost) are justified.

Inlet Control 6.10.3

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.

Common Entrances 6.10.4

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.

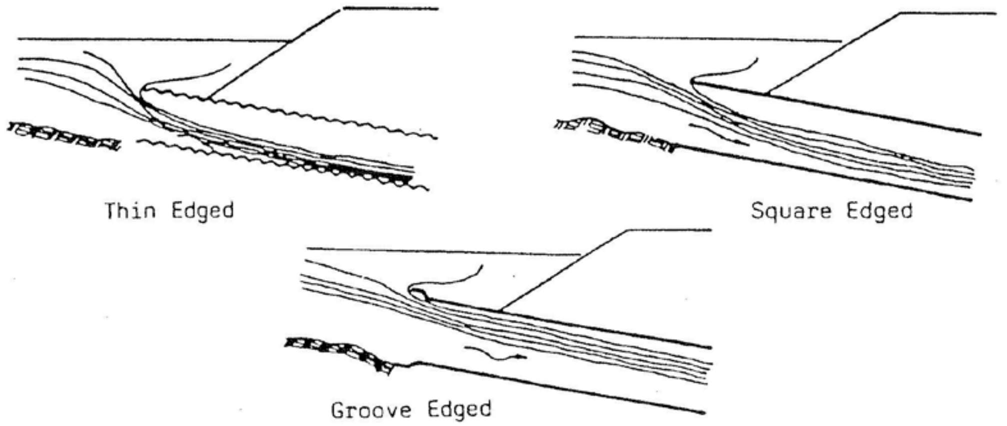


PERFORMANCE CURVES INLET CONTROL

Capacity Determinations 6.10.4

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 that 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.



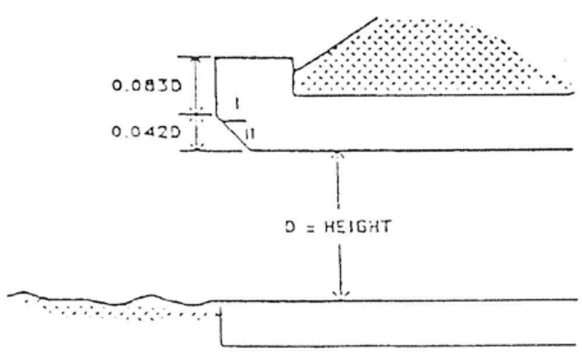
Schematic Flow Contractions for Conventional Inlets

Improved Inlets 6.10.5

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.

Bevel-edged Inlet 6.10.6.1

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 construction.



BEVELED EDGE

Bevel-edged Inlet (continued)

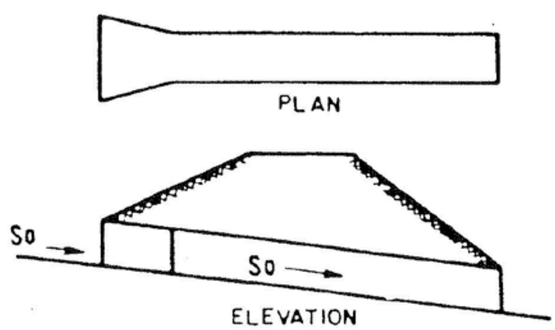
Adding bevels to a conventional culvert design with a square-edged inlet increases culvert capacity by 5 to 20 percent. The higher increase results from comparing a bevel-edged inlet with a square-edged inlet at high headwaters. The lower increase is the result of comparing inlets with bevels, with structures having wingwalls of 30 to 45 degrees. Although the bevels referred to in this 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. The 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, ke, is reduced from 0.5 for a square edge to 0.2 for beveled edges.

Side-tapered Inlet 6.10-6.2

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 the sidewalks. 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 wingwalls 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 culvert installation, a side-tapered inlet should be considered.



Slope-Tapered Inlet 6.10.6.3

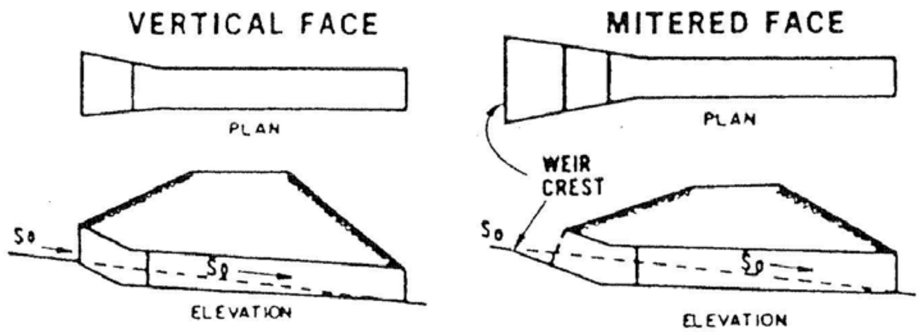
A slope-tapered inlet is the third degree of improvement. Its tapered advantage over the side-tapered inlet without a depression is that more head is available at the inlet. This is accomplished by incorporating a fall in the enclosed entrance section.

Slope-Tapered Inlet (continued)

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 increase capacities is possible.

Side- and slope-tapered inlet 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 pages 65-93, FHWA, HDS-5, 1985.



Improved Inlet Performance 6.10.6.4

The two tables below compare the inlet control performance of the different inlet types. The first table shows the increase is 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 6-4

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*

6.11 Design Procedures for Beveled-Edged Inlets

Introduction 6.11.1

This section will outline procedures and charts to use when incorporating bevel edged inlets in the design of culverts. Those designers interested in using side- and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the U.S. Department of Transportation publication Hydraulic Engineering Circular No. 5 entitled, Hydraulic Design of Highway Culverts.

Design Figures 6.11.2

Four inlet control figures for culverts with beveled edges are included in Appendix B at the end of this chapter.

Figure	*Use for*
1	*90° headwalls (same for 90° wingwalls)*
2	*skewed headwalls*
3	*wingwalls with flare angles of 18 to 45 degrees*
4.	*circular pipe culverts with beveled rings*

Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier.

Note: Figures 2, 3, and 4 apply only to bevels having either a 33° angle (1:5:1) or a 45° angle (1:1).

For box culverts, the dimensions of the bevels to be used are based on the culvert dimensions. The top bevel dimension is determined by multiplying the height of the culvert by a factor. The side bevel dimensions are determining by multiplying the width of the culvert by a factor. For a 1:1 bevel, the factor is 1/2 inch/ft. For a 1.5:1 bevel the factor is 1 inch/ft.

For example the minimum bevel dimensions for a 8 ft x 6 ft box culvert with 1:1 bevels would be:

Top Bevel D = 6 ft x 1/2 inch/ft = 3 inches

Side Bevel B = 8 ft x 1/2 inch/ft = 4 inches

For a 1.5:1 bevel similar computations would result in D = 6 and B 8 inches.

Design Figure Limits 6.11.4

The improved inlet design figures are based on research results from culvert models with ratios of barrel width, B, to depth, D, of 0.51 to 2.1.

For box culverts with more than one barrel, the figure are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

For example, in a double 8 ft x 8 ft box culvert:

Top Bevel- is proportioned based on the height of 8 ft which results in a bevel of 4 inches for the 1:1 bevel and 8 inches for the 1.5:1 bevel.

Side Bevel- is proportioned based on clear width of 16 ft which results in a bevel of 8 inches for the 1:1 bevel and 16 inches for the 1.5:1 bevel.

Area Ratios 6.11.5

The ratio of the inlet face area to the barrel area remains the same as for a single barrel culvert. Multibarrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

Multibarrel Installations 6.11.6

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

Skewed Inlets 6.11.7

It is recommended that Figure 3 in Appendix B for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in under design due to a greatly reduced capacity. Skewed inlets should be avoided whenever possible, and should not be used with side or slope-tapered inlets.

6.12 Construction and Maintenance Considerations

An important step in the design process involved identifying whether special provisions are warranted to properly construct or maintain proposed facilities. Maintenance concerns of storm sewer 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.

REFERENCES

American Association of State Highway and Transportation Officials. 1982. Highway Drainage Guidelines.

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Federal Highway Administration. 1985. Hydraulic design of highway culverts. Hydraulic Design Series No. 5.

Federal Highway Administration. 1971. Debris-Control Structures. Hydraulic Engineering Circular No. 9.

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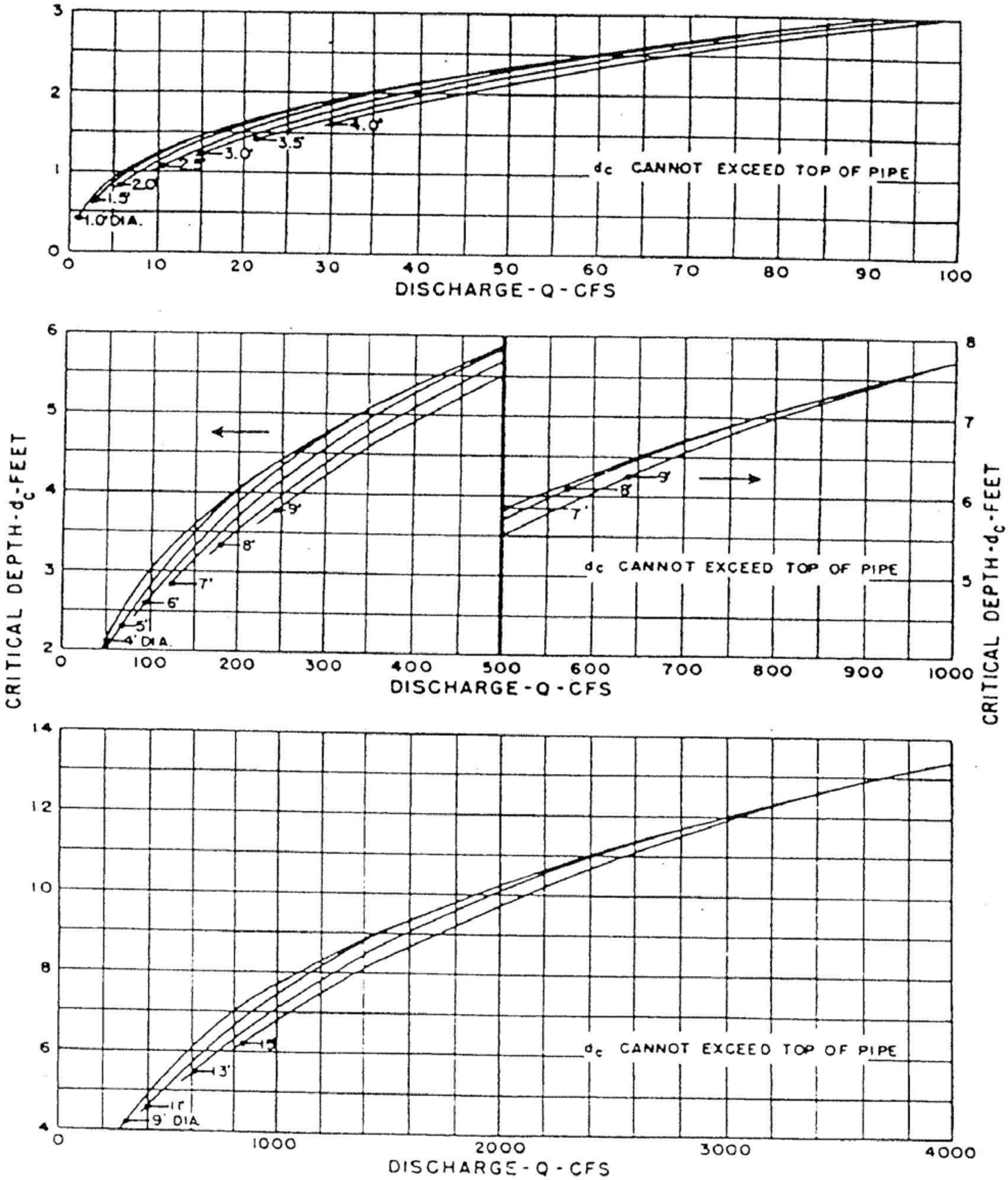
HYDRAIN Culvert Computer Program (HY8). Available from McTrans Software, University of Florida, 512 Weil Hall, Gainesville, Florida 32611.

U.S. Department of Interior. 1983. Design of small canal structures.

APPENDIX A – CRITICAL DEPTH CHARTS



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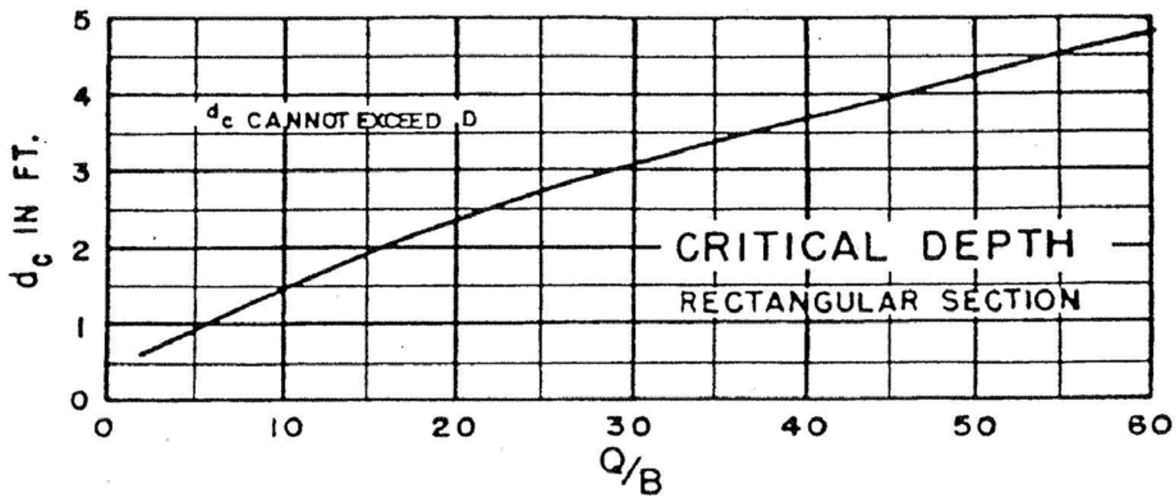
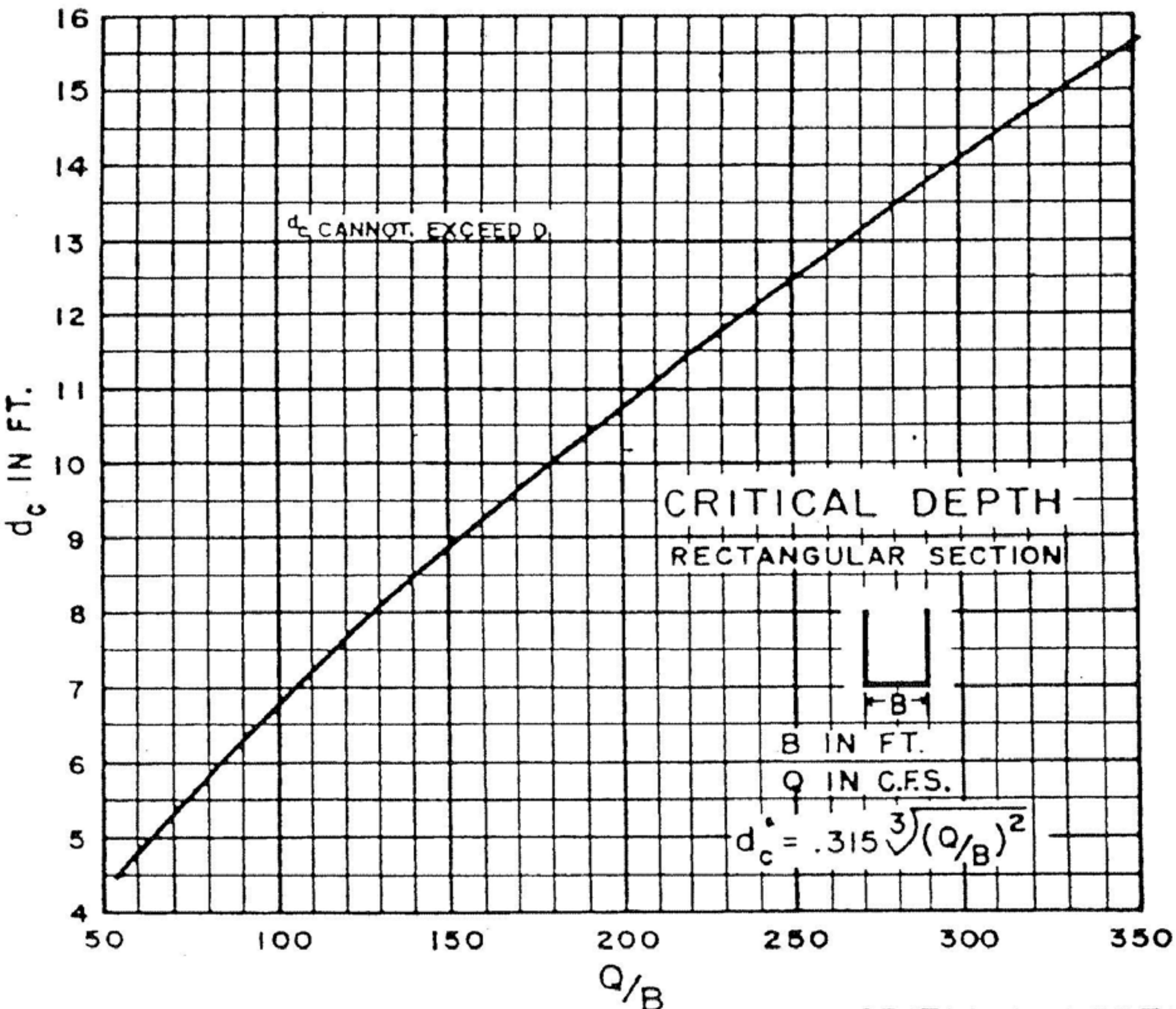
JAN 1964

CRITICAL DEPTH
CIRCULAR PIPE





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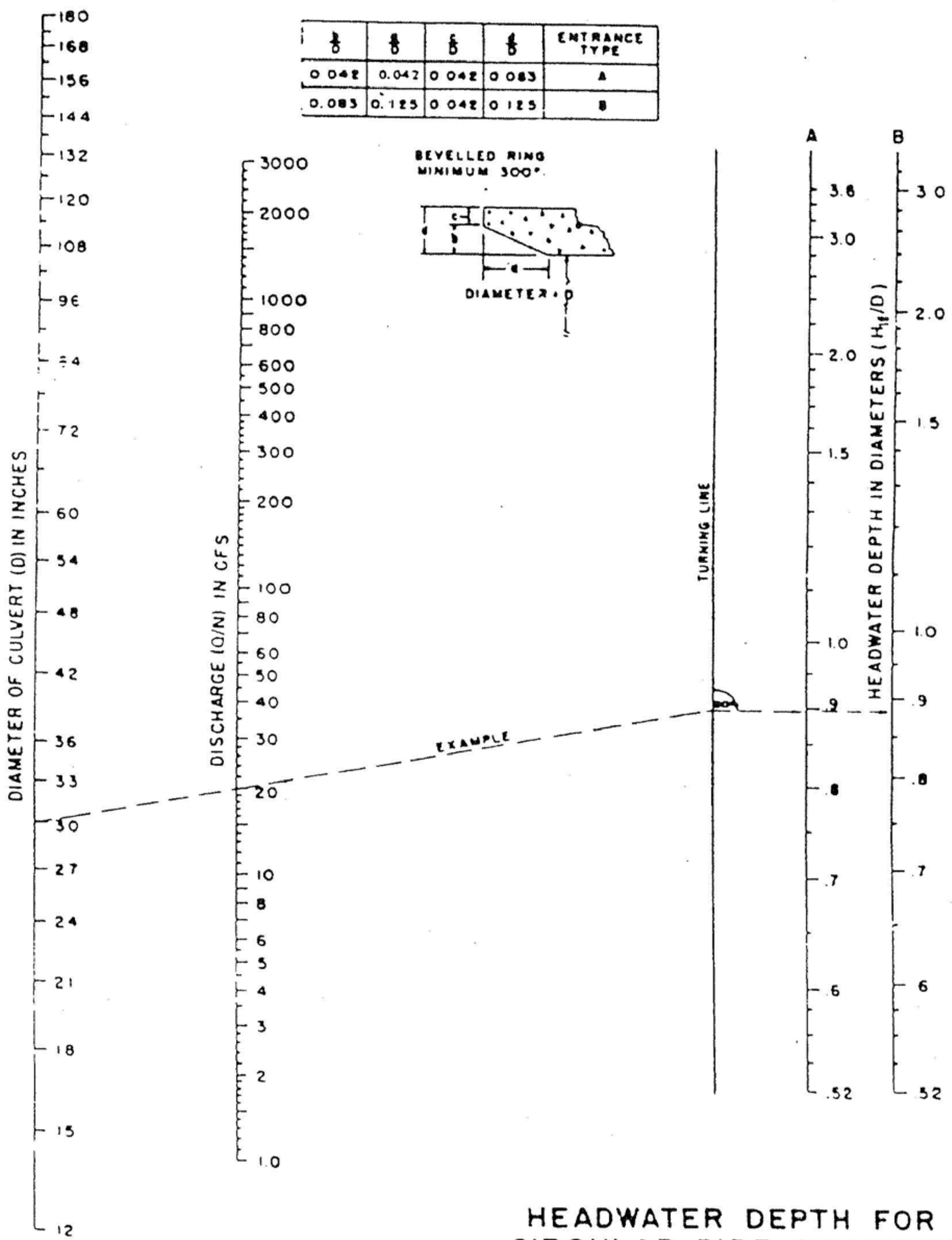
CRITICAL DEPTH
RECTANGULAR SECTION

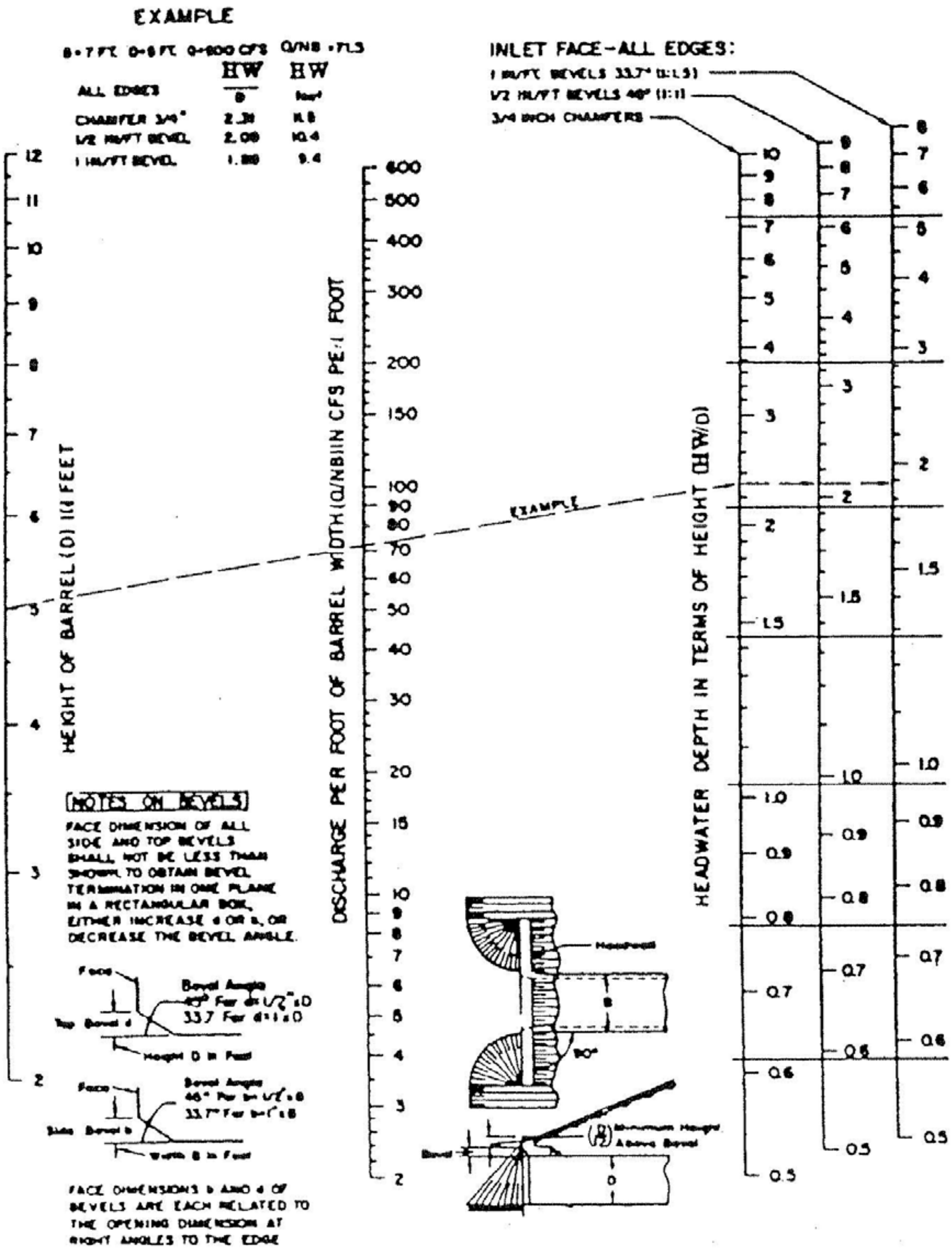
APPENDIX B – IMPROVED INLET FIGURES



a/D	b/D	c/D	d/D	ENTRANCE TYPE
0.042	0.042	0.042	0.083	A
0.083	0.125	0.042	0.125	B

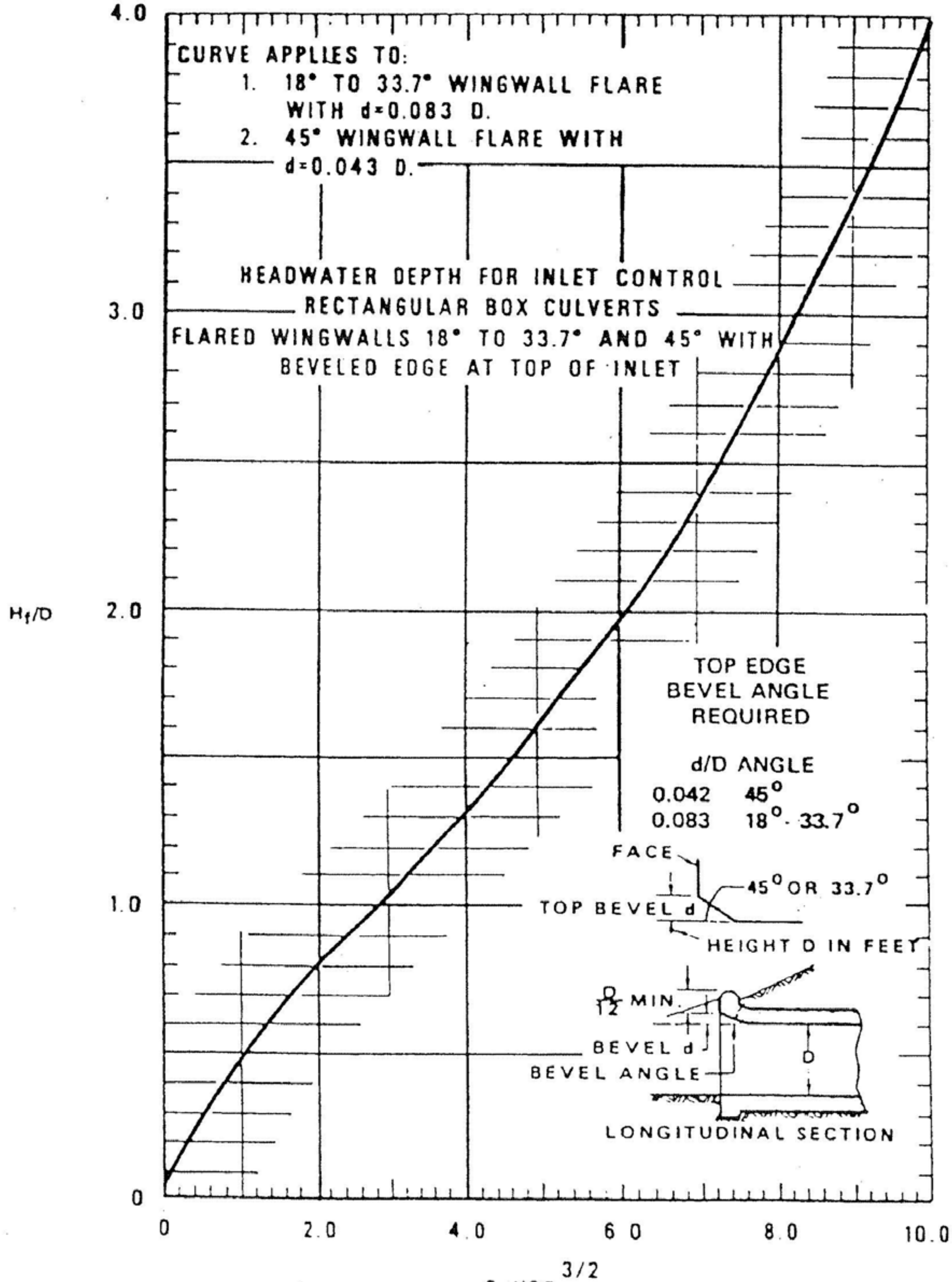
ERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES

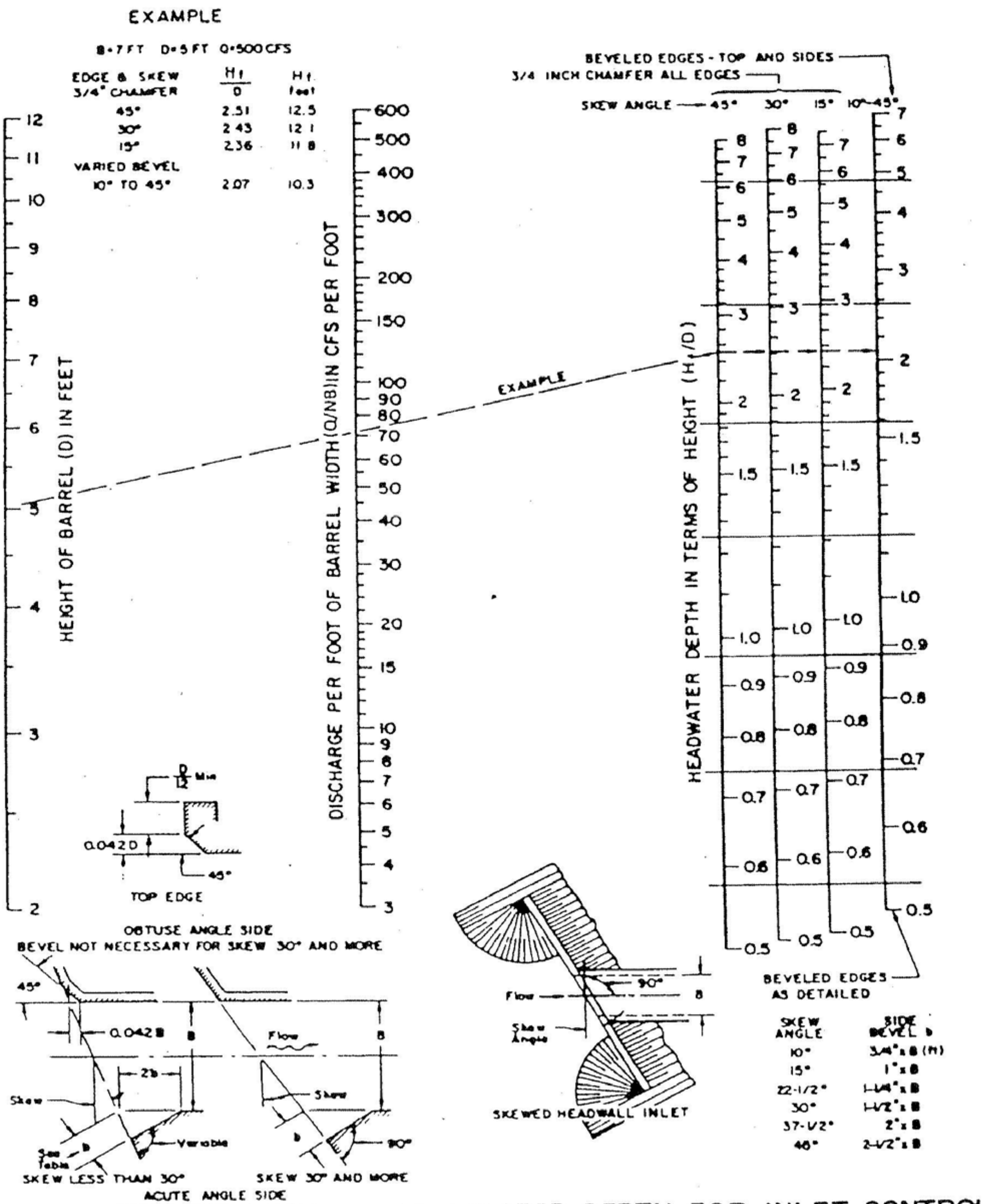
FEDERAL HIGHWAY ADMINISTRATION
MAY 1973



FEDERAL HIGHWAY ADMINISTRATION

MAY 1973

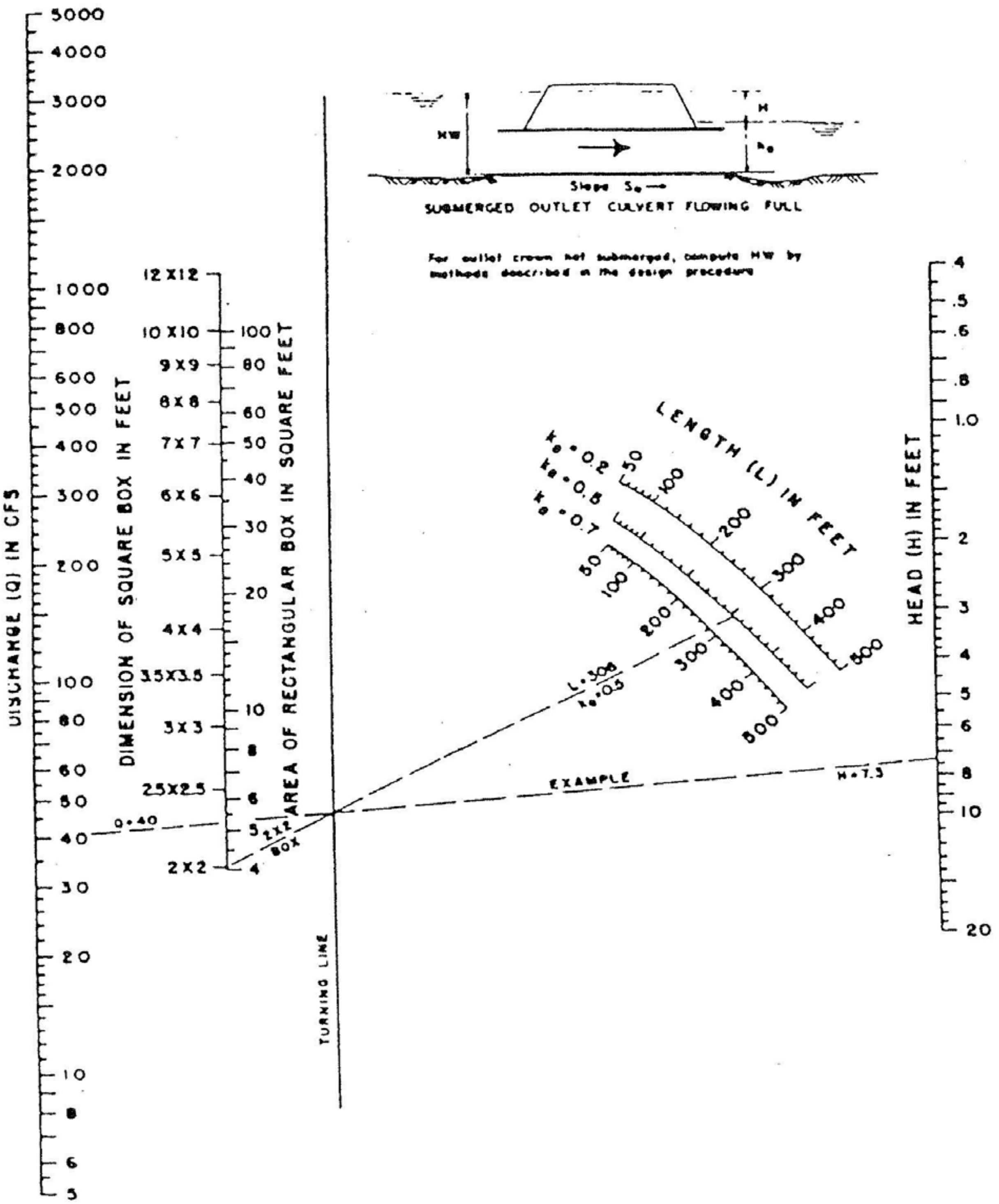
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° AND 45°
WITH BEVELED EDGE AT TOP OF INLET



HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

APPENDIX C – CONVENTIONAL NOMOGRAPHS



HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL n = 0.012

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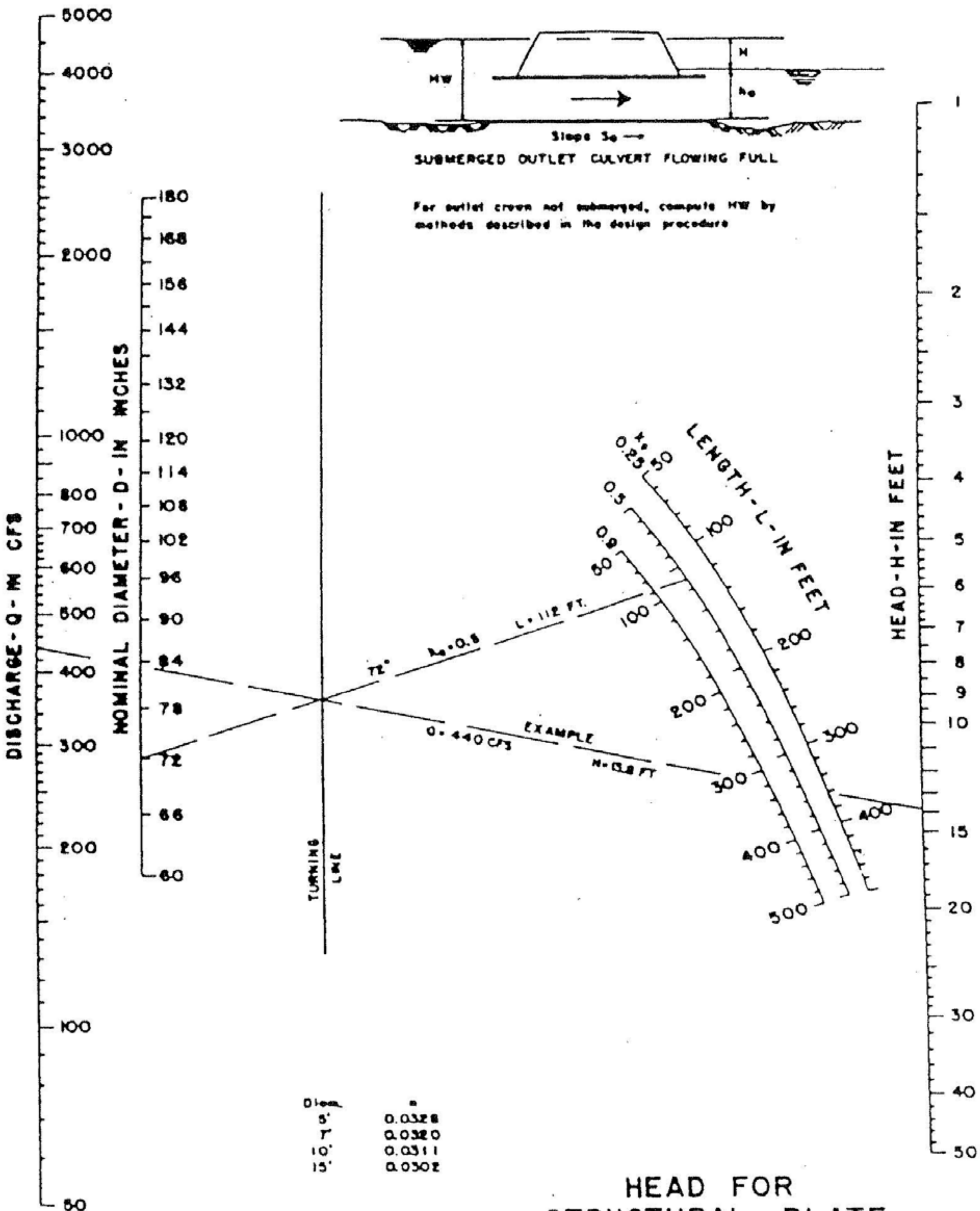

HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL
n = 0.0328 TO 0.0302

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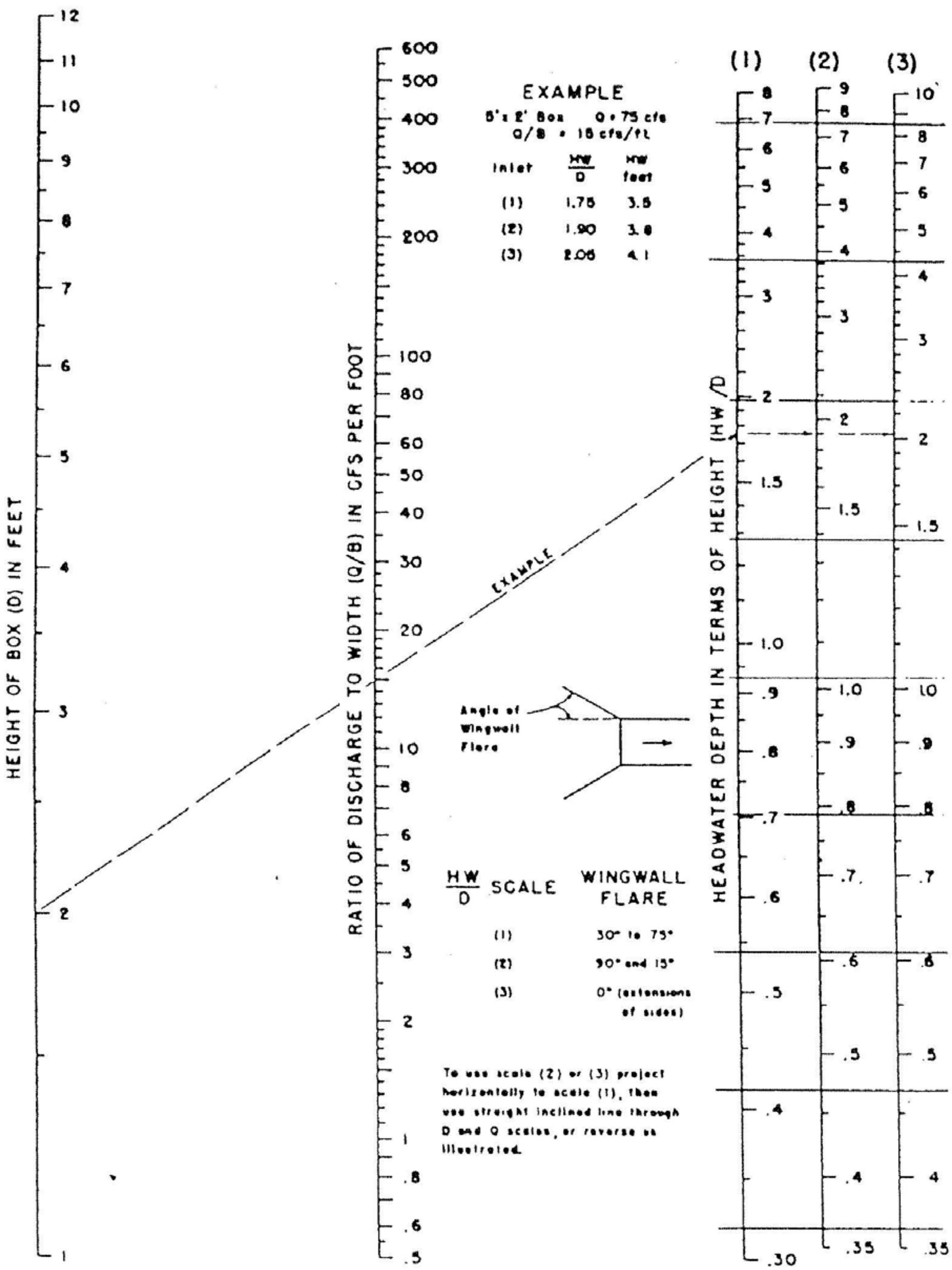
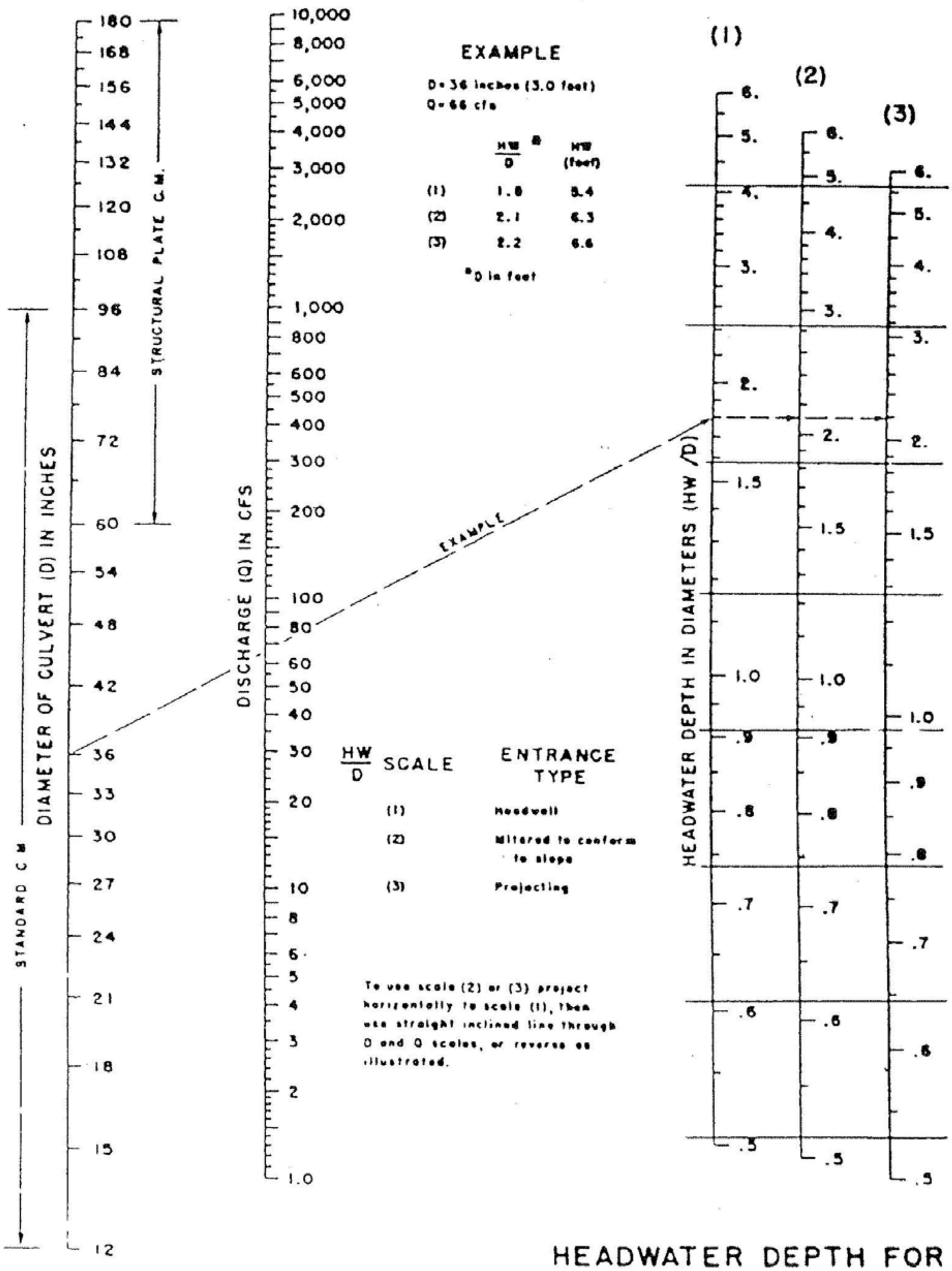


HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL



HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

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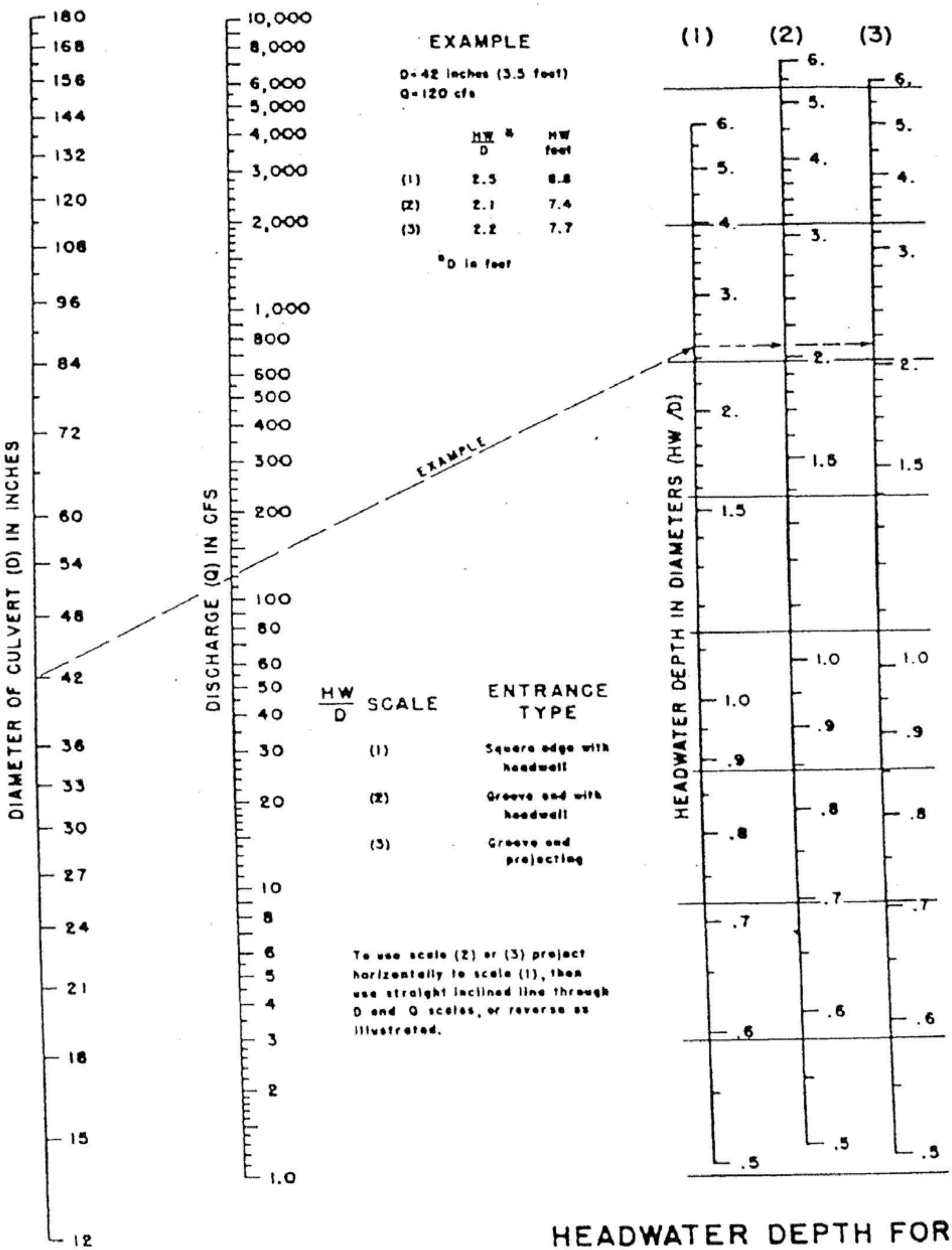


HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

CHAPTER 7

STORAGE AND DETENTION

Chapter Table of Contents

7.1 Overview

Introduction 7.1.1

The traditional design of storm systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where is 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.

RESERVED 7.1.2

Location Considerations 7.1.3

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.

Detention Retention 7.1.4

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For the purpose 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.

Computer Programs 7.1.5

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.

7.2 Symbols and Definitions

Symbol Table

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in storage and detention publications.

Table 7-1

SYMBOLS AND DEFINITIONS

Symbol	*Definition*	*Units*
a	*Constant in stage-storage power equation*	
A	*Cross-sectional area*	ft^2
a_0	*Cross-sectional area of orifice*	ft^2
b	*Constant in the stage-discharge power*	
C	*Weir coefficient or discharge coefficient*	--
dt	*Routing time period*	*min*
D	*Diameter of pipe*	*ft*
g	*Acceleration due to gravity (32.2 ft/s2)*	*ft/s2*
_H	*Head on emergency overflow spillway*	*ft*
H	*Head water depth*	*ft*
Hc	*Height of weir crest above channel bottom*	*ft*
Hs	*Stage*	*ft*
Hv	*Head on Vortex of V-Notch Weirs*	*ft*
I	*Inflow Rate*	*cfs*
K	*Expression combining stage-storage and stage-discharge parameters*	
Kw	*Weir discharge coefficient*	
L	*Horizontal weir length*	*ft*
m	*Coefficient in stage-storage power equation*	
n	*Coefficient in the stage-discharge power equation*	
Nr	*Routing number*	
Q, O	*Discharge*	*cfs*
Qf	*Free flow*	*cfs*
Qj	*Peak inflow rate*	*cfs*
Qo	*Peak outflow rate*	*cfs*
Qs	*Submerged flow*	*cts*
R	*Attenuation Ratio*	
S	*Storage*	*ac-ft*
tb	*Time base of inflow hydrograph*	*hrs*
Tb	*Time base of routed hydrograph*	*min*
Tj	*Duration of basin inflow*	*min*
t0	*Time to peak of routed hydrograph*	*min*
tp	*Time to peak of inflow hydrograph*	*hrs*
Vr	*Runoff volume*	*in*
Vs	*Storage volume*	*in*
θ	*Angle of v-notch weir*	*degrees*

7.3 Design Criteria

General Criteria 7.3.1

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, 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).Storage basins in parking areas shall be drained within 24 hours.

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.

Release Rate 7.3.2

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 the 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.

Storage 7.3.3

Storage volume shall be adequate to attenuate the post-development peak discharge rate 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 volume shall be drained within 72 hours. 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.

Grading And depth of Earthen Storage Facility 7.3.4

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 0.5 percent toward the facilities to allow drainage and to prevent standing water.

Grading and Depth of Earthen Storage Facility (Cont.)

Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. This bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 0.5 percent bottom slope is recommended. A concrete paved low flow or pilot channel 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 for impoundment depths of less than 15 feet. Impoundment depths greater than 15 feet are in addition subject to the requirements of the Safe Dam Act (see Section 7.3.6).

Outlet Works 7.3.5

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.

Safe Dams Act 7.3.6

Under the Dam Safety Act regulations, a dam is an artificial barrier than does or may impound water and that is 15 feet or greater in height and has a maximum storage volume of 10 acre-feet or more. A number of exemptions are allowed from the Safe Dams Act and any questions concerning a specific design or application should be addressed to the North Carolina Department of Environment, Health, and Natural Resources, Dam Safety Section of Land Resources (919-733-4574).

Downstream Impacts 7.3.7

Downstream impacts shall be evaluated for potential flood damages when proposed development will increase discharges for runoff frequencies between 2 year and 100 year. Downstream impacts due to existing conditions and proposed development shall be identified by establishing flood elevations for each condition. This requirement may be relaxed by the Town Engineer where it can be shown that development will not cause adverse downstream impacts.

7.4 General Procedure

Data Needs 7.4.1

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 7-1 below for an example).*
- *Stage-discharge curve for all outlet control structures (see figure 7-2 below for an example).*

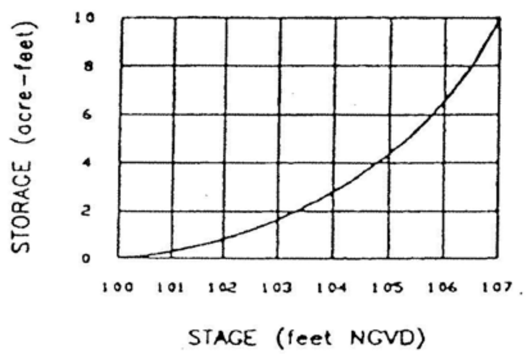


Figure 7-1 Example Stage-Storage Curve

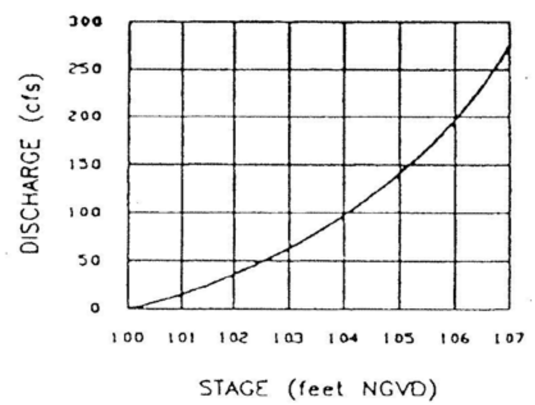


Figure 7-2 Example Stage-Discharge Curve

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.

Procedure 7.4.2

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 Section 7.6). 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 from the 2 and 10-year design storms exceed the pre-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 Dam Safety Act, whichever is more restrictive.*

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

7.5 Outlet Hydraulics

Outlets 7.5.1

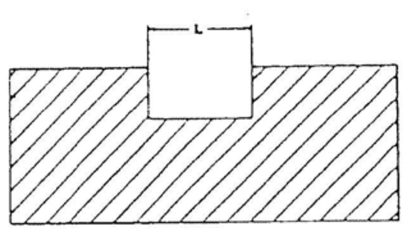
Sharp-crested weir flow equation for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data.

Sharp-Crested Weirs 7.5.2

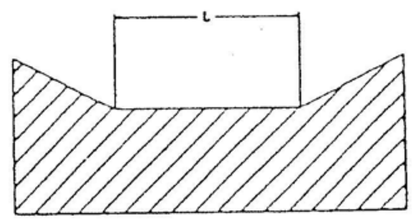
A sharp-crested weir with no end contractions is illustrated below. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [3.27 + 0.4(H/H_c)]\ LH^{1.5}} \qquad \mathbf{(7.1)}$$

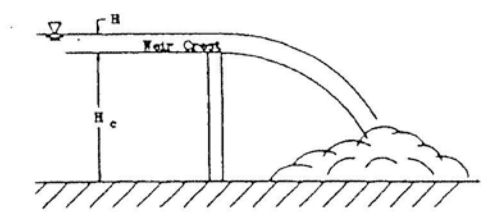
Where: Q = *discharge (cfs)*
H = *head above weir crest excluding velocity head (ft)*
H_c = *height of weir crest above channel bottom (ft)*
L = *Horizontal weir length (ft)*



Sharp–Crested Weir, Two End Contractions



Sharp–Crested Weir, No End Contractions



Sharp–Crested Weir and Head

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

$$\mathbf{Q = [3.27 + 0.4(H/H_c)]\ (L - 0.2H)\ H^{1.5}} \qquad \mathbf{(7.2)}$$

Where: Variables are the same as equation 7.1.

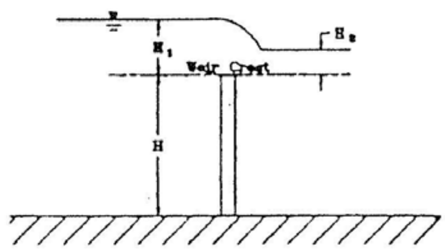
Sharp-Crested Weirs (continued)

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

$$\mathbf{Q_s = Q_f\,(1\text{-}(H_2/H_1)^{1.5})^{0.385}} \qquad \mathbf{(7.3)}$$

Where: Q_s *= submerged flow (cfs)*
Q_f *= free flow (cfs)*
H_1 *= upstream head above crest (ft)*
H_2 *= downstream head above crest (ft)*

Note: H_1 *and* H_2 *should be located far enough upstream and downstream to ensure that normal depth flow has been established.*



Sharp–Crested Weir and Head

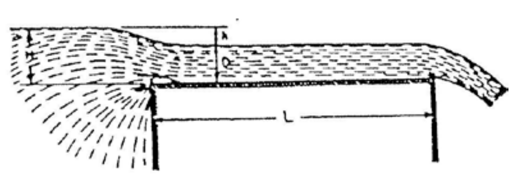
Broad-Crested Weirs 7.5.3

The equation generally used for the broad-crested weir (see sketch below) is (Brater and King, 1976):

$$\mathbf{Q = CLH^{1.5}} \qquad \mathbf{(7.4)}$$

Where: Q = discharge (cfs)
C = broad-crested weir coefficient
L = broad-crested weir breadth (ft)
H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is a 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.



Broad-crested weir.

V-Notch
7.5.4

The discharge through a v-notch weir can be calculated from the Weirs following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2) H_v^{2.5} \quad (7.5)$$

Where: Q = *discharge (cfs)*
θ = *angle of v-notch (degrees)*
H_v = *head on vortex of notch (ft)*

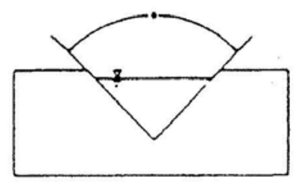


Table 7-2

Broad-Crested Weir Coefficient C Values As A Function of Weir Crest Breadth and Head

Measured Head, H* (ft)	Weir Crest Breadth, L (ft)										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

** Measured at least 2.5H upstream of the weir.*

Reference: Barter and King (1976).

Proportional Weirs 7.5.5

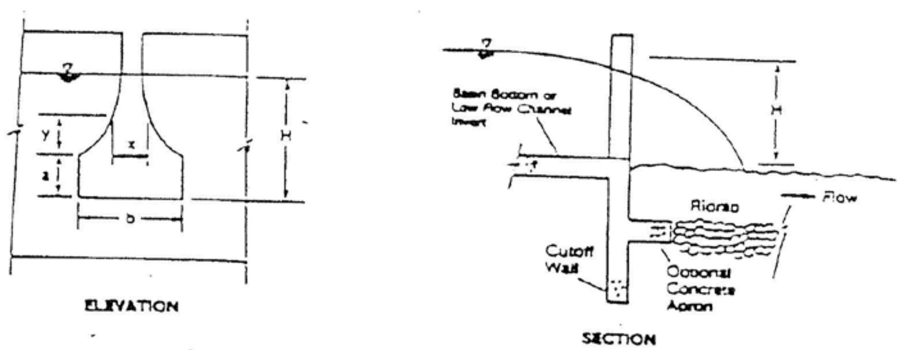
Although more complex to design and construct, a proportional weir may reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are (Sandvik, 1985):

$$\mathbf{Q = 4.97\ a^{0.5}\ b(H - a/3)} \qquad \textbf{(7.6)}$$

$$\mathbf{x/b = 1 - [(1/3.27)\ (arctan(y/a)^{0.5})]} \qquad \textbf{(7.7)}$$

Where: Q = discharge
Arctan (y/a) is in radians
Dimensions a, b, H, x, and y are shown below
(Note: Given dimensions a and b, a series of values for x can be determined from a series of values for y to determine the dimensions of the proportional weir.)



Orifices 7.5.6

The general equation for discharge through a submerged orifice is:

$$\mathbf{Q = CA(2gH)^{0.5}} \qquad \textbf{(7.6)}$$

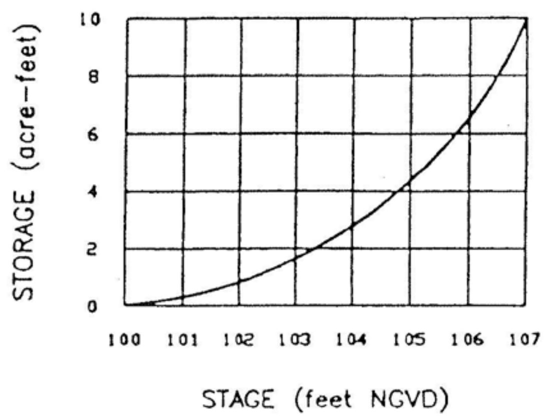
Where: Q = discharge (cfs)
A = cross-section area of smallest section (ft2)
g = acceleration due to gravity, 32.2 ft/s2
D = diameter of pipe (ft)
H = head on pipe, or the vertical distance from the center of the orifice to the upstream free-water surface
C = discharge coefficient (usually equal to 0.61-0.65, although, for relatively large orifices it may be as large as 0.8)

An orifice smaller than 4 inches in diameter must be protected to prevent blockage.

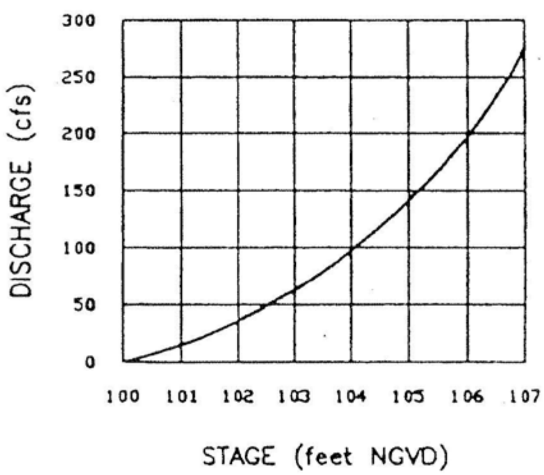
7.6 Routing Calculations

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. Example stage-storage and state-discharge curves are shown below.*



Example Stage-Storage Curve



Example Stage-Discharge Curve

Select a routing time period, dt, to provide at least three points on the rising limb of the inflow hydrograph.

Routing Calculations (continued)

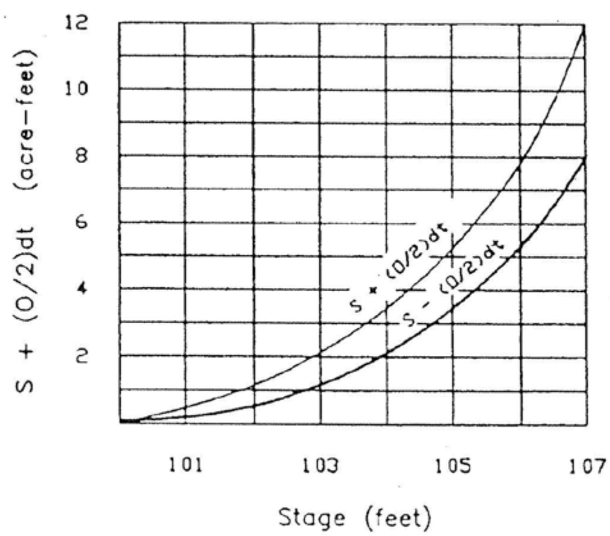
3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of S ± (O/2)dt versus stage. An example tabulation of storage characteristics curve data is shown below.

(1) Stage (H_s) (ft)	(2) Storage[1] (S) (ac-ft)	(3) Discharge[2] (O) (cfs)	(4) (ac-ft/hr)	(5) S - (O/2)dt (ac-ft)	(6) S + (O/2)dt (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: dt = 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 – (O1/2)dt can be determined from the appropriate storage characteristics curve (example given below).



Routing Calculations (continued)

5. Determine the value of S_2 + (O_2/2)dt from the following equation:

$$S_2 + (O_2/2)dt = [S_1 + (O_1/2dt] + [\{I_1 + I_2/2\}dt] \qquad \textbf{(7.7)}$$

Where: S_2 = storage volume at time 2 (ft3)
O_2 = outflow rate at time 2 (cfs)
dt = routing time period (sec)
S_1= storage volume at time 1 (ft3)
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 S2 + (O2/2)dt determined in Step 5 and read off a new depth of water, Hs2.

7. Determine the value of O2, which corresponds to a stage of Hs2 determined in Step 6, using the stage-discharged curve.

8. Repeat Steps 1 through 7 by setting new values of I1, O1, S1, and Hs1 equal to the previous I2, O2, S2, and Hs2 and using a new I2 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 deisgn 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)

7.7 Chainsaw Routing

Introduction 7.7.1

The chainsaw routing procedure is a short-cut method of routing runoff hydrographs through storage facilities to approximate the outflow hydrograph. The same information is required for the chainsaw routing procedure and the storage indication method, as follows:

- *Inflow hydrograph for all design storms*
- *Stage-storage curve for the proposed facility*
- *Stage-discharge curve for all outlet control structures*

The actually routing procedure has been simplified to allow the computations to be performed by hand or in a standard spreadsheet program. Details of the methodology can be found in the publication Elements of Urban Storm Water Design. This method can be combined with the hydrograph generation technique described in Section 3.11 to allow a complete hydrologic design to be performed using a spreadsheet program.

Limitation 7.7.2

The following limitations of the chainsaw routing procedure must be considered.

1. *The computations interval must be approximately equal to 10 percent of the time to peak of the inflow hydrograph.*
2. *The watershed size contributing to the detention facility must be less than or equal to 50 acres.*

Inflow Hydrograph 7.7.2

The inflow hydrograph may be estimated using any of the methods described in Chapter 3, Hydrology. The SCS Unit Hydrograph, SCS Simplified Hydrograph and the SCS Step Function Unit Hydrograph are methodologies which generate an acceptable inflow hydrograph. The step function hydrograph formulation lends itself to easy application in a spreadsheet, and is the method used by the author of the chainsaw routing procedure. The limitations of each hydrograph generation method must be considered when selecting the appropriate method to use.

Stage-Storage 7.7.4

The stage-storage function represents the relation of accumulated storage volume to elevation within the basin. This relation can be expressed as a graph or as a function. However, writing the relation as a function is most beneficial for the chainsaw routing method and is a requirement for other routing techniques (described in Section 7.8). The source of data for the stage-storage function is typically a site plan or topographic map which illustrates the contours of the area proposed to be used for detention storage.

Stage-Storage (continued)

One method for writing a stage-storage function for a typical basin shape is accomplished using the following steps. An assumption is made that the natural logarithm of the stage and storage is a linear relation. In addition, the average-end method for volume computations is used for this application. The steps outlined in the following procedure will result in the storage function having the form:

$$\mathbf{S = K_s H_s b} \quad \textbf{(7.8)}$$

The steps for writing a stage-storage function are:

1. *Determine the elevations of interest within the storage volume of the detention basin and list them in increasing order in column 1 of a spreadsheet or table (like the one shown on page 7-18). The units should be in feet.*

2. *Planimeter or measure the contour elevations for all stages within the detention basin storage volume and enter the areas in column 2. The units should be in square feet.*

3. *Compute the incremental storage volume by using the average-end area method (upper contour area plus the lower contour area divided by two and multiplied by the difference in elevation) and enter in column 3.*

4. *Compute the accumulated volume by adding each incremental volume calculated in step 3 and enter in column 4. These four steps are the standard method of developing a stage-storage relation for a detention facility.*

5. *Compute the relative stage by setting the bottom elevation of the pond equal to zero and adjusting all subsequent elevations appropriately and enter in column 5.*

6. *Calculate S_j, the natural logarithm of the accumulated volume in Column 4 and enter in column 6.*

7. *Calculate H_{si}, the natural logarithm of the stage listed in Column 5 and enter in column 7.*

8. *Calculate b, the exponent in equation 7.8, by selecting two representative points on the relation list in Columns 6 and 7, preferably points at the extreme of the range of values. However, these selected points must be within the representative portion of the stage-storage function. The exponent, b, is then computed as:*

$$\mathbf{b = [\ln (S_2 / S_1)] / [\ln (H_2 / H_1)]} \quad \textbf{(7.9)}$$

9. *Compute the variable K_s from equation 7.10.*

$$\mathbf{K_s = S_2 / H_{S2}^{\,b}} \quad \textbf{(7.10)}$$

10. *Check the resulting equation with several of the other pairs of storage and elevation points to determine the accuracy of the resulting equation. Modify the equation as necessary by selection of new stage and storage values and repeat Steps 8-10 if needed to get a better fit for the stage-storage curve.*

Stage-Discharge 7.7.5

Stage-discharge relations should be computed using the equations listed in Section 7.5, Outlet Hydraulics. Care must be taken to include all possible control sections of the outlet structure in the computations so that the resulting stage-discharge function is accurate.

Procedure 7.7.6

The chainsaw routing method assumes that the change in storage over the computation time increment can be estimated as a parallelogram instead of a trapezoid as used in the storage-indication method. The following procedure should be used:

1. *Create column 1 of a spreadsheet or table (like the one shown on page 7-19) which consists of each incremental computational time interval which must match the computational interval of the inflow hydrograph. The time increment should be set approximately equal to 0.10 times the time to peak of the inflow hydrograph. The time increment should be converted to seconds to be consistent with the units of other computations.*

2. *Create column 2 which consists of the entire inflow hydrograph for each time increment. The inflow hydrograph is generated using any of the appropriate methods described in Chapter 3 of this document.*

3. *Compute column 3 as the change in storage. The value for the cell of row j is the inflow at row i minus outflow of row i multiplied by the time increment.*

4. *Compute column 4 as the updated stage based on the new storage volume by interpolating the stage-storage relation or by computing the stage based on the derived stage-storage function (as described in Section 7.7.4).*

5. *Compute column 5 as the outflow for row j based on the stage-discharge function and the revised stage listed in Column 4. A multiple stage outlet function can be modeled with additional columns to represent each control section of the spillway configuration.*

In order to begin the computations, the spreadsheet or table must be initialized by performing the following three steps:

1. *Set the initial inflow and outflow equal to zero.*

2. *Set the initial stage equal to the flow invert elevation of the outlet spillway out outlet pipe.*

3. *Set the initial storage volume to zero.*

In some cases (when the stage-discharge relation has high outflows relative to the stage-storage relation with low storage values) this method will result in unstable results. This becomes evident when the outflow exceeds the inflow, perhaps to the degree that negative storage will be computed. This error can be corrected by re-initializing the row at which this instability occurs. with the following three steps:

1. *Set outflow (Column 5) equal to inflow (Column 2).*

Procedure (continued)

2. *Set stage (Column 4) equal to outflow (Column 5) using the stage-discharge function.*

3. *Set storage (Column 3) equal to state (Column 4) using the stage-storage function.*

Example 7.7.7

Route the 10-year, 6-hour storm generated by the HEC-1 model in the example in Section 3.11.7 through a detention basin with a 3-inch orifice at elevation 667.00, 10-foot weir at elevation 670.00, and a storage volume of 0.37 acre-feet at elevation 671.00.

Develop a stage-storage function based on the computed contours of the facility as follows:

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6	Column 7
Elevation (feet)	Contour Area (sq.ft.)	Incremental Volume (cu.ft.)	Accumulated Volume (cu.ft.)	Stage (feet)	lnS	lnZ
667	0		0	0		
668	2,270	1,135	1,135	1	7.0344	0.000
669	3,820	3,045	4,180	2	8.3381	0.6931
670	6,210	5,015	9,195	3	9.1264	1.0986
671	8,600	7,405	16,600	4	9.7172	1.3863

1. *Input the values of storage and stage from elevations 668 feet and 671 feet into equation (7.9).*

   b = ln (16,600 / 1,135) / ln (4 / 1) = 1.94

2. *Determine Ks from equation 7.10.*

   Ks = 16,600 / 4 1.94 = 1,127.5

3. *The stage-storage function may be written as:*

   S = 1,127 Hs 1.94

4. *Check to determine if the function approximates the storage volume correctly by inputting the stage of 3 feet.*

   S = 1,127.5 (3) 1.94 = 9,500 cubic feet (approximately equal to 9,195 cubic feet as shown in the above table).

Example Problem (continued)

The second step of the process is to develop a stage-discharge function for the outlet structures. The orifice and weir equations are used as follows:

1. *For elevations from 667 feet to 670 feet, the discharge is (weir flow has been neglected for low stages in this computation):*

$$Q = C\,A\,(2\,g\,H)^{0.5}$$

$$Q = (0.6)\,(\pi)\,(1.5 / 12)^2\,[(2)\,(32.2)\,(H)]^{0.5}$$

$$Q = 0.24\,H^{0.5}$$

2. *For elevations 670 feet to 671 feet, the emergency spillway discharge function will be added to the orifice outflow as follows:*

$$Q = C\,L\,H^{1.5} = (3.0)\,(10)\,(H)^{1.5} = 30\,H^{1.5}$$

The final step for completing the chainsaw routing procedure is to perform the actual routing. The following table lists the results. The first 122 minutes of the hydrograph have been eliminated for clarity. The values enclosed with parentheses are lines of the spreadsheet which were re-initialized due to stability problems of the computations.

Example Chainsaw Routing Computations

Column 1 Time (min.)	Column 2 Inflow (cfs.)	Column 3 Storage (cu. ft.)	Column 4 Stage (ft.)	Column 5 Outflow (cfs.)
0	0.000	0.0	0.00	0.000
2	0.001	0.0	0.00	0.000
4	0.006	0.1 (0.0)	0.01 (0.00)	0.024 (0.006)
6	0.016	0.0	0.0	0.000
8	0.029	1.9 (0.1)	0.04 (0.01)	0.048 (0.029)
10	0.043	0.1 (1.3)	0.04 (0.03)	0.048 (0.043)
12	0.059	1.3	0.03	0.042
14	0.075	3.3	0.05	0.054
16	0.093	5.8	0.07	0.063
18	0.112	9.4	0.08	0.068
20	0.133	14.7	0.11	0.080
22	0.155	21.1	0.13	0.087
24	0.181	29.3	0.15	0.043
26	0.221	39.9	0.18	0.102
28	0.283	54.2	0.21	0.110
30	0.351	75.0	0.25	0.120

Column 1	Column 2	Column 3	Column 4	Column 5
Time (min.)	Inflow (cfs.)	Storage (cu. ft.)	Stage (ft.)	Outflow (cfs.)
32	0.415	102.7	0.29	0.129
34	0.477	137.0	0.34	0.140
36	0.549	177.4	0.39	0.150
38	0.632	225.3	0.44	0.159
40	0.717	282.1	0.49	0.168
42	0.929	348.0	0.55	0.178
44	1.212	438.1	0.61	0.187
46	1.445	561.1	0.70	0.201
48	1.729	710.4	0.79	0.213
50	2.210	892.3	0.89	0.226
52	2.910	1,130.4	1.00	0.240
54	4.456	1,450.8	1.14	0.256
56	6.306	1,954.8	1.33	0.277
58	7.643	2,678.3	1.56	0.300
60	7.612	3,559.5	1.81	0.323
62	6.558	4,434.2	2.03	0.342
64	5.674	5,180.1	2.19	0.355
66	4.508	5,818.4	2.33	0.366
68	3.675	6,315.4	2.43	0.374
70	3.250	6,711.5	2.51	0.380
72	2.745	7,055.9	2.57	0.385
74	2.258	7,339.1	2.63	0.389
76	2.015	7,563.4	2.67	0.392
78	1.849	7,758.2	2.70	0.394
80	1.726	7,932.8	2.73	0.397
82	1.644	8,092.3	2.76	0.399
84	1.564	8,241.7	2.79	0.401
86	1.438	8,381.3	2.81	0.402
88	1.263	8,505.6	2.83	0.404
90	1.115	8,608.7	2.85	0.405
92	1.032	8,693.9	2.87	0.407
94	0.984	8,768.9	2.88	0.407
96	0.942	8,338.1	2.89	0.408

Column 1	Column 2	Column 3	Column 4	Column 5
Time (min.)	Inflow (cfs.)	Storage (cu. ft.)	Stage (ft.)	Outflow (cfs.)
98	0.905	8,902.2	2.90	0.409
100	0.879	8,961.7	2.91	0.409
102	0.850	9,018.1	2.92	0.410
104	0.822	9,070.9	2.93	0.411
106	0.802	9,120.2	2.94	0.412
108	0.780	9,167.0	2.95	0.412
110	0.758	9,211.2	2.95	0.412
112	0.741	9,252.7	2.96	0.413
114	0.722	9,292.1	2.97	0.414
116	0.686	9,329.1	2.97	0.414
118	0.628	9,361.7	2.98	0.414
120	0.578	9,387.4	2.98	0.414
122	0.550	9,407.1	2.99	0.415
124	0.535	9,423.3	2.99	0.415
126	0.522	9,437.7	2.99	0.415
128	0.509	9,450.5	2.99	0.415
130	0.501	9,461.5	2.99	0.415
132	0.490	9,471.8	3.00	0.416
134	0.479	9,480.7	3.00	0.416
136	0.471	9,488.3	3.00	0.416
138	0.462	9,494.9	3.00	0.416
140	0.454	9,500.4	3.00	0.416
142	0.447	9,505.0	3.00	0.416
144	0.439	9,508.7	3.00	0.416
146	0.423	9,511.5	3.00	0.416
148	0.395	9,512.3	3.00	0.416
150	0.371	9,509.8	3.00	0.416
152	0.359	9,504.4	3.00	0.416
154	0.352	9,497.6	3.00	0.416
156	0.346	9,489.9	3.00	0.416
158	0.339	9,481.5	3.00	0.416
160	0.334	9,472.3	3.00	0.416
162	0.329	9,462.5	2.99	0.415

Column 1	Column 2	Column 3	Column 4	Column 5
Time (min.)	Inflow (cfs.)	Storage (cu. ft.)	Stage (ft.)	Outflow (cfs.)
164	0.325	9,452.2	2.99	0.415
166	0.319	9,441.4	2.99	0.415
168	0.314	9,429.9	2.99	0.415
170	0.311	9,417.8	2.99	0.415
172	0.307	9,405.3	2.98	0.414
174	0.303	9,392.5	2.98	0.414
176	0.299	9,379.2	2.98	0.414
178	0.295	9,365.4	2.98	0.414
180	0.291	9,351.1	2.98	0.414
182	0.287	9,336.3	2.97	0.413
184	0.285	9,321.2	2.97	0.413
186	0.281	9,305.8	2.97	0.413

7.8 Graphical Method for Small Detention Ponds

Introduction 7.8.1

There are many approximate methods for estimating the volume of storage required for detention design including graphical estimation procedures and a number of variations involving the Rational Method. Each of these methods attempt to approximate the results which would be generated through a full storage-indication routing described in previous sections without actually performing the routing. The method provided here was developed based on the work of Horn (1987) and graphically approximates the routing of the volume of flow from the Charlotte 6-hour storm, using a hydrograph shape equivalent to the SCS dimensionless unit hydrograph and approximated by either the step function or subsequent equation provided in Section 3.11.

Limitations 7.8.2

This method is approximately equivalent to a standard routing method and can be used with the following limitations:

The method is subject to the limitations of the SCS step function in section 3.11.
The method is limited to watersheds of five (5) acres or less
The method requires the use of a single outlet with a continuous stage-discharge curve which can be expressed as a power function of the form.

$$S = bH^n \qquad (7.11)$$

Where: b = *constant in power equation for either weir or orifice flow as* (LK_w) *for weir flow or* $(a_o\, k_o\, (2g)^{1/2})$ *for orifices.*

n = *coefficient in stage-discharge equation equal to 0.5 for orifice flow and 1.5 for weir flow.*

Outflow must begin when inflow begins (no dead storage capacity) and fits the power equation from the start of outflow
inflow hydrograph approximates the SCS dimensionless unit hydrograph
stage-storage can be expresses as a power function of the form:

$$S = aH^m \qquad (7.12)$$

Where: a = *constant in the stage-storage power equation*
m = *coefficient in the stage-storage power equation*

Basic Overview 7.8.3

Given that the inflow hydrograph can be approximated by a function similar to the step function and that both the stage-storage and stage-discharge functions can be expressed as simple power functions, a series of dimensionless substitutions can be made to the hydrologic continuity equation that is used for storage routing. The results are the following dimensionless parameters:

$$\boldsymbol{\alpha = m/n} \qquad \textbf{(7.13)}$$

$$\boldsymbol{K = a/b_{\alpha}} \qquad \textbf{(7.14)}$$

$$\boldsymbol{N_r = (KQ_j^{\alpha-1})/t_p} \qquad \textbf{(7.15)}$$

$$\boldsymbol{R = Q_o / Q_1} \qquad \textbf{(7.16)}$$

Basic Overview (continued)

N_r is called the “routing number” and R is called the “attenuation ratio”. Figures 7-4 and 7-5 show a relationship between N_r and R for either spillway outlets or orifice outlets for a range of values of α. These figures constitute the “routing” in this method. For a known attenuation ratio, R, the routing number, N_r, can be read directly from the graph. Alternately, for a known routing number, N_r, the attenuation ratio, R, can be read.

For design, one must use a known attenuation ratio target and determine N_r. K can be determined from N_r, t_p, Q_j, and a known stage-storage function. From a known K, b can be found and thus the orifice size or weir length.

For analysis the process works in reverse. N_r is calculated from known physical site information. From either Figure 7-4 or Figure 7-5 the attenuation ratio is read and the outflow peak value is calculated.

Emergency Spillway Approximation 7.8.4

All detention facilities are required to safely pass the 50-year –post-development peak flow through an emergency spillway. If this spillway is to be the only spillway for the detention facility, then the orifice or weir and embankment need to be sized for this flow as well as the 2- and 10-year flow. If an additional overflow weir is to be added to the minor storage facility, an approximate method will normally give adequate results.

The basic procedure is to distribute the additional flow from the 50-year peak inflow between the previously sized orifice or weir and an overflow spillway. No routing is done and no peak attenuation assumed. It is assumed that the crest of the emergency overflow spillway is set at an elevation equal to the stage (H) of the peak 10-year storm outflow.

Using orifice or weir flow for the main outlet, and weir flow for the overflow emergency spillway the combined flow equation is:

$$\mathbf{Q_d = b[(H + \Delta H)^n - (H)^n] + k_{ew} (L_{ew}) (\Delta H^{1.5})} \qquad \mathbf{(7.17)}$$

Where: Q_d *= difference between the 50-year peak post-development inflow and the orifice or weir outflow (Qp) at peak stage (H) (i.e. the flow to be proportioned between the orifice or weir and the emergency overflow spillway)- cfs*

k_{ew} *= emergency overflow spillway discharge coefficient*

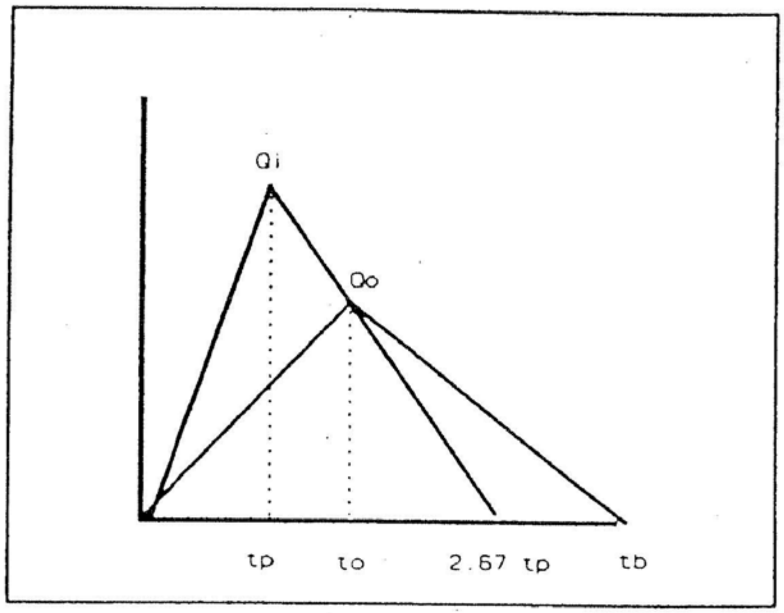
L_{ew} *= emergency overflow spillway effective length – feet*

ΔH *= head on the emergency overflow spillway measured from H upward – feet*

And b and n are the constant and coefficient in the stage-discharge equations for either orifice or weir flow of the principle outlet for the detention pond as described previously.

Time of Peak 7.8.5

Without storage indication routing of an actual hydrograph the distribution of runoff and routed flow volume in time can only be approximated. The approximation is based on the use of the SCS triangular approximation to the curvilinear dimensionless unit hydrograph and a straight line routing graphical approximation. This is illustrated in Figure 7-3 below.



Triangular Approximation

Figure 7-3

The time base of the routed hydrograph starts at the time indicated by the value calculated using the method in Section 3.11.6 and Appendix C. The time base of the routed hydrograph is then calculated by equating the volumes in and out for the triangular approximation and can be expressed as:

$$\mathbf{T_b = 2.67\ (Q_j/Q_o)} \qquad \mathbf{(7.18)}$$

Using triangular approximation, the time of peak outflow of the routed hydrograph, T., from the start of runoff can be found from the following equation:

$$\mathbf{t_o = 1.67\ (t_p)\ (1.6 - Q_o/Q_j)} \qquad \mathbf{(7.19)}$$

Time of Peak (continued)

Because this method does not consider the long tail of both the runoff and the routed hydrograph, it will not yield a time period for runoff and the routed hydrograph as extended as an actual storage-indication or the chainsaw routing method.

Stage-Storage Curves 7.8.6

A stage-storage equation of the detention basin must be written to apply the graphical method of routing. The method for writing the stage-storage function is described in Section 7.7 and should be followed for this method.

Procedure 7.8.7

For design, the following steps should be followed to size a small detention pond. The orifice or weir sizing must be done for both the 2-year and 10-year storm to determine which storm controls orifice or weir size.

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1. Data Input

Calculate or estimate C factors, Curve Numbers, and times of concentration for the pre and post development site conditions. Determine the location and approximate size limits (vertical and horizontal) of the detention pond. Choose the outlet device (orifice or weir) and the discharge coefficient (k_o or k_w).

2. Basic Calculations

Calculate peak flows for the 2-, 10- and 50- year discharges. Calculate the runoff volume from SCS curve number equations. Calculate the time-to-peak for the 2- and 10-year storms as described in Section 3.11.

3. Volume Estimate and Detention Layout

Estimate the 10-year storage volume required from the equation:

$$V_s = 1.39\ (60)\ (t_p)\ (Q_j - Q_o)$$

Estimate a reasonable peak 10-year stage at peak 10-year outflow using the 10-year pre-developed peak flow as the flow target and the weir or orifice equation.

With an estimated volume and stage at peak outflow, develop the detention pond layout and develop the stage-storage power function.

4. Routing Number

Calculate R and α for the 2-year and 10-year storms from equations (7.13) and (7.16). Using figure 7-4 for spillway flow or figure 7-5 for orifice flow, determine the routing number, Nr, for both storm events.

5. Preliminary Weir or Orifice Size and Other Data

Calculate K for the 2- and 10-year storm event routing numbers. From the two K values, calculate b and the preliminary orifice or weir sizes. From the orifice or weir sizes and peak outflow targets, calculate the actual head and storage from the stage discharge and stage-storage equations for the 2- and 10-year storm events. Select the smaller of the outflow sizes and the larger of the storage volumes as the result.

Procedure and (continued)

If the weir or orifice size is to be rounded to a next available size proceed through steps 6

7. If not, go to step 8 to size the emergency spillway.

6. <u>(Optional) Routing Number Recalculation</u>

From a chosen (rounded) weir or orifice size for the 10-year flow, calculate b and K. Calculate a new routing number, N_r, based on this b and K.

7. <u>(Optional) R and Other Data Recalculation</u>

Enter the appropriate figure 7-4 or 7-5 for either spillway or orifice flow with the new routing. number, N_r, and find the attenuation number, R.

From R, calculate the new Q_o and make sure it is less than the target. If so, calculate the new stage and storage for this peak outflow for the 10-year flow. This is the final size and stage data.

8. <u>Emergency Spillway</u>

Using equation (7.17) for emergency spillway sizing, input and assumed ΔH and calculate the emergency overflow spillway length, L_{ew}. Often one foot is used for ΔH, though site constraints and safety considerations will determine the assumed head.

9. <u>Hydrograph Shift in Time</u>

Find the total initial losses for the site from the SCS initial loss equation as shown in section 3.11. Calculate the time base of the routed hydrograph and the time of the peak of the hydrograph. This approximates the routed hydrograph in time and shape distribution.

Example 7.8.8

Sizing of a Small Pond for Orifice Row

Input Data

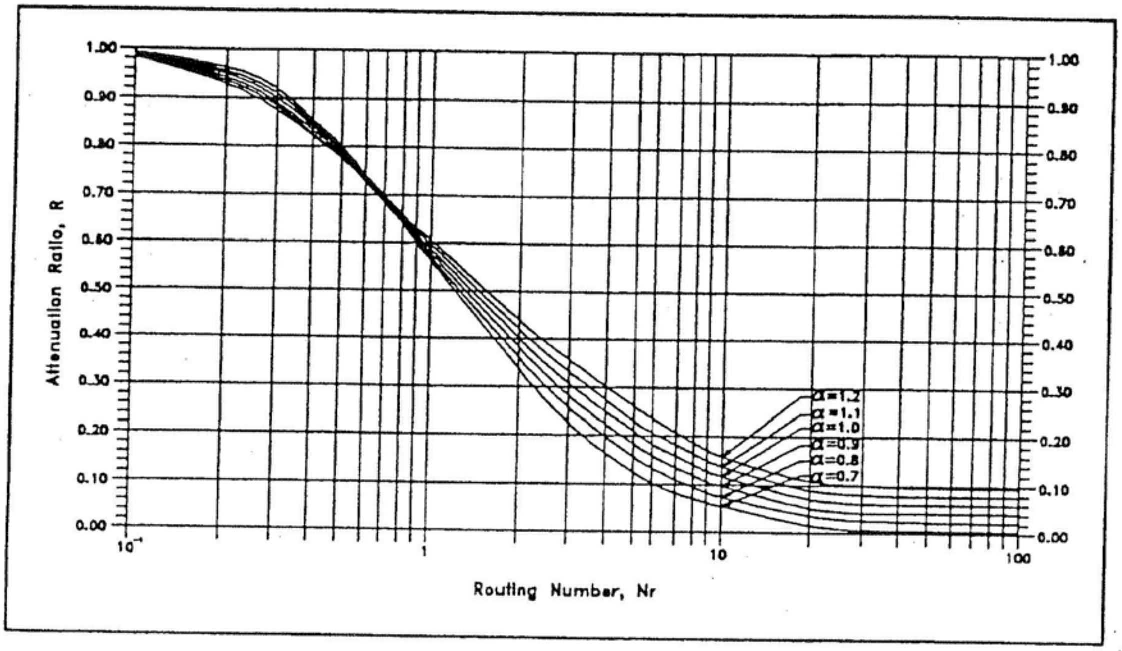
Area = 2 acres	K_o = 0.6
C pre-development = 0.3	C post-development = 0.8
CN pre-development = 70	CN post-development = 92
t_c pre-development = 5 min	t_c post-development = 5 min

Basic Calculations

Rational Method Peak Flow and SCS Method for volume and time to peak:

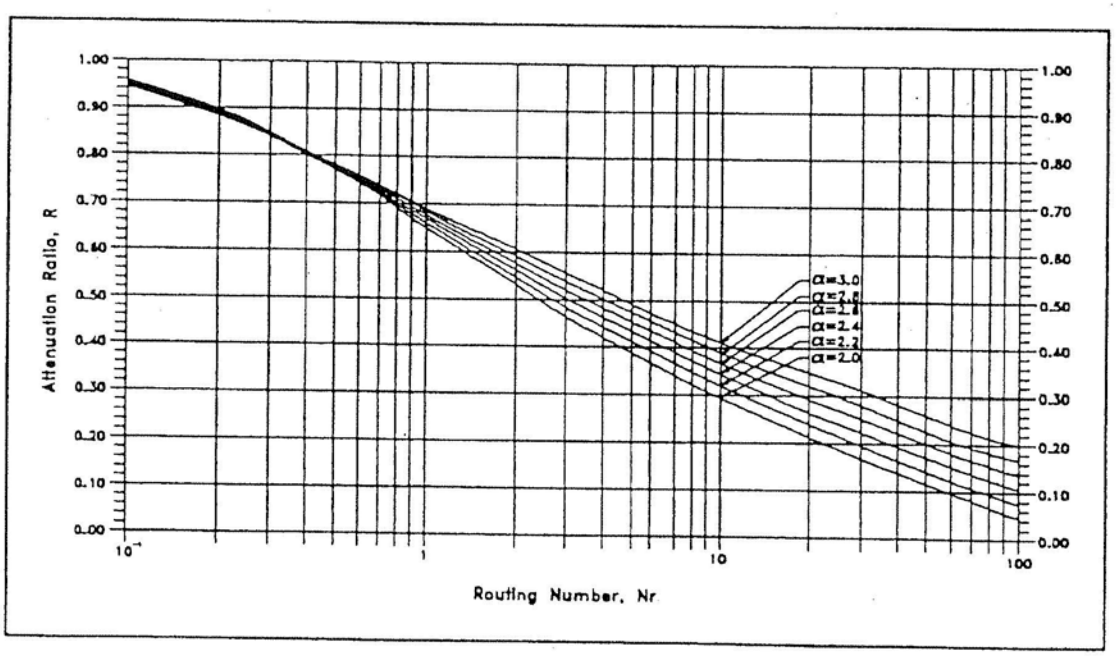
Q_2pre = 3.02 cfs	$Q_v$post = 1.139 in
Q_2post = 8.05 cfs	t_{P2} = 12.88 min
Q_{10}pre = 4.22 cfs	$Q_v$post = 2.800 in
Q_{10}post = 11.25 cfs	t_{P10} = 22.66 min

Q_{60}post = 14.40 cfs



Variation in Attenuation Ratio as a
Function of N, and σ for Reservoirs
With Spillway Outlet

Figure 7-4



Variation in Attenuation Ratio as a
Function of N, and σ for Reservoirs
With Orifice Outlet

Figure 7-5

Example (continued)

Preliminary Estimates

Based on a triangular hydrograph approximation, a first estimate of required volume (using the 10-year storm peaks and volume) is:

$$S = 1.39\ (60)\ (t_p)\ (Q_j - O_o)$$
$$S = 1.39\ (60)\ (22.66)\ (11.24 - 4.22)$$
$$S = 13{,}266 \text{ cu ft}$$

Using 0.6 for an orifice discharge coefficient, several trial values of an orifice are chosen to produce an acceptable head for the target flow of 4.22 cfs.

A diameter of 0.708 feet (8.5 inches) will approximately produce a 5 foot depth for the 10-year peak outflow (4.22 cfs) and about 2.5 foot depth for the 2-year target flow of 3.02 cfs. This fits site characteristics.

$$a_o = 3.14159\ (D^2/4) = 0.394 \text{ sq ft.}$$

$$H = [Q_o/(8.02)\ (k_o)\ (a_o)]^{1/0.5} = 4.95 \text{ ft}$$

Therefore the basin should produce a storage volume of approximately 13,000 cubic feet at a depth of about 5 feet. Using a spreadsheet, a basin was chosen with bottom width = 25 feet, length = 55 ft, and side-slopes 2.51. A curve fit to this data yielded values for the stage-storage equation of a = 1535 and m = 1.35.

2-Year Flow Calculations

The following calculations size the 2-year orifice:

$$\alpha = m/n = 1.35/0.5 = 2.7$$
$$R = Q_o/Q_j = 3.02/8.04 = 0.376$$
$$N = 11 \text{ (from Figure 7-5)}$$

Then:

$$K = Nr\ (t_p)\ Q_j^{\alpha-1}$$
$$K = 11\ (12.88)\ (60)\ /\ (8.04^{2.7-1})$$
$$K = 246$$

$$b = (a/K)^{n/m}$$
$$b = (1535/246)^{0.37}$$
$$b = 1.97$$

$$a_o = b/[(8.02)\ (k_o)]$$
$$a_o = 1.97/(8.02)\ (0.6)$$
$$a_o = 0.41 \text{ sq ft}$$

$$D = [(4/3.14159)\ a_o]^{1/2}$$
$$D = 0.72 \text{ feet} = 8.66 \text{ inches}$$

Example (continued)

In this case it is desirable to use a standard available size of 8.5 inches. To check, proceed backward as follows to get a new N_r.

$a_o = 0.39$ *sq ft*
$b = 1.90$
$K = 271$
$N_r = 12.33$

From Figure 7-5, R = 0.365

$Q = 2.94$ *cfs (Target 3.02 cfs) (OK)*
$H = 2.39$ *ft (peak stage)*
$S = 5006$ *cu ft (peak storage)*

<u>10-year Rood Calculations</u>

For the 10-year flood, the orifice and site information stays the same, only the peak flows and time-to-peak change to the 10-year values.

$K = 271$
$N_r = 12.4$

From Figure 7-5, R = 0.36 (attenuation of the 10-year flood)

Then: $Q_o = 4.05$ *cfs (Target 4.22 cfs, 2-year controls)*
$H = 4.55$ *ft (peak stage)*
$S = 11{,}932$ *cu ft = 0.274 ac ft (peak storage)*

Emergency Spillway Approximation

Assume $\Delta H = 1$ *foot and* $k_{ew} = 3.0$.

$Q_d = b\,[(H + \Delta H)^n - (H)^n] + k_{ew}\,(L_{ew})\,(\Delta H^{1.5})$

$14.4 - 4.05 = 1.9 * [(4.55 + 1)^{0.5} - (4.55)^{0.5}] + 3\,(L_{ew})\,(1^{1.5})$

$L_{ew} = 3.3$ *feet (OK)*

7.9 Example Problem

Example 7.9.1

This example demonstrates the application of the storage indication method presented in this chapter in routing an inflow hydrograph through a storage facility.

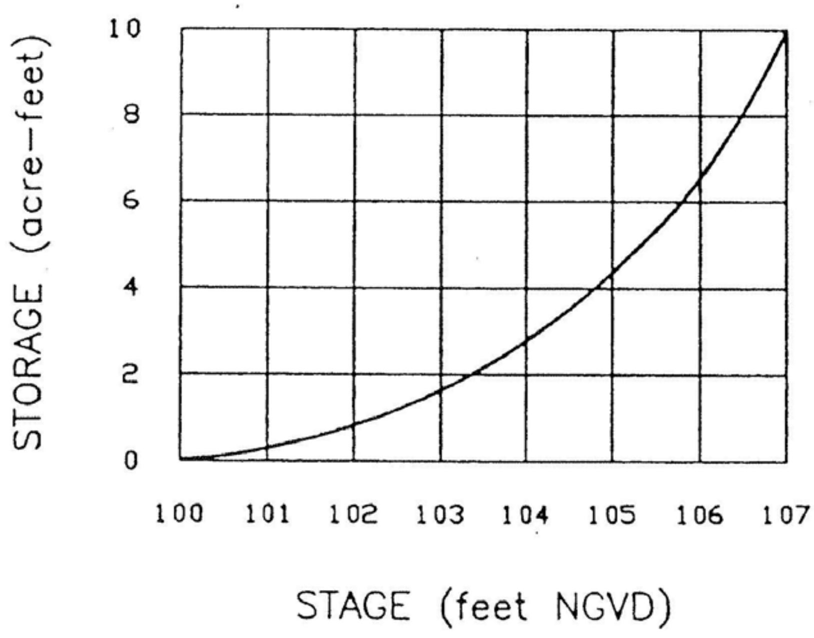
Inflow Hydrograph 7.9.2

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)*	*Inflo (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*		

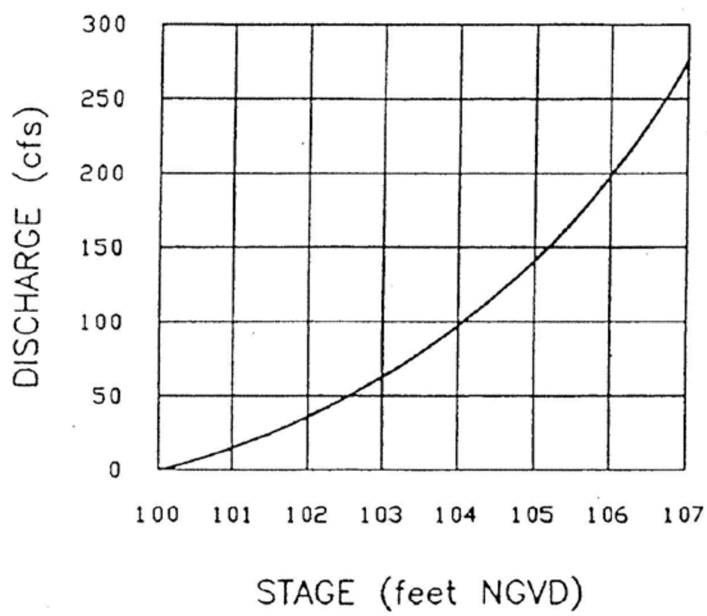
Stage-Storage Curve 7.9.3

From the physical characteristics of the storage facility to be used, a stage-storage is developed. The following curve will be used for this example



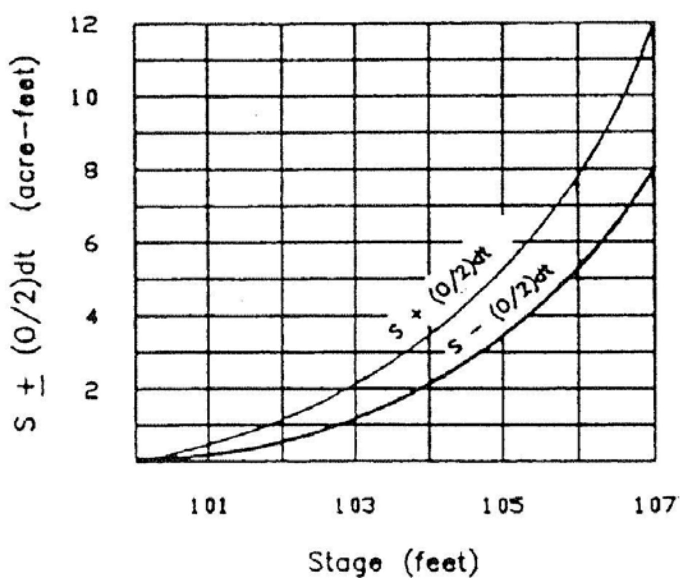
Stage Discharge Curve 7.9.4

From the characteristics of the outlet device, a stage-discharge curve is developed. The following curve will be used for this example



Storage Character-istics Curve 7.9.5

A routing time period (dt) 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. The following storage characteristics curves will be used for this example.



Routing Calculations 7.9.6

Following are routing calculations for this example. Table 7.3 on the next pages gives the results of these calculations.

1. *Given that S_1 – (O_1/2dt = 0.05 acre-foot for H_{s1} = 0 foot, find S_2 + (O_2/2)dt by adding 0.05 + 0.01 (column 5 value plus column 3 value) and tabulate 0.06 acre-foot in column 6 of Table 7.3.*

2. *Enter the S + (0/2)dt 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 7.3.*

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 7.3.*

4. *Assign the value of Hs2 to Hs1, find a new value of S1 – (O1/2)dt 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.*

Storage Facility Routing Calculations

(1) Time	(2) Inflow	(3) $[(I_1+I_2)/2]\,dt$	(4) H_1	(5) $S_1-(O_1/2)\,dt$	(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	0.61	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	106.05	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 7 – 3

7.10 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 may be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across the 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 unless protected from blockage.*

- *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.*

7.11 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-Mecklenburg 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 be sealed joints. The use of o-rings on reinforced concrete or neoprene gaskets for coupling on corrugated metal pipe are 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%.

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CHAPTER 8

ENERGY DISSIPATION

Chapter Table of Contents

8.1 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 dissipater becomes an intergral part of the drainage facility design.

8.2 Symbols and Definitions

To provide consistency within this chapter as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in many energy dissipation publications.

Table 8-1

SYMBOLS AND DEFINITIONS

Symbol	*Definition*	*Units*
A	*Cross section area*	*sq ft*
D	*Height of box culvert*	*ft*
d	*Depth of flow*	*ft*
d_{50}	*Size of riprap*	*ft*
d_{max}	*Maximum stone diameter*	*ft*
d_w	*Culvert width*	*ft*
g	*Acceleration due to gravity*	ft/s^2
L	*Length*	*ft*
L_a	*Riprap apron length*	*ft*
Q	*Rate of discharge*	*cfs*
TW	*Tailwater depth*	*ft*
V	*Velocity*	*ft/s*
W	*Width of apron*	*ft*

8.3 Design Criteria

General Criteria 8.3.1

Energy dissipaters shall be employed whenever the velocity of flows leaving a storm water management facility exceeds the erosive velocity of the downstream change system. Several standard energy dissipater designs have been documented by the U.S. Department of Transportation including impact basins, drop structures, stilling wells, and riprap.

Erosion Hazards 8.3.2

Erosion problems at the outlets of culvert 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 headcutting, 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 dissipater. Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

Recommended Dissipators 8.3.3

For many designs, the following outlet protection and energy dissipators provide sufficient protection at a reasonable cost.

- *Riprap apron*
- *Riprap outlet basins*
- *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 (Hydraulic Design of Energy Dissipators For Culverts and Channels) for design procedures of other energy dissipators.

8.4 Riprap Aprons

Uses 8.4.1

A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet velocity is less than or equal to 10 fps for pipes ≤ 48 inches in diameter.

Riprap aprons downstream from flared end sections must adhere to standard 20.23 of the Charlotte-Mecklenburg Land Development Standards Manual.

Procedure 8.4.2

The procedure presented in this section is taken from USDA, SCS (1975). 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, they 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 8-1 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 8-2 should be used.*

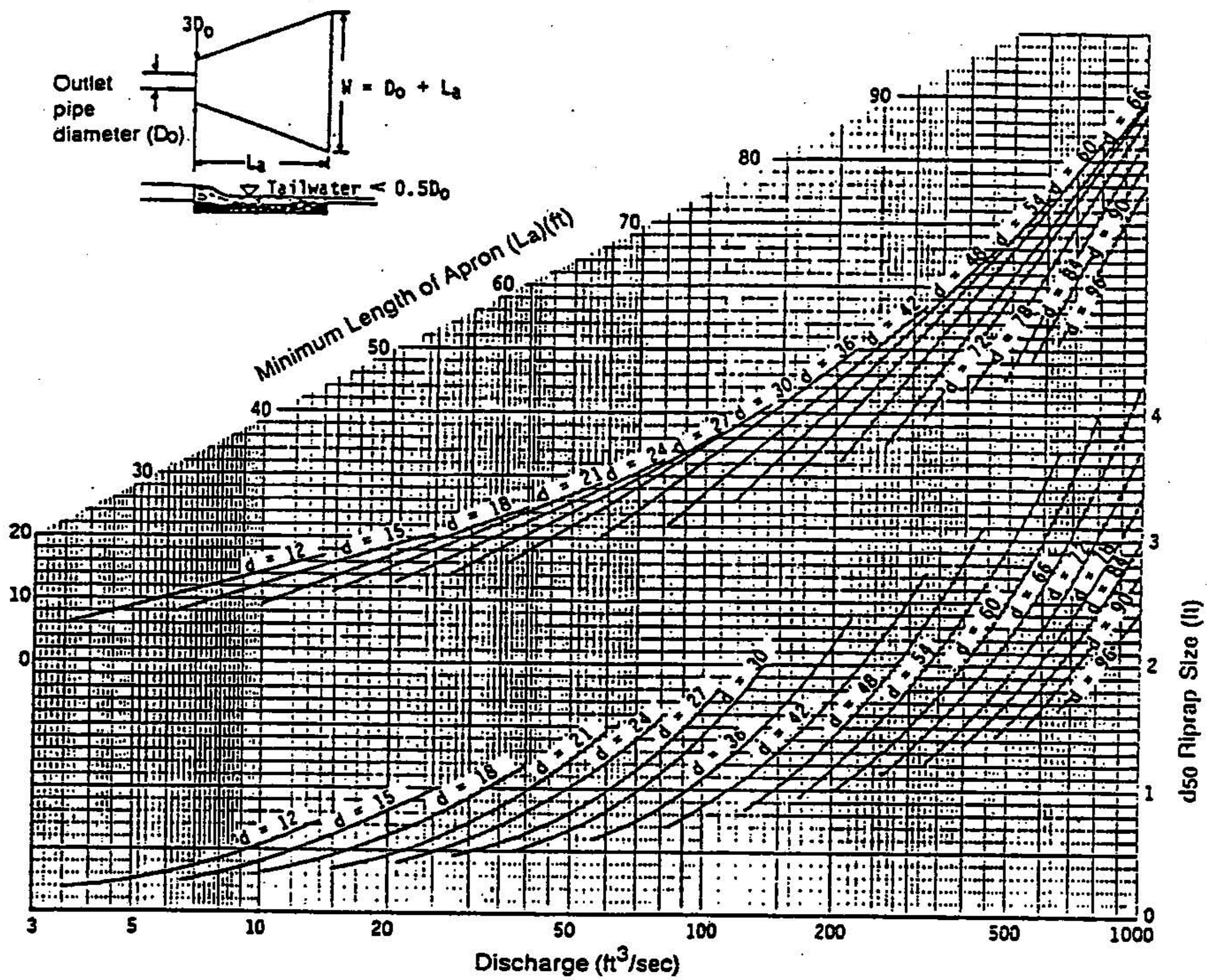
2. *Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figure 8-1 and 8-2. If tailwater conditions are uncertain find the values for both minimum and maximum conditions and size the apron as shown in Figure 8-3.*

   a. *For pipes flowing full:*

      Use the depth of flow, d_{50}, 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:*

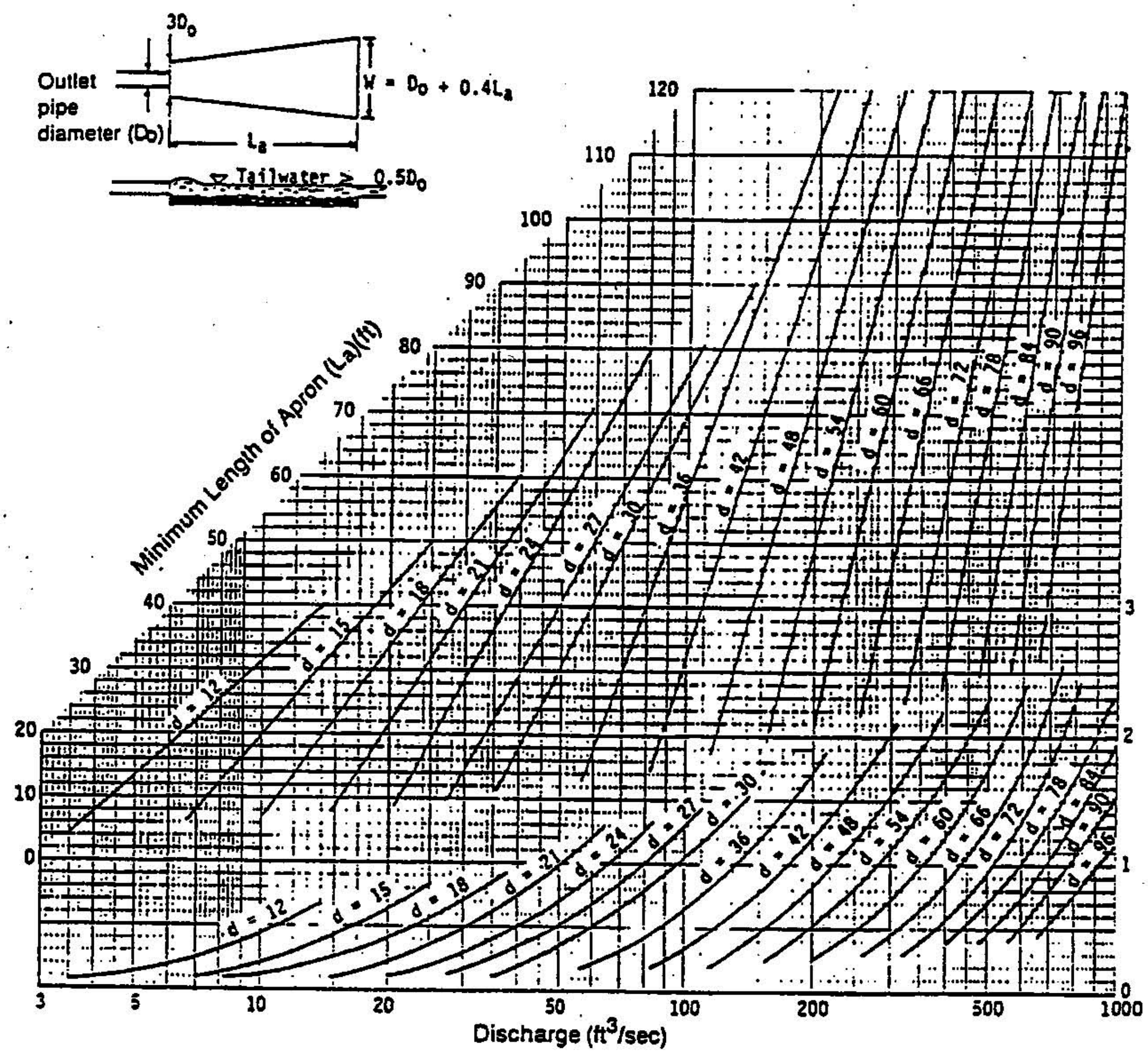
      Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the 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.



Curves may not be extrapolated.

Design of Riprap Apron Under Minimum Tailwater Conditions

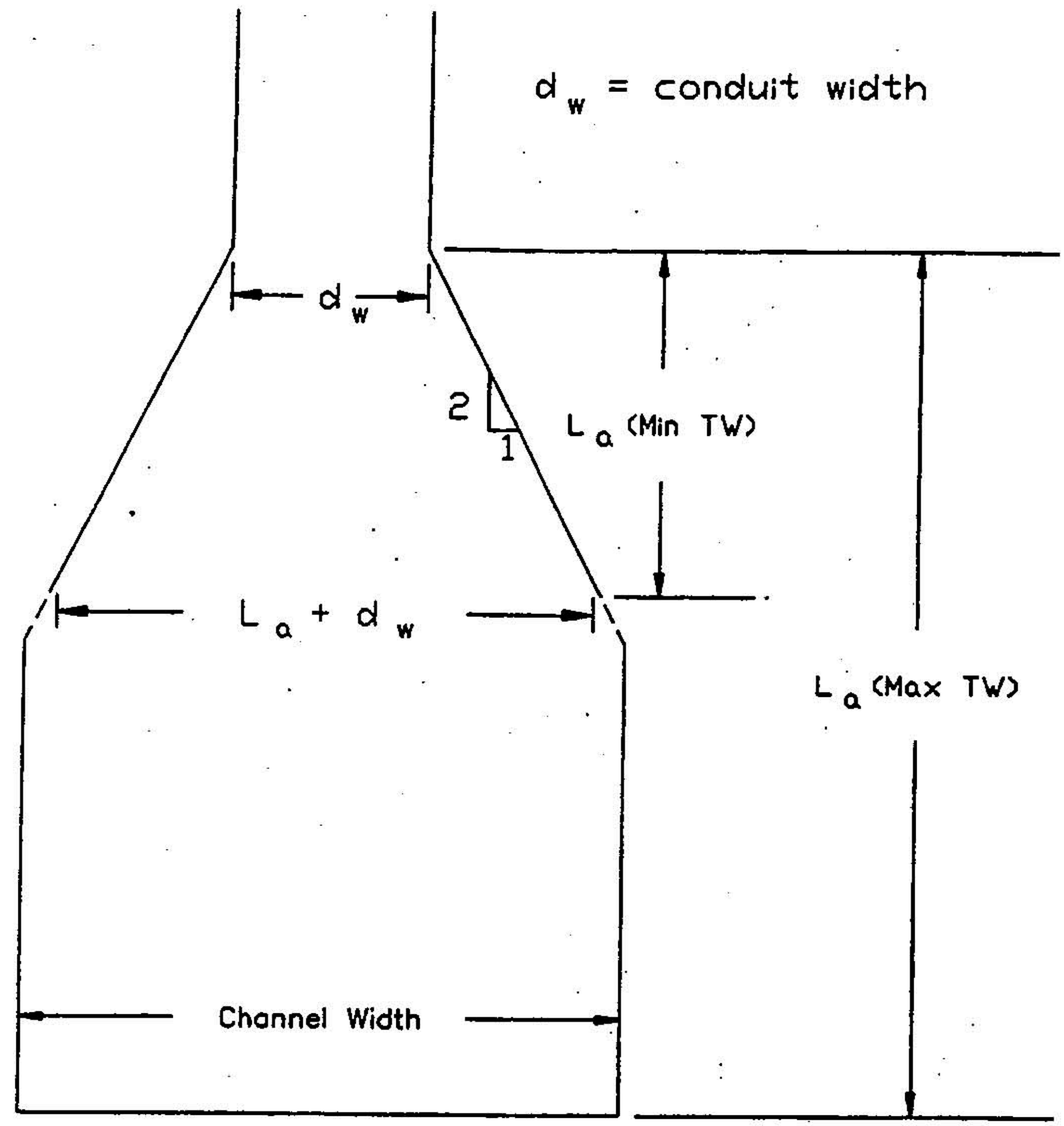
Figure 8-1



Curves may not be extrapolated.

Design of Riprap Apron Under Maximum Tailwater Conditions

Figure 8-2



Riprap Apron Schematic For Uncertain Tailwater Conditions

Figure 8-3

Procedure (continued)

c. *For Box culverts:*

Use the depth, d, in feet, and velocity, v, in feet/second. On the tower portion, of the appropriate figure, find the intersection of the d and v curves. then find the riprap median diameter, d50, from the scale on the right. From the lower d and v intersection point move vertically to the upper curve unfit intersecting the curve equal to the flow depth, d. Find the minimum apron length, La, using the scale on the left.

3. *If tailwater conditions are uncertain, the median riprap diameter should be larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 8-3. This will provide protection under either of the tailwater conditions.*

Design Considerations 8.4.3

The following items should be considered during the riprap apron design:

1. The maximum stone diameter should be 1.5 times the median riprap diameter.

$d_{max} = 1.5 \times d50$

d_{50} *= the median stone size in well-graded riprap apron*

2. The riprap thickness should be 1.5 times the maximum stone diameter or 10 inches, whichever is greater.

Apron thickness $= 1.5 \times d_{max}$

(Apron thickness may be reduced to 1.5 x d_{50}*, when an appropriate filter fabric is used under the apron).*

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 sidewalks 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. The apron should be extended as necessary until the, slope is gentle enough to prevent further erosion.

6. 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 or grout.

NY DOT Method 8.4.3

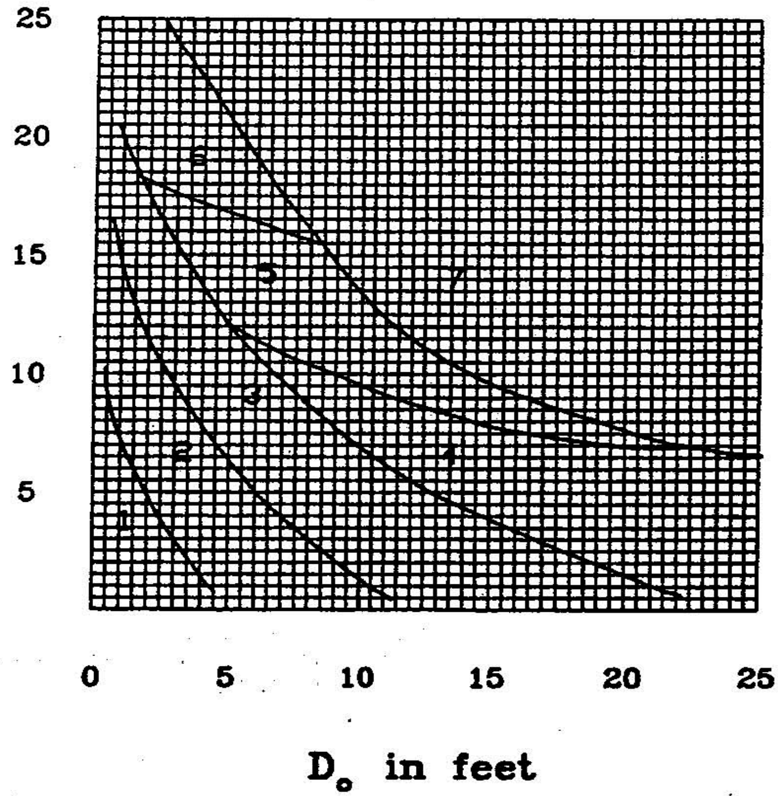
The procedure presented in this section is taken from New York Department of Transportation (1971). The riprap apron dimensions and stone sizes are determined from the charts and graphs in Figure 8-4 through 8-7.

The sizes for the riprap aprons may be determined using Figures 8-4 and 805 following the steps below.

1. *Estimate flow velocity V_0 at the culvert or paved channel outlet.*
2. *For pipe culverts, D is the diameter. For each pipe, box culverts, and paved channel outlets, $D_0 = A_0$ where A_0 = cross sectional area of flow at outlet.*
3. *For apron grades of 10% or steeper, use the recommended next higher zone (zones 1 through 6).*

The following steps may be followed to determine stone sizes for riprap aprons using Figures 8-6 and 8-7.

1. *Use Figure 8-6 to determine maximum stone size (e.g., for 12 fps = 20" or 550 lbs).*
2. *Use Figure 8-7 to determine acceptable size range for stone (for 12 fps it is 125-500 lbs. for 75% of stone, and the maximum and minimum range in weight should be 25-500 lbs).*



Riprap Apron Design Graph

Figure 8–4

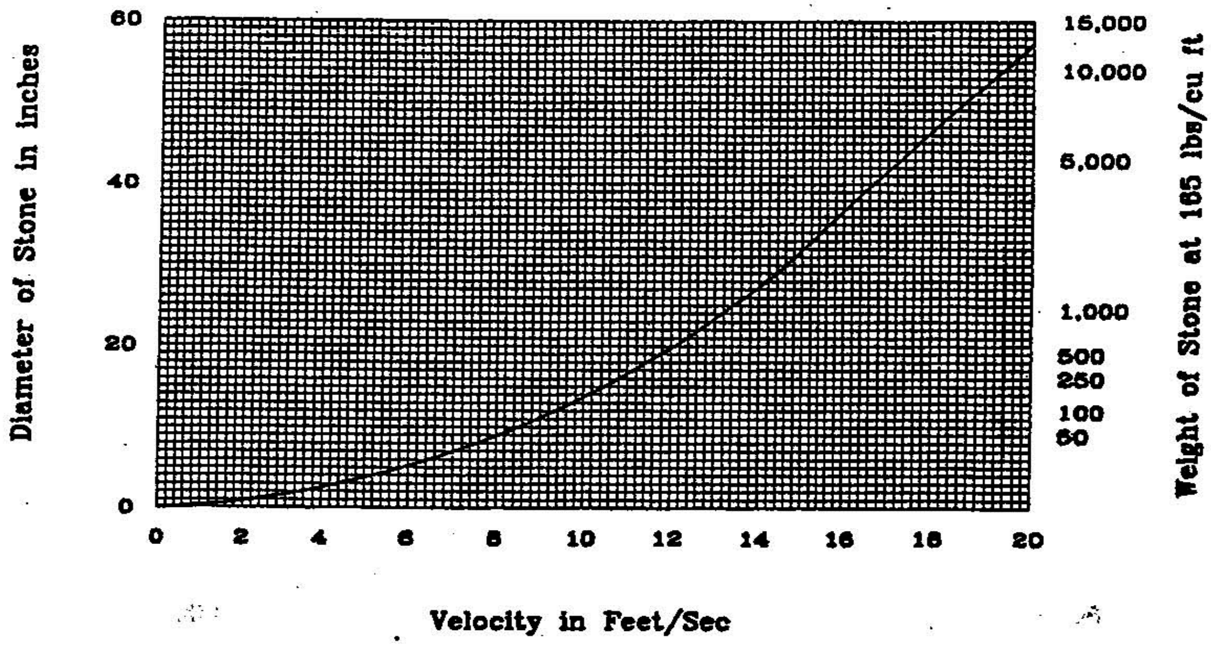
Source: "Bank and Channel Lining Procedures," New York Department of Transportation, Division of Design and Construction, 1971

ZONE	APRON MATERIAL	LENGTH OF APRON	
		TO PROTECT CULVERT L_1	TO PREVENT SCOUR HOLE L_2
1	Stone Filling (Fine)	3 x D.	4 x D.
2	Stone Filling (Light)	3 x D.	6 x D.
3	Stone Filling (Medium)	4 x D.	8 x D.
4	Stone Filling (Heavy)	4 x D.	8 x D.
5	Stone Filling (Heavy)	5 x D.	10 x D.
6	Stone Filling (Heavy)	6 x D.	12 x D.
7	Special study required (energy dissipators, stilling basin, or larger size stone).		

Riprap Apron Design Chart

Figure 8-5

Source: "Bank and Channel Lining Procedures," New York Department of Transportation, Division of Design and Construction, 1971



Maximum Stone Size for Riprap

Note: In determining channel velocities for stone linings and revetments, use the following coefficients of roughness:

	Diameter (inches)	Manning's 'n'	Min. Thickness of Lining (inches)
Fine	3	0.031	9
Light	6	0.035	12
Medium	13	0.040	18
Heavy	23	0.044	30

Figure 8–6

Source: "Bank and Channel Lining Procedures," New York Department Of Transportation, Division of Design and Construction, 1971.

Maximum Weight of Stone Required	Minimum and Maximum Range In Weight of Stones	Weight Range of 75% of Stones
(lbs)	(lbs)	(lbs)
150	25 – 150	50 – 150
200	25 – 200	50 – 200
250	25 – 250	50 – 250
400	25 – 400	100 – 400
600	25 – 600	150 – 600
800	25 – 800	200 – 800
1,000	25 – 1,000	250 – 1,000
1,300	25 – 1,300	325 – 1,300
1,600	25 – 1,600	400 – 1,600
2,000	25 – 2,000	600 – 2,000
2,700	25 – 2,700	800 – 2,700

Gradation of Riprap

Figure 8 – 7

Source : " Bank and Channel Lining Procedures, " New York Department of Transportation, Division of Design and Construction, 1971.

8.5 Example Problem

Example 1
8.5.1

*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_0 = 66 in = 5.5 ft*
   Therefore, 0.5 d_0 = 2.75 ft.

2. *Since TW = 2 ft, use Figure 8-1 for minimum tailwater conditions.*

3. *By Figure 8-1, 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:*

   $W = d + L_0 = 5.5 + 38 = 1.8$ 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.*

Example 2
8.5.2

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 design dimensions for a riprap apron.

1. *Compute 0.5 d_0 = 0.5 (5.0) = 2.5 ft.*

2. *Since TW = 5.0 ft is greater than 2.5 ft, use Figure 8-2 for maximum tailwater conditions.*

   $v = Q/A = 600/[(5)(10)] = 12$ ft/s

3. *On Figure 8-2, at the intersection of the curve, d_0 = 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 (0-4) = 0.6 foot.*

6. *Riprap depth = 1.5 d_{max} = 1.5 (0.6) 0.9 foot.*

References

Federal Highway Administration. 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14.

Federal Highway Administration. 1967. Use of Riprap for Bank Protection. Hydraulic Engineering Circular No. 11.

Searcy, James K. 1967. Use of Riprap for Bank Protection. Federal Highway Administration. Washington, D.