Town of Waxhaw

Stormwater Design Manual

Version 4.0

Updated: November 11, 2020

Revisions to the Stormwater Design Manual were prepared during the summer and fall of 2020. During this time the town has also been revising the Unified Development Ordinance (UDO). Upon completion and approval of the document revision by the Town Board of Commissioner the UDO will be re-issued as the Land Development Code (LDC). The approval and implementation of the LDC is anticipated to occur in late winter or early spring of 2021. Until the approval of the new LDC any references to the LDC should be considered a reference to the current UDO.

November 11, 2020

Dear Professional Engineer,

The purpose of this Storm Water Design Manual is to provide you with a guide to the planning and design of storm water control structures and systems in the Town of Waxhaw, N.C., that will meet the requirements of the Land Development Code (LDC), Town Code of Ordinances, and good engineering practice.

This Manual is intended to assist you in the design of storm water structures and systems. It has been modified from the Charlotte-Mecklenburg Storm Water Design Manual to suit 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 storm water. In order to protect the streams and public and private properties in the area, the Storm Water 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 or Storm Water Administrator 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.

The Storm Water Design Manual is 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 storm water needs. These updates will be made available through the town's website. Please feel free to contact us with any questions.

Sincerely,

Orion J. Holtey

Orion J. Holtey, PE Staff Engineer
Storm Water Administrator
Town of Waxhaw, N.C.

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Chapter 1

INTRODUCTION

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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 Storm water Ordinance is contained in Chapter 2. Following are the chapters included in the manual.

- Chapter 1 Introduction
- Chapter 2 Storm Water Management Ordinance
- 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

This manual 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. The most current version of this manual can be found on Waxhaw.com.

Erosion Control & Stormwater Details 1.5

For erosion and sediment control see the North Carolina Erosion and Sediment Control Planning and Design Manual. The Engineering Design & Construction Procedures 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 within 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. All offsite and on-site drainage area maps for pre and post construction.
E. A pre/post analysis using TR-55 methodology or other volume based hydrograph
F. Storm water management and drainage design calculations

Chapter 2

POST CONSTRUCTION STORMWATER ORDINANCE

Section: 58-19
ORD: 2020004

SECTION 58-19 OF THE TOWN OF WAXHAW, N.C.
POST CONSTRUCTION STORMWATER ORDINANCE

Purpose

The purpose of these regulations is to protect, maintain and enhance the public health, safety, environment and general welfare by establishing minimum requirements and procedures to control the adverse effects of increased post construction stormwater runoff and non-point source pollution associated with new development and redevelopment. It has been determined that proper management of construction-related and post construction stormwater runoff will minimize damage to public and private property and infrastructure, safeguard the public health safety and general welfare, and protect water and aquatic resources.

Permitting Authority

The Department of Environmental Quality for the State of North Carolina (Department) maintains permitting authority for post construction stormwater permitting in the Town of Waxhaw. Although Waxhaw is in Union County which is a Phase II tipped county the State has not assigned jurisdiction to the local entity. (January 2020). Permit applications are to be submitted to the Department of Water Quality. The Town of Waxhaw is in the Department of Environmental Quality Mooresville Region. Applications shall be prepared per current North Carolina administrative Code found under the Stormwater rules 15A NCAC 02H.

Drainage Plan Approval Requirements

(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. The drainage plan shall include water quality and detention requirements in accordance with the Town of Waxhaw Stormwater Design Manual. 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.

(3) Development plans shall be prepared per this Post Construction Ordinance (PCO) and all applicable State, County and Local regulations.

Required Contents of Drainage Plan.

The drainage plan submitted for approval under this Part shall include a site plan showing existing and proposed buildings, stormwater drainage facilities, site construction plans, grading plans including drainage system; drainage facility design data, including a drainage area map which identifies all the Best Management Practices (BMPs), engineering calculations, area of impervious cover, and total land area; and any other appropriate information requested by the Town Engineer.

Standards for Plan Approval

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

(1) The Town Engineer shall review the drainage plan for compliance with the standards contained in the Town of Waxhaw Stormwater Design Manual, the current edition of the Engineering Design and Construction Standards Manual, and any other relevant and appropriate standard established by the Town Engineer.

(2) The Town Engineer will not approve a drainage plan with increased impervious ground cover, 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.

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Design Manual

(1) REFERENCE TO DESIGN MANUAL. The Town Engineer/Stormwater Administrator shall use the policy, criteria, and information, including technical specifications and standards, in the Design Manual as the basis for decisions about the design, implementation and performance of structural and non-structural stormwater BMPs. The Design Manual includes a list of acceptable stormwater treatment practices, including the specific design criteria for each stormwater practice. Stormwater treatment practices that are designed and constructed in accordance with these design and sizing criteria will be presumed to meet the minimum water quality performance standards of the PCO and the Phase II laws. Failure to construct stormwater treatment practices in accordance with these criteria may subject the violator to a civil penalty.

(2) RELATIONSHIP OF DESIGN MANUAL TO OTHER LAWS AND REGULATIONS. If the specifications or guidelines of the Design Manual are more restrictive or apply a higher standard than other laws or regulations, that fact shall not prevent application of the specifications or guidelines in the Design Manual.

(3) CHANGES TO STANDARDS AND SPECIFICATIONS. Standards, specifications, guidelines, policies, criteria, or other information in the Design Manual in effect at the time of acceptance of a complete application shall control and shall be utilized in reviewing the application and in implementing the PCO with regard to the application.

(4) AMENDMENTS TO DESIGN MANUAL. The Design Manual may be updated and expanded from time to time, based on advancements in technology and engineering, improved knowledge of local conditions, or local monitoring or maintenance experience. Prior to amending or updating the Design Manual, proposed changes shall be generally publicized and made available for review, and an opportunity for comment by interested persons shall be provided. The current Design Manual is available on the Town of Waxhaw website.

Relationship to Other Laws

(1) CONFLICT OF LAWS. The PCO is not intended to modify or repeal any other ordinance, rule, regulation or other provision of law. The requirements of the PCO are in addition to the requirements of any other ordinance, rule, regulation or other provision of law, and where any provision of the PCO imposes restrictions different from those imposed by any other ordinance, rule, regulation or other provision of law, whichever provision is more restrictive or imposes higher protective standards for human or environmental health, safety, and welfare, shall control.

(2) PRIVATE AGREEMENTS. The PCO is not intended to revoke or repeal any easement, covenant, or other private agreement. However, where the regulations of the PCO are more restrictive or impose higher standards or requirements than such easement, covenant, or other private agreement, then the requirements of the PCO shall govern. Nothing in the PCO shall modify or repeal any private covenant or deed restriction, but such covenant or restriction shall not legitimize any failure to comply with the PCO. In no case shall the Town be obligated to enforce the provisions of any easements, covenants, or agreements between private parties.

Administration and Procedures

(1) An approved stormwater permit from the Department of Environmental Quality shall be submitted prior to the approval of the Drainage plan from the Town.

(2) The stormwater permit number shall be provided on the Final Plat and the approved operations and maintenance plan shall be submitted for Final Plat approval.

Standards

All development and redevelopment to which the PCO applies shall comply with the standards of this section.

(1) All built-upon area for development and redevelopment subject to the requirements of this PCO shall be at a minimum thirty feet (30') landward of all perennial and intermittent surface waters. This built-upon area setback can be located within the stream buffer area defined by this PCO, but any disturbances within the built-upon area setback must also comply with the regulated floodways. A surface water shall be deemed present if the feature is approximately shown on either the most recent version of the soil survey map prepared by the Natural Resources Conservation Service of the United States Department of Agriculture or the most recent version of the 1:24,000 scale (7.5 minute) quadrangle topographic maps prepared by the United States Geologic Survey (USGS). An exception to this requirement shall be granted if one or more of the following is satisfied and documented:

a. When a landowner or other affected party believes that the maps have inaccurately depicted surface waters, he or she shall consult the Army Corps of Engineers. The Army Corps of Engineers shall make on-site determinations. Surface waters that appear on the maps shall not be subject to this standard if this on-site determination shows that they fall into one of the following categories:

i. Ditches and manmade conveyances other than modified natural streams unless constructed for navigation or boat access;

ii. Manmade ponds and lakes located outside natural drainage ways;

iii. Ephemeral (stormwater) streams.

b. An unnecessary hardships would result from the strict application of this requirement.

c. Based on a determination by the Army Corps of Engineers, a lack of practical alternatives exists for accomplishing the basic purpose of the project in a manner that would avoid or result in less adverse impact to surface waters considering the potential for a reduction in size, configuration, or density and all alternative designs.

Standards for Stormwater Control Measures

(1) Evaluation according to contents of the Design Manual. All stormwater control measures and stormwater treatment practices (also referred to as Best Management Practices, or BMPs) required under the PCO shall be evaluated by the Town Engineer/Stormwater Administrator according to the policies, criteria, and information, including technical specifications, standards and the specific design criteria for each stormwater best management practice contained in the Design Manual. The Town Engineer/Stormwater Administrator shall determine whether these measures will be adequate to meet the requirements of the PCO.

(2) Determination of Adequacy; Presumptions and Alternative. Stormwater treatment practices that are designed, constructed, and maintained in accordance with the criteria and specifications in the Design Manual will be presumed to meet the minimum water quality and quantity performance standards of the PCO. Whenever an applicant proposes to utilize a practice or practices not designed and constructed in accordance with the criteria and specifications in the Design Manual, the applicant shall have the burden of demonstrating that the practice(s) will satisfy the minimum water quality and quantity performance standards of the PCO before it can be approved for use. The Town Engineer/Stormwater Administrator may require the applicant to provide such documentation, calculations, and examples as necessary to determine whether such an affirmative showing is made.

(3). A Digital Record Submittal shall be provided to the Town of Waxhaw. Upon submittal of as-built plans, the location of storm drainage pipes, inlets, outlets and the location of all BMPs as well as Tree Save Area must be delivered to the Town Engineer/Stormwater Administrator in the digital format specified in the Design Manual.

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<u>Maintenance</u>

(1) It shall be the responsibility of the Home owner or Homeowner's Association to provide for maintenance (as specified in the Design Manual) of structural BMPs that are installed pursuant to the PCO. Maintenance responsibility shall be clearly identified on the construction documents and final plat.

(2) All BMPs shall include adequate and perpetual access and sufficient area, by easement or otherwise, for inspection, maintenance, and repair or reconstruction.

(3) The owner of a structural BMP installed pursuant to the PCO and as identified on the final plat shall maintain and operate the BMP so as to preserve and continue its function in controlling stormwater quality and quantity at the degree or amount of function for which the structural BMP was designed.

(4) The following provisions apply to trees, contained in permitted Tree Save Areas or in BMPs that are damaged or removed:

a. For trees damaged or removed due to natural disasters, the owner shall be required to replace the trees in accordance with the tree save criteria within a timeframe specified by the Zoning Director or Stormwater Administrator.

b. For trees damaged or removed due to reasons other than (a) above, the owner shall be required to replace the trees in accordance with the tree save criteria within a timeframe specified by the Zoning Director or Stormwater Administrator with the following exception, the trees shall be replaced at twice the specified density. In addition, the owner may be subject to fines.

(5) An annual maintenance inspection and report is required by the permit. The person responsible for maintenance of any BMP installed pursuant to the PCO shall submit to the Town Engineer/Stormwater Administrator an inspection report from a qualified registered North Carolina professional engineer or landscape architect performing services only in their area of competence. An original inspection report shall be provided to the Town of Waxhaw beginning one year from the date of as-built certification and each year thereafter on or before the anniversary date of the as-built certification.

(6) At the time that as-built plans are provided to the Stormwater Administrator and prior to final approval of a project for compliance with the PCO, but in all cases prior to placing the BMPs in service, the applicant or owner of the site must execute an operation and maintenance agreement that shall be binding on all current and subsequent owners of the site, portions of the site, and lots or parcels served by the structural BMP. This maintenance agreement shall be recorded along with the Final Plat to the register of deeds. Failure to execute an operation and maintenance agreement within the time frame specified by the Town Engineer/Stormwater Administrator may result in assessment of penalties. Until the transference of all property, sites, or lots served by the structural BMP, the original owner or applicant shall have primary responsibility for carrying out the provisions of the maintenance agreement. At the discretion of the Town Engineer/Stormwater Administrator, certificates of occupancy may be withheld pending receipt of an operation and maintenance agreement. The operation and maintenance agreement shall require the owner or owners to maintain, repair and, if necessary, reconstruct the structural BMP, and shall state the terms, conditions, and schedule of maintenance for the structural BMP. In addition, it shall grant to the Town a right of entry in the event that the Town Engineer/Stormwater Administrator has reason to believe it has become necessary to inspect, monitor, maintain, repair, or reconstruct the structural BMP. However, in no case shall the right of entry, of itself, confer an obligation on the Town to assume responsibility for the structural BMP. The operation and maintenance agreement must be approved by the Town Engineer/Stormwater Administrator prior to plan approval, and it shall be referenced on the final plat.

(7) For all structural BMPs required pursuant to the PCO that are to be or are owned and maintained by a homeowners' association, property owners' association, or similar entity, the required operation and maintenance agreement shall include the provisions described in the Design Manual.

(8) Inspections and inspection programs by the Town may be conducted or established on any reasonable basis, including but not limited to routine inspections; random inspections; inspections based upon

complaints or other notice of possible violations; and joint inspections with other agencies inspecting under environmental or safety laws. Inspections may include, but are not limited to, reviewing maintenance and repair records; sampling discharges, surface water, groundwater, and material or water in BMPs; and evaluating the condition of BMPs. If the owner or occupant of any property refuses to permit such inspection, the Town Engineer/Stormwater Administrator shall proceed to obtain an administrative search warrant pursuant to G.S. 15-27.2 or its successor. No person shall obstruct, hamper or interfere with the Town Engineer/Stormwater Administrator while carrying out his or her official duties.

(9) The owner of each structural BMP shall keep records of inspections, maintenance, and repairs for at least five (5) years from the date of creation of the record and shall submit the same upon reasonable request to the Town Engineer/Stormwater Administrator.

(10) Every structural BMP installed pursuant to the PCO shall be made accessible for adequate inspection, maintenance, reconstruction and repair by a maintenance easement. The easement shall conform to standards listed in the Design Manual, and be recorded and its terms shall specify who may make use of the easement and for what purposes.

(11) This ordinance prohibits the direct discharge of runoff to the roadway or storm sewer system from private property that is not explicitly identified in the approved construction documents. Examples of such discharge are but would not be limited to extending roof drains to the right of way or direct discharge of pool water to the storm drain.

Definitions

In the construction of the PCO, the definitions contained in this section shall be observed and applied, unless other provisions of the PCO specifically indicate otherwise. The following words and terms when used in the interpretation and administration of the PCO shall have the meaning set forth below except where otherwise specifically indicated.

Best Management Practices (BMPs) – Shall mean a structural management facility used singularly or in combination for stormwater quality and quantity treatment to achieve water quality protection goals.

Buffer – Shall mean a natural or vegetated area through which stormwater runoff flows in a diffuse manner so that the runoff does not become channelized and which provides for infiltration of the runoff and filtering of pollutants.

Buffer Widths – Shall mean viewed aerially, the stream buffer width is measured horizontally on a line perpendicular to the surface water, landward from the top of the bank on each side of the stream.

Built-Upon Area (BUA) – Shall mean that portion of a development project that is covered by impervious or partially impervious surface including, but not limited to, buildings; pavement and gravel areas such as roads, parking lots, and paths; and recreation facilities such as tennis courts. “Built-upon area” does not include a wooden slatted deck or the water area of a swimming pool.

Commercial Development – Shall mean any development that is not residential development as defined below.

Design Manual – Shall mean the stormwater design manual prepared for use in the Town of Waxhaw which shall be used in conjunction with the stormwater design manual developed for use in Phase II jurisdictions by the Department for the proper implementation of the requirements of the federal Phase II stormwater program. All references to the Design Manual are to the latest published edition or revision.

Department - The North Carolina Department of Environmental Quality

Development – Shall mean new development created by the addition of built-upon area to land void of built-upon area as of the Effective Date of the Post Construction Regulations.

Disturbance – Shall mean any use of the land by any person or entity which results in a change in the natural cover or topography of the land.

Drainage Area – Shall mean that area of land that drains to a common point on a project site.

Floodplain – Shall mean the low, periodically-flooded lands adjacent to streams. For land use planning purposes, the regulatory floodplain is usually viewed as all lands that would be inundated by the Regulatory Flood.

Industrial Uses – Shall mean land used for industrial purposes only; commercial (or other non-industrial) businesses operating on industrially zoned property shall not be considered an industrial use.

Multi-family – Shall mean a group of two or more attached, duplex, triplex, quadruplex, or multi-family buildings, or a single building of more than two (2) units constructed on the same lot or parcel of land under single ownership, and planned and developed with a unified design of buildings and coordinated common open space and service areas.

Non-Point Source (NPS) Pollution – Shall mean forms of pollution caused by sediment, nutrients, organic and toxic substances originating from land use activities and carried to lakes and streams by surface runoff.

Owner – Shall mean the legal or beneficial owner of land, including but not limited to a fee owner, mortgagee or vendee in possession, receiver, executor, trustee, or long term or commercial lessee, or any other person or entity holding proprietary rights in the property or having legal power of management and control of the property. “Owner” shall include long term commercial tenants; management entities, such as those charged with or engaged in the management of properties for profit; and every person or entity having joint ownership of the property. A secured lender not in possession of the property does not constitute an owner, unless the secured lender is included within the meaning of “owner” under another description in this definition, such as a management entity.

Person(s) – Shall mean any individual, partnership, firm, association, joint venture, public or private corporation, trust, estate, commission, board, public or private institution, utility, cooperative, interstate body, or other legal entity.

Redevelopment – Shall mean rebuilding activities on land containing built-upon area as of the effective date of the PCO and where any pre-existing impervious surface remains intact and is not removed during the rebuilding or redevelopment process.

Residential Development – Shall mean a development containing dwelling units with open yards on at least two sides where land is sold with each dwelling unit.

Stormwater Administrator – Shall mean the position or individual that has been designated by the Town Engineer to administer and enforce the PCO.

Stormwater Management Permit – Shall mean a permit required for all development and redevelopment unless exempt pursuant to the PCO and the Department, which demonstrates compliance with the PCO.

Top of Bank – Shall mean the landward edge of the stream channel during high water or bankfull conditions at the point where the water begins to overflow onto the floodplain.

Topsoil – Shall mean natural, fertile soil capable of sustaining vigorous plant growth that is of uniform composition throughout with an admixture of subsoil, has an acidity range of pH 5.5 - 7.0.

Townhomes – Shall mean attached dwellings where a lot is created for each unit.

Tree Save Areas – Shall mean land that consists of natural areas containing trees and other natural shrubs consisting of either undisturbed areas or disturbed areas that have been replanted in accordance with the criteria established in the Unified Development Ordinance.

Section: 58-19
ORD: 2020004

Effective Date: This ordinance shall be effective on the 10th day of March 2020.

Ronald P. Pappas, Mayor

Melody Shuler, Town Clerk

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 included 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 3.6.
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.

[1]Size limitations refers to the subwatershed 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. If alternative modeling programs are used the designer should request for pre-approval prior to preparing and submitting any design calculations.

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 or existing use; whichever is greater.

All detention facilities shall be designed to maintain the pre-developed runoff rate for the 2-year and the 10-year 24-hour 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.

- 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 peak 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

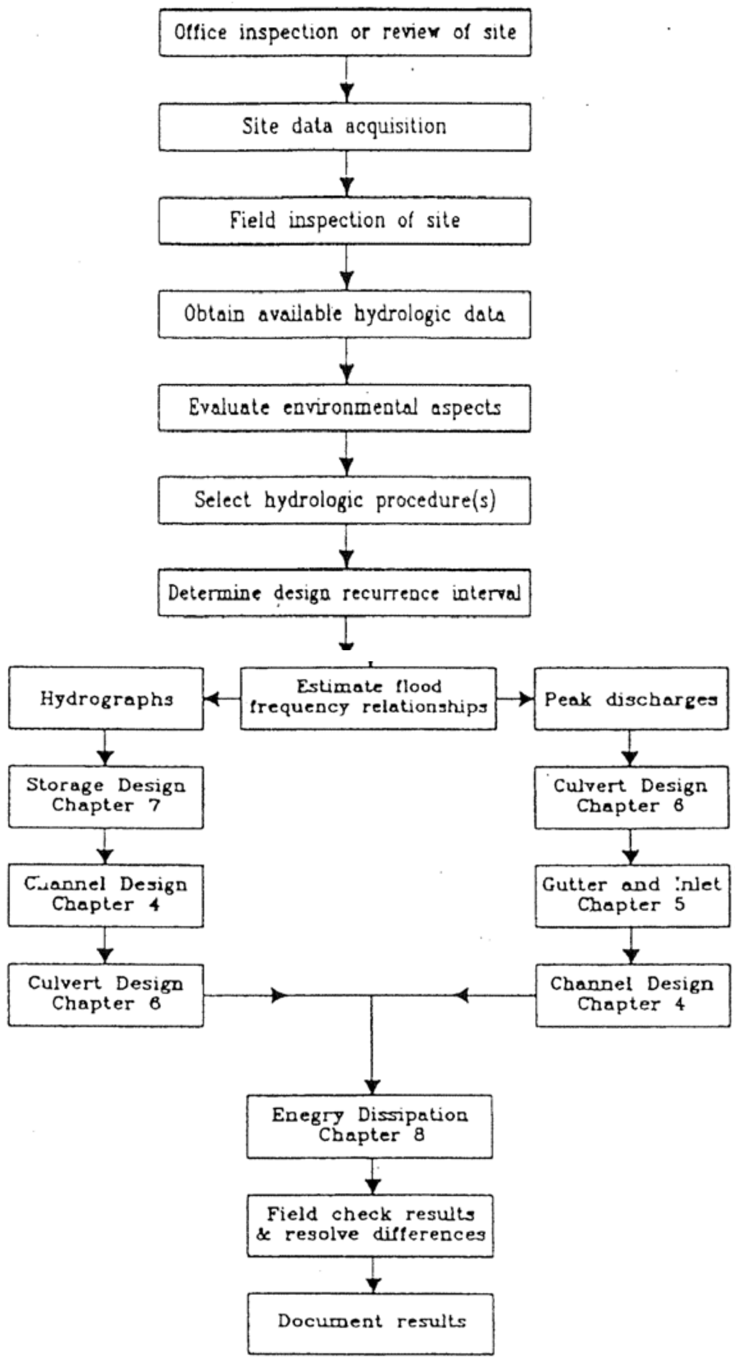
Symbols	Definition	Units
A or a	Drainage area	acres
A_f	Channel flow area	ft2
B	Channel bottom width	ft
C	Runoff Coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
D	Depth of flow	ft
d	Time interval	hours
F_p	Pond and swamp adjustment factor	
I	Runoff intensity	in./hr.
I	Percent of impervious 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 & 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 (below about the 10-year event threshold). As the frequency of a flood event 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. 24 hr
Ditch systems	10 yr. 24 hr
Culverts (subdivision streets)	25 yr. 24 hr
Culverts (thoroughfare roads)	50 yr. 24 hr
Culverts over regulated floodways	100 yr. 24 hr

Rainfall Intensity 3.5.2

The most recent NOAA Atlas 14 or other acceptable sources may be used for hydrologic analysis
Location information for Waxhaw shall be obtained from the Monroe, North Carolina Station Monroe 4E (31-5771).
As of the date of this revision the link to North Carolina is:
https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nc

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:

$$\mathbf{Q = CIA} \qquad \textbf{(3.1)}$$

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 3-4 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:

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

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

Table 3-3

Frequency Factors For Rational Formula

Recurrence Interval (years)	C_f
10 or less	1.0
25	1.1
50	1.2
100	1.25

Time of Concentration 3.6.4

Time of concentration (Tc) is the time required for a particle of water to flow from the hydraulically most distant point on a watershed to the design point in question. This definition indicates that Tc is a function of length and velocity.

Use NCRS Module 206A to calculate time of concentration.

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 should be obtained from the most recent NOAA Atlas 14 or other acceptable sources.

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 3-4 below gives the recommended runoff coefficients for the Rational Method.

Table 3-4 Recommended Runoff Coefficient Values

Description of Area	Runoff Coefficient (C)
Lawns	0.30
Wooded	0.25
Streets	0.95
Gravel areas	0.55
Drives, walks, roofs	0.95
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 3-4 by using percentages of different land uses. The composite procedure can be applied to an entire drainage area or to typical “sample” blocks as a guide to selection 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 3-4 to be 0.30.
Time of Concentration	From Module 206A Sheet Flow + Shallow Concentrated Flow + Channel Flow = Total t_c For this example assume Total t_c = 15 min
Rainfall Intensity	The most recent NOAA Atlas 14 or other acceptable sources may be used for hydrologic analysis Location information for Waxhaw shall be obtained from the Monroe, North Carolina Station Monroe 4E (31-5771).

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined in the following table by utilizing the values from Table 3-4.

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:

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

$Q_{100} = C_f CIA = 1.25 \times .54 \times 7.09$ 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 NRCS Unit Hydrograph

Introduction
3.8.1

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

The NRCS method includes the following basic steps:

1. Determination of composite curve number which represent different land uses within the drainage area.

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

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

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

Unit
Hydrograph
3.8.2

NRCS uses unit hydrograph in Watershed Evaluation, Floodplain Delineation, Design and Damn Breach. Basic principles reason that for a given watershed, all hydrographs resulting from rains of the same period of excess (unit duration) has equal time bases. A unit hydrograph is a hydrograph for a specific time period of rainfall excess (runoff) and uniform distribution and whose volume is equal to one inch of water over the entire watershed.

Unit rainfall duration refers to the time period of rainfall producing runoff (rainfall excess). The unit hydrograph resulting from a six-hour excess rainfall duration is referred to as a six-hour unit hydrograph. The precipitation is assumed to occur uniformly over the entire watershed and has a uniform time distribution.

Unit hydrographs are used to estimate flood hydrographs by multiplying each ordinate of unit hydrograph by the volume of runoff.

NRCS provides a dimensionless unit hydrograph methodology based upon the many natural unit Hydrographs from small watersheds in widely varying locations.

Equations and
Concepts
3.8.3

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.

Rainfall- The NRCS method applicable to the Town of Waxhaw area is based on a storm which has a Type II time distribution. Figure 3-4 shows this distribution. To use this distribution it is necessary for the user to obtain the 24-hour rainfall volume for the frequency of the design storm. This volume is

Equations and Concepts (continued)

then distributed according to Figure 3-4.

Rainfall-Runoff Equation- The following runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

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

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 runoff equation for estimating I_a is:

$$\mathbf{I_a = 0.2S} \qquad \mathbf{(3.13)}$$

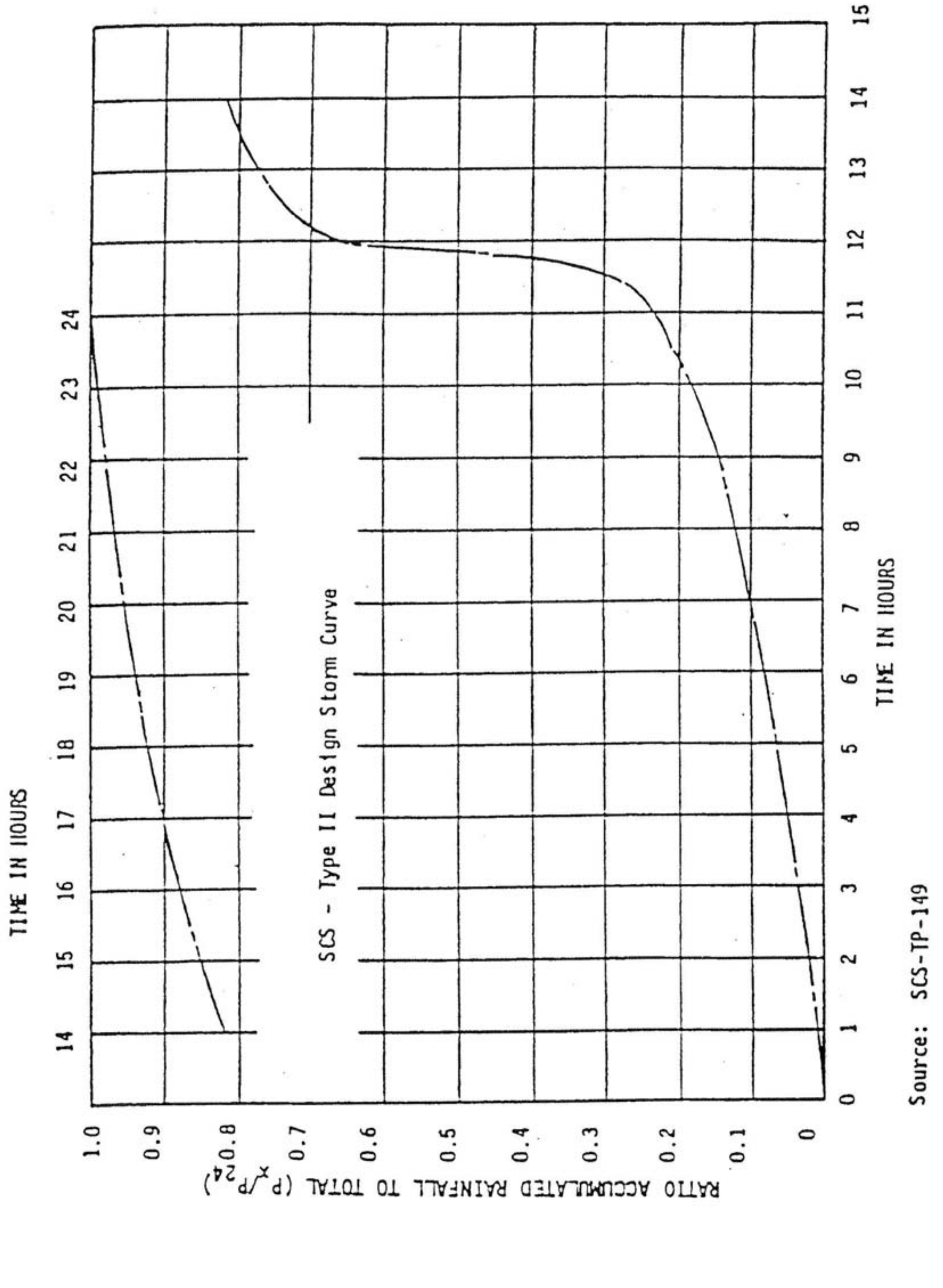
Substituting 0.2S for I_a in equation 3.12, the rainfall-runoff equation becomes:

$$\mathbf{Q = (P - 0.2S)^2 / (p + 0.8S)} \qquad \mathbf{(3.14)}$$

Where: S = 1000/CN – 10

CN = NRCS curve number

Figure 3-2 Solutions to Runoff Equations, on page 3-19 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.



SCS Type II Design Storm Curve

Figure 3-1

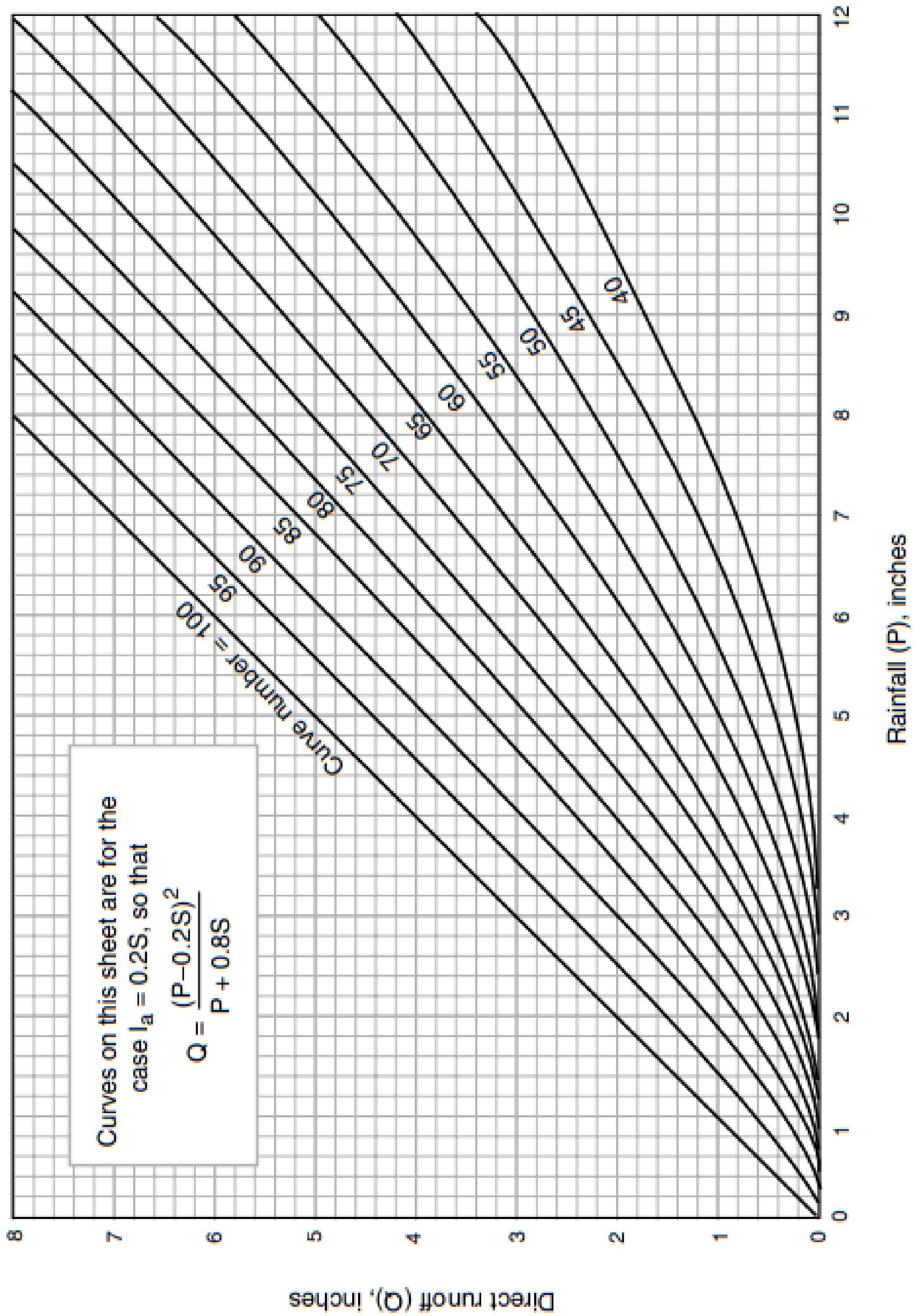


Figure 3-2 Solutions to Runoff Equation

Runoff Factor 3.8.4

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 3-5 below. Soil Survey maps can be obtained from local SCS offices.

Table 3-5 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 3-6 on the next page gives recommended curve number values for a range of Different land uses, and soil types.

Urban Modifications 3.8.5

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, consider whether impervious areas connect directly to the drainage system, or to lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 3-10 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 3-6

Runoff Curve Numbers[1]

Cover description: Cover type and hydrologic condition	Average percent impervious area[2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Agricultural Lands					
Pasture, grassland or range (continuous for age for grazing)[4]					
Poor hydrologic condition		68	79	86	89
Fair hydrologic condition		49	69	79	84
Good hydrologic condition		39	61	74	80
Woods					
Poor hydrologic condition		45	66	77	83
Fair hydrologic condition		36	60	73	79
Good hydrologic condition		30	55	70	77
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

1 Average runoff condition, and Ia = 0.2S.

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

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

4 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 litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

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

Travel Time Estimation 3.8.6

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 3.8.6.1

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:

$$\mathbf{T_t = \frac{L \times 0.0167}{V}} \qquad \textbf{(3.15)}$$

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

Time of Concentrations 3.8.6.2

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.

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

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

Sheet Flow 3.8.6.3

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. Also please note, when designing a drainage system, the sheet flow path is not necessarily the before and after development and grading operations have been completed. **Selecting sheet flow paths in excess of 100 feet in developed areas and 300 feet in undeveloped areas should be done only after careful consideration.**

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

$$\mathbf{T_t = [0.42\ (nL)^{0.8} / (P_2)^{0.5}(S)^{0.4}]} \qquad \textbf{(3.17)}$$

Where: T_t = travel time (min),
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:

$$\mathbf{Tt = [0.238\ (nL)^{0.8}] / (S)^{0.4}]} \qquad \mathbf{(3.18)}$$

Thus the final equations for paved and unpaved areas are:

Paved (n = 0.011)

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

$$\mathbf{V = 2.56(S)^{0.4}(L)^{0.2}} \qquad \mathbf{(3.20)}$$

Unpaved (n = 0.024)

$$\mathbf{T_t = 0.076\ [(L)^{0.8} / (S)^{0.4}]} \qquad \mathbf{(3.21)}$$

$$\mathbf{V = 0.22(S)^{0.4}(L)^{0.2}} \qquad \mathbf{(3.22)}$$

Where: **V = velocity (fps)**

T_t = Travel time (min)

Table 3-7

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: TR-55, Section Edition, June 1986.

Shallow Concentrated Flow 3.8.6.4

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 3-3 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 3-3, or the following equations. These equations can also be used for slopes less then 0.005 ft/ft.

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

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

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 3-3 or equations 3.23 or 3.24, use Equation 3.15 to estimate travel time for the shallow concentrated flow segment.

Open Channels 3.8.6.5

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 $$\mathbf{V = [1.49(r)^{2/3}(s)^{1/2}]/n} \qquad \mathbf{(3.25)}$$

Where: V = average velocity ft/s),
r = hydraulic radius (ft) and is equal to a/p_w,
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft),
s = slope of the hydraulic grade line ft/ft 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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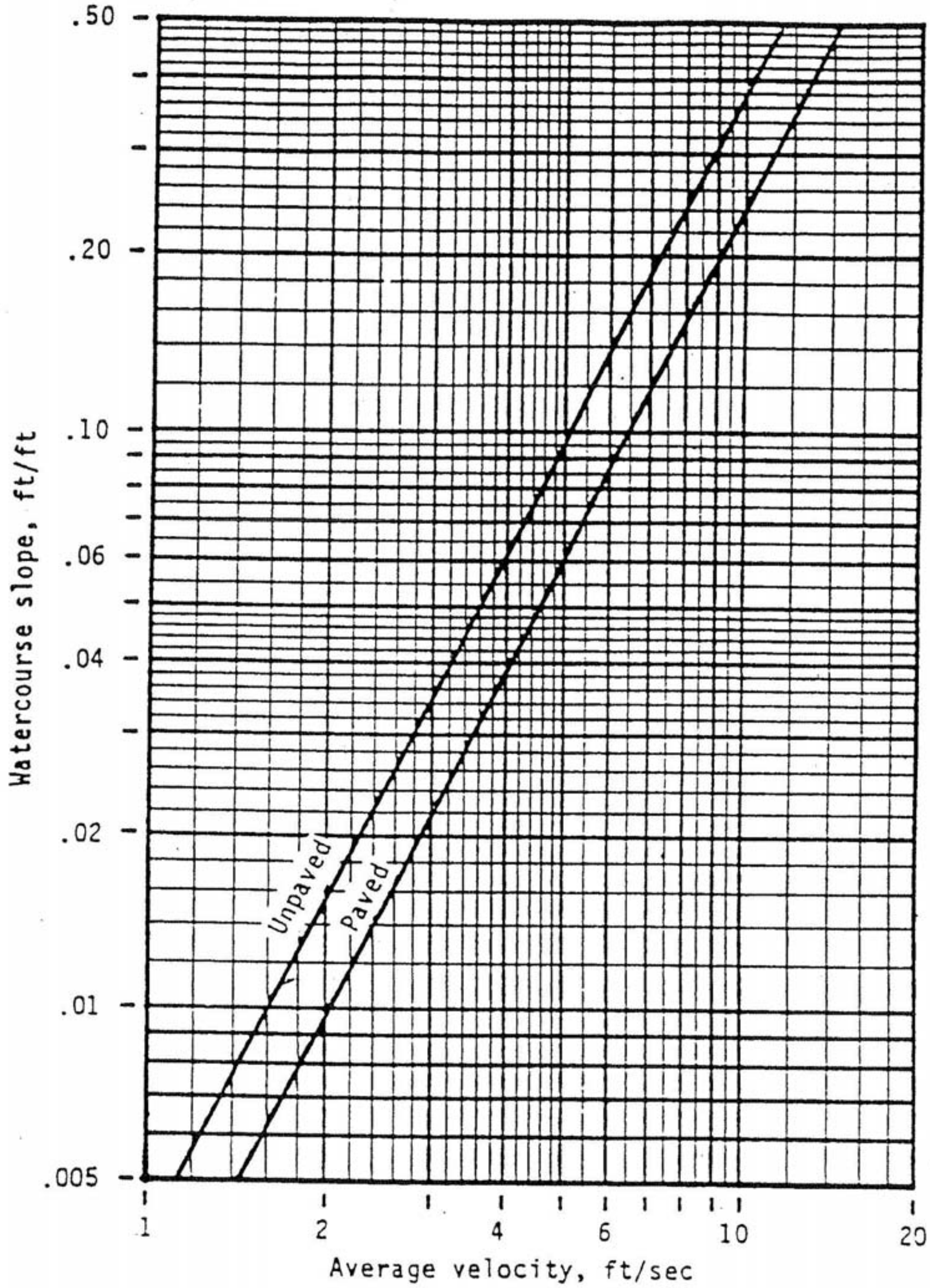


Source: USDA, SCS, TR55(1986)

Figure 3-3

Average Velocities - Shallow Concentrated Flow

Reservoirs And Lakes 3.8.6.6

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 3.8.6.7

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

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

3.9 Simplified SCS Method

Overview 3.9.1

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 3.9.2

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:

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

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 3.9.3

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	3.56 inches
5-year	4.47 inches
10-year	5.20 inches
25-year	6.22 inches
50 year	7.04 inches
100-year	7.91 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 3-6 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 3-4. If the ration I_a/P lies outside the range shown in Figure 3-4, 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.

Limitations 3.9.4

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.

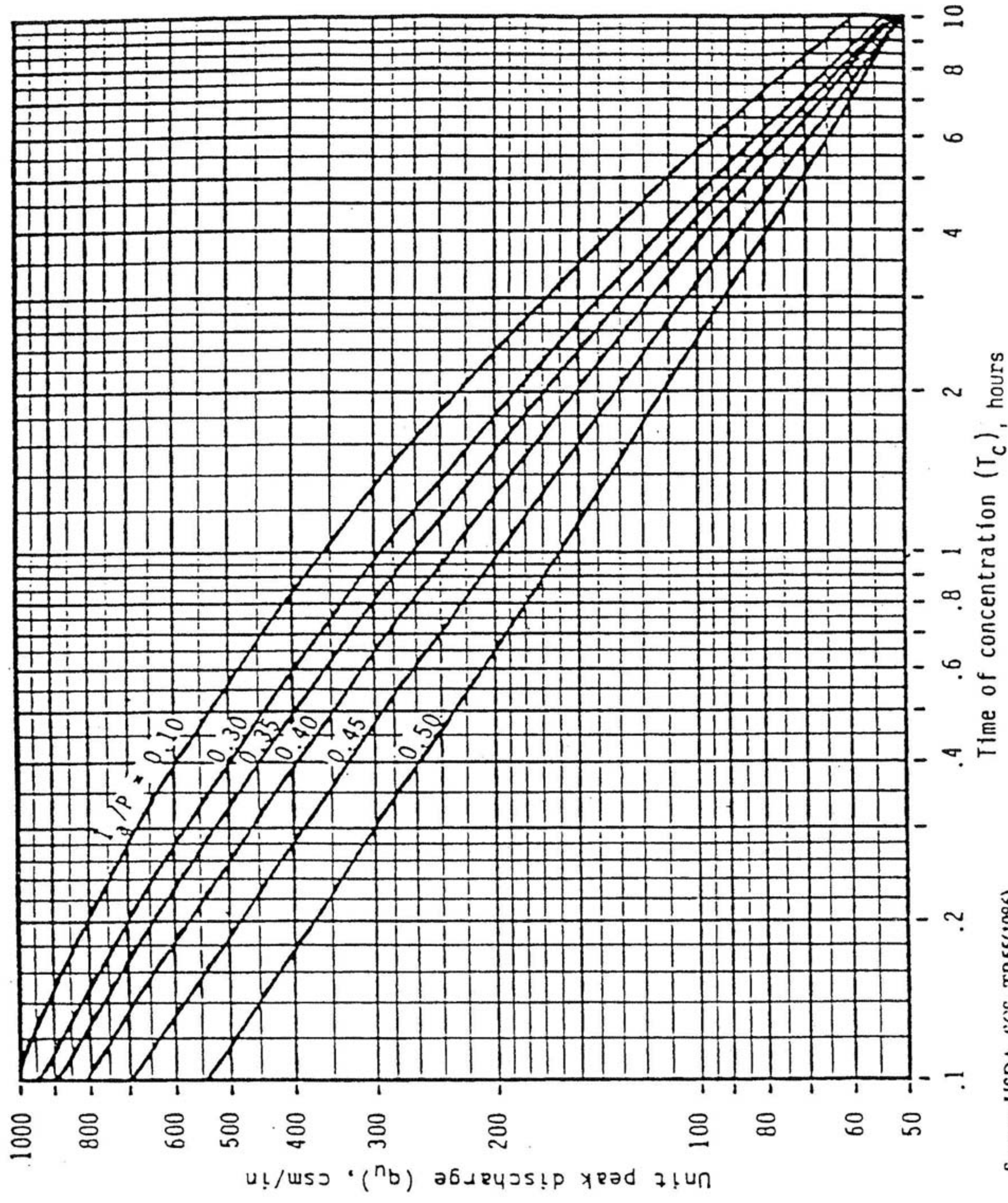
4. The pond and swamp adjustment factor, F_p, applies only to areas located away from the main flow path.

5. Accuracy is reduced if the ratio I_a/P is outside the range given in Figure 3-4.

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 3-4

Table 3-8

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 3.9.5

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 6.22 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 3-4 and equation 3.15

V = 2.6 ft/sec (from Figure 3-4 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 3-4 = 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 3.9.6

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 3.9.7

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 3.9.8

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

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Appendix A – Impervious Area Calculations

Urban Modifications A.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, consider whether the impervious areas connect directly to the drainage system, or to lawns or other pervious areas where infiltration can occur.

The curve number values given in Tables 3-7 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 3-7 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 3-8.

If all the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 3-8 are not applicable, use Figure A-1 to compute a composite CN. For example, Table 3-8 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 A-1 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Composite CN With Connected Impervious Area

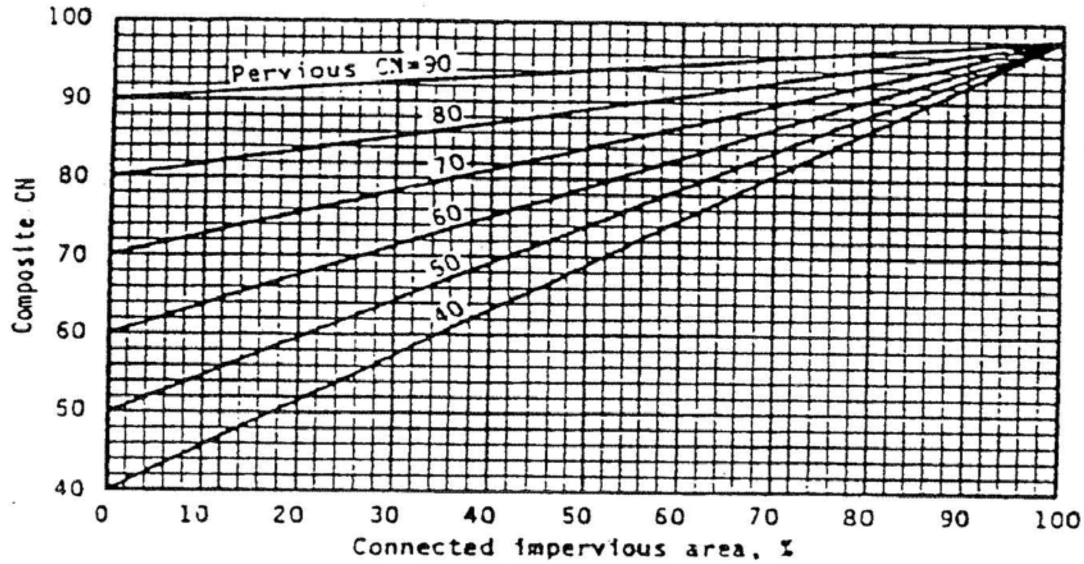
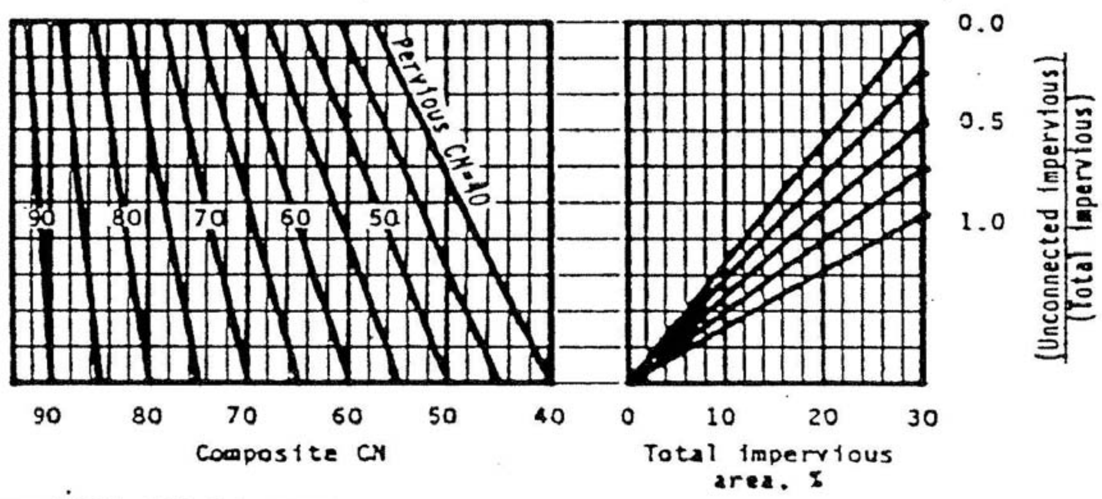


FIGURE A-1

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



Source: USDA, SCS, TR55(1986)

FIGURE A-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 A-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 A-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 A-2 is 66. If all of the impervious area is connected, the resulting CN (from Figure A-1) would be 68.

Composite Curve Numbers A.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 but sees the drainage area as a uniform land use represented by the composite curve number.

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

Table A-1 Composite Curve Numbers Calculations

(1) Acreage	(2) Land Use	(3) Soil Type	(4) Hydrologic Group	(5) CN	(6) Weighted CN Acreage/Total Area) x (CN)

The composite curve number for the total drainage area is then the sum of the composite curve numbers from column 6.

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.

Table A-1

Composite Curve Numbers

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

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, consolidates 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 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 mitigates 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 head cutting, 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
Δ d	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 for no more than 5 feet vertical.

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 24 hour design storm. The 100-year 24 hour 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.

3. 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.

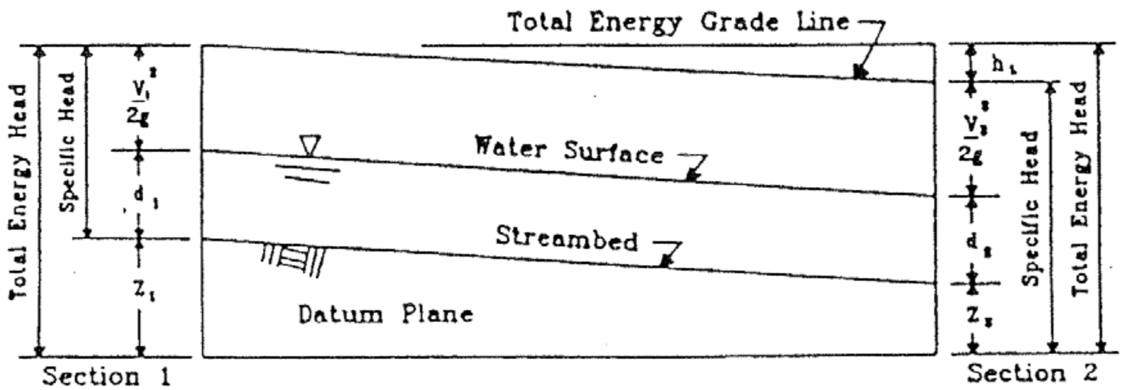


Figure 4-1 Energy In Open Channel Flow

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

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/s²)
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:

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

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.

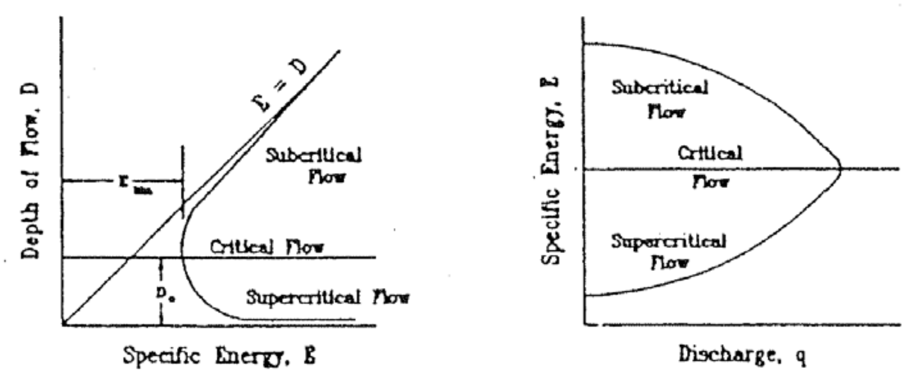


Figure 4-2 Definition Sketch of Specific Energy

Subcritical Flow 4.4.6

When the Froude Number is smaller than 1, the state of flow is defined as subcritical 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)

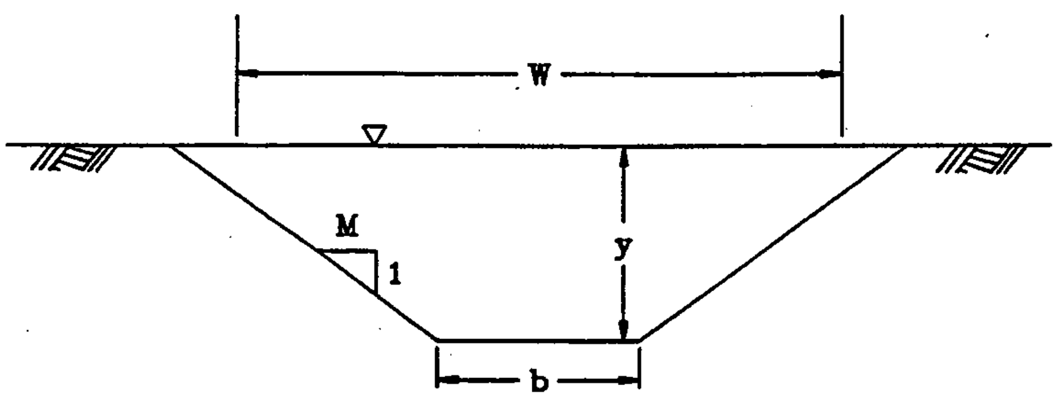


Figure 4-3 - Trapezoidal Channel - Definition of Variables

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:

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

Where:

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

The associated bottom width is:

$$b = ky \tag{4.6}$$

The cross-sectional area of the resulting channel is:

$$A = by + My^2 \tag{4.7}$$

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:

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

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

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

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.

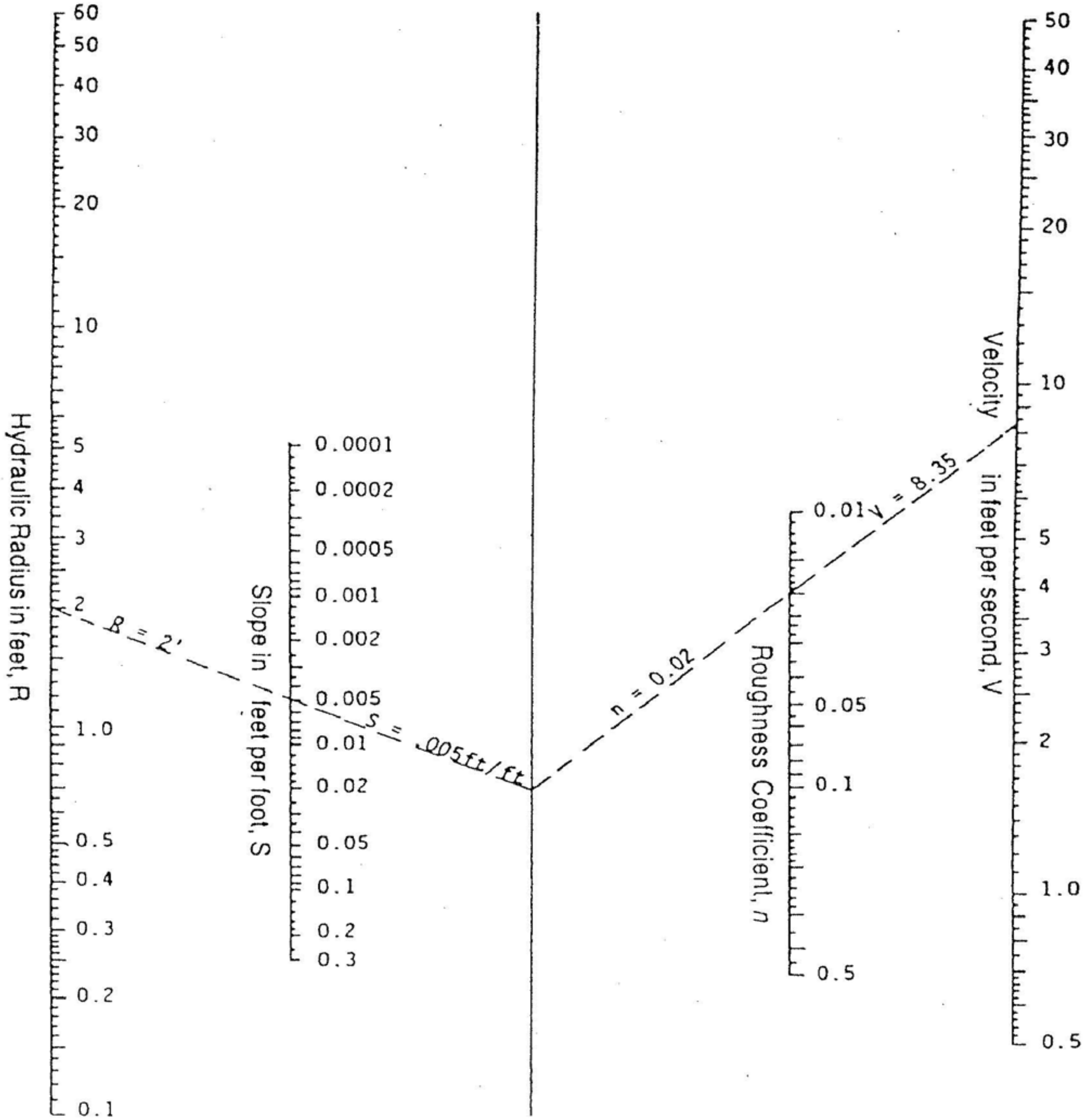


Figure 4-4
Nomograph for the Solution of Manning's Equation

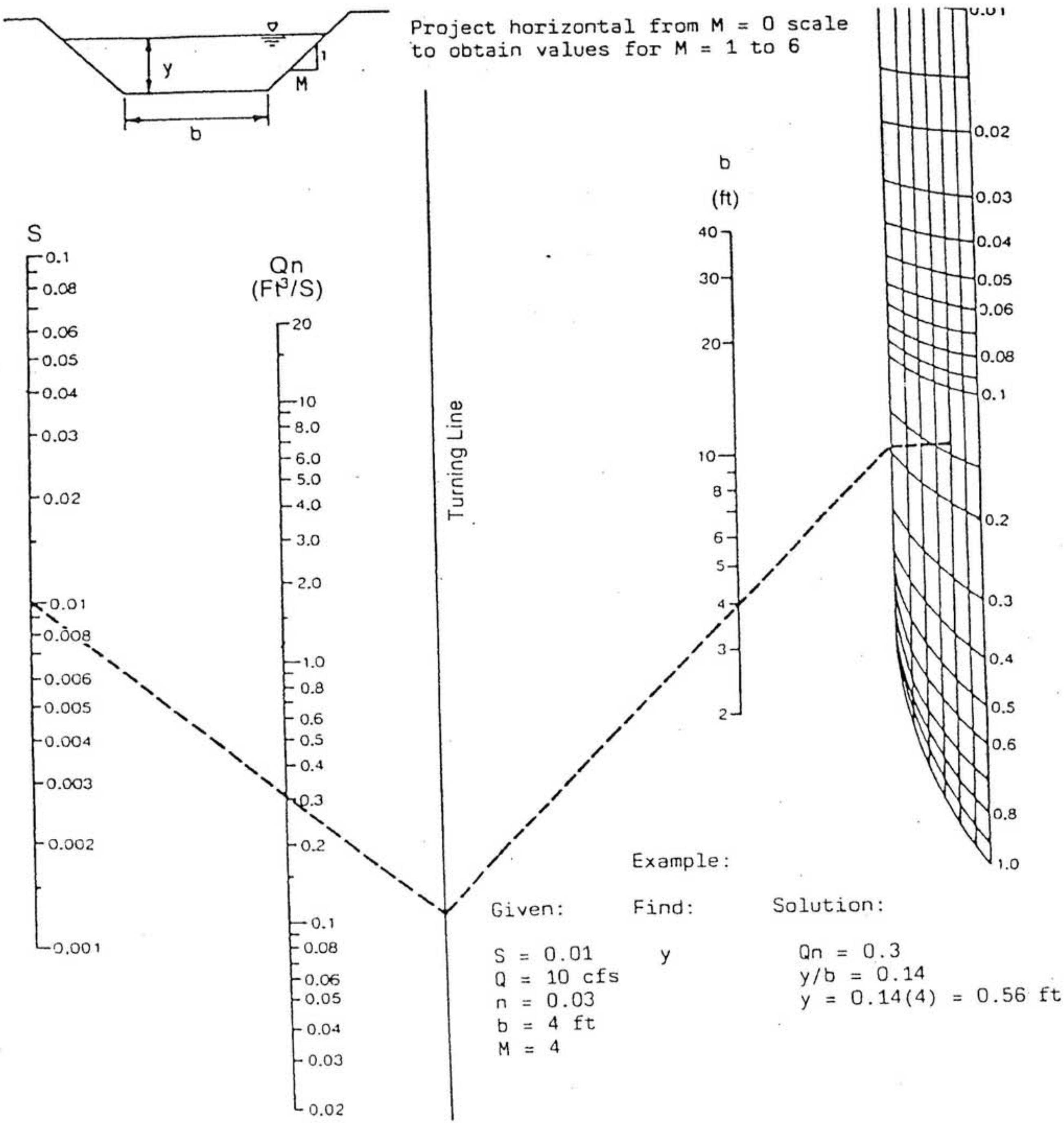


Figure 4-5

Solution of Manning's Equation for Trapezoidal Channels

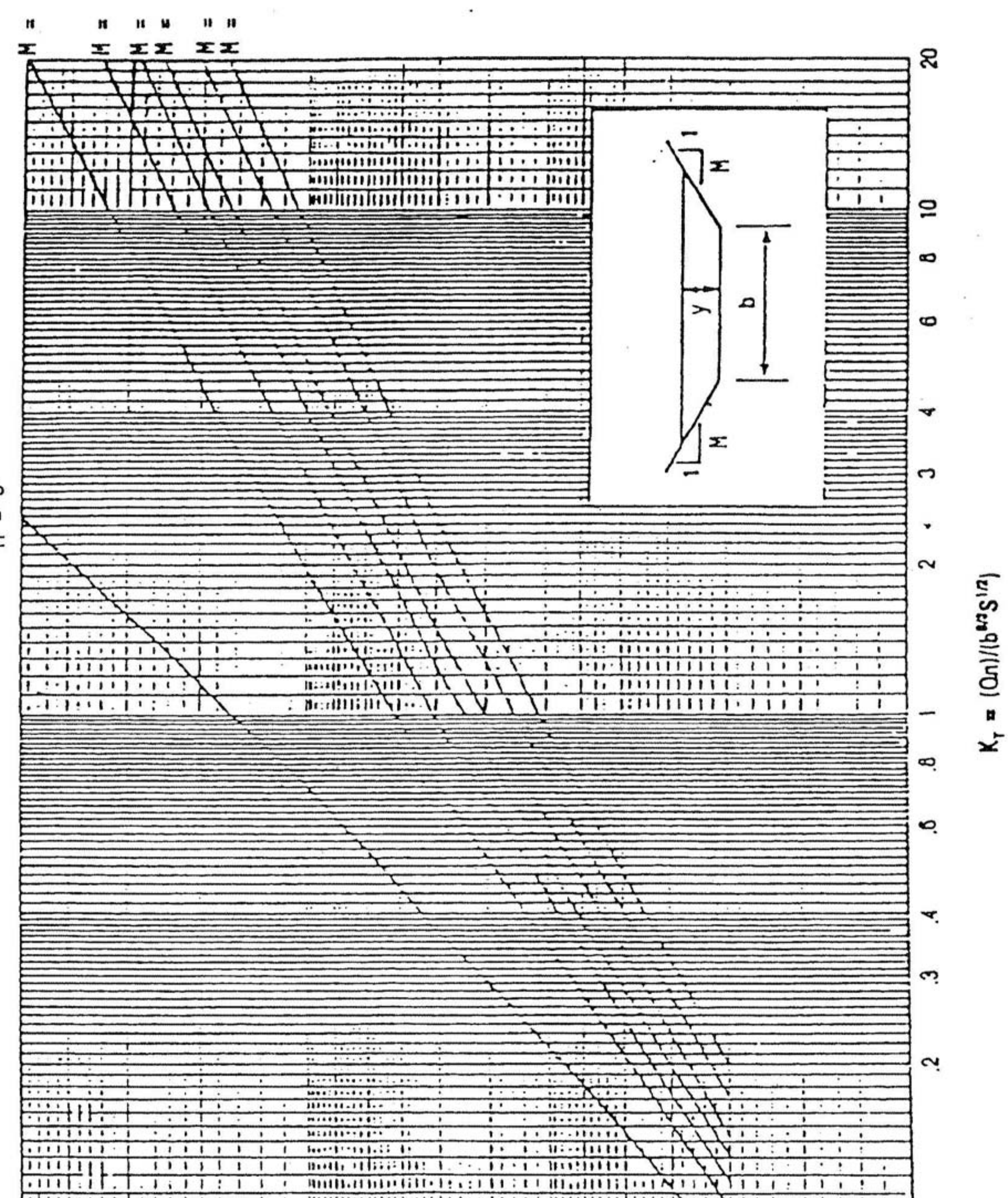


Figure 4-6

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:

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

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:

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

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.

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

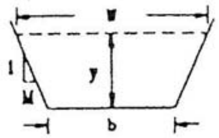
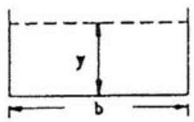
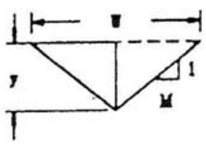
[a]See Figure 4-7 for channel sketches

[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:

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

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:

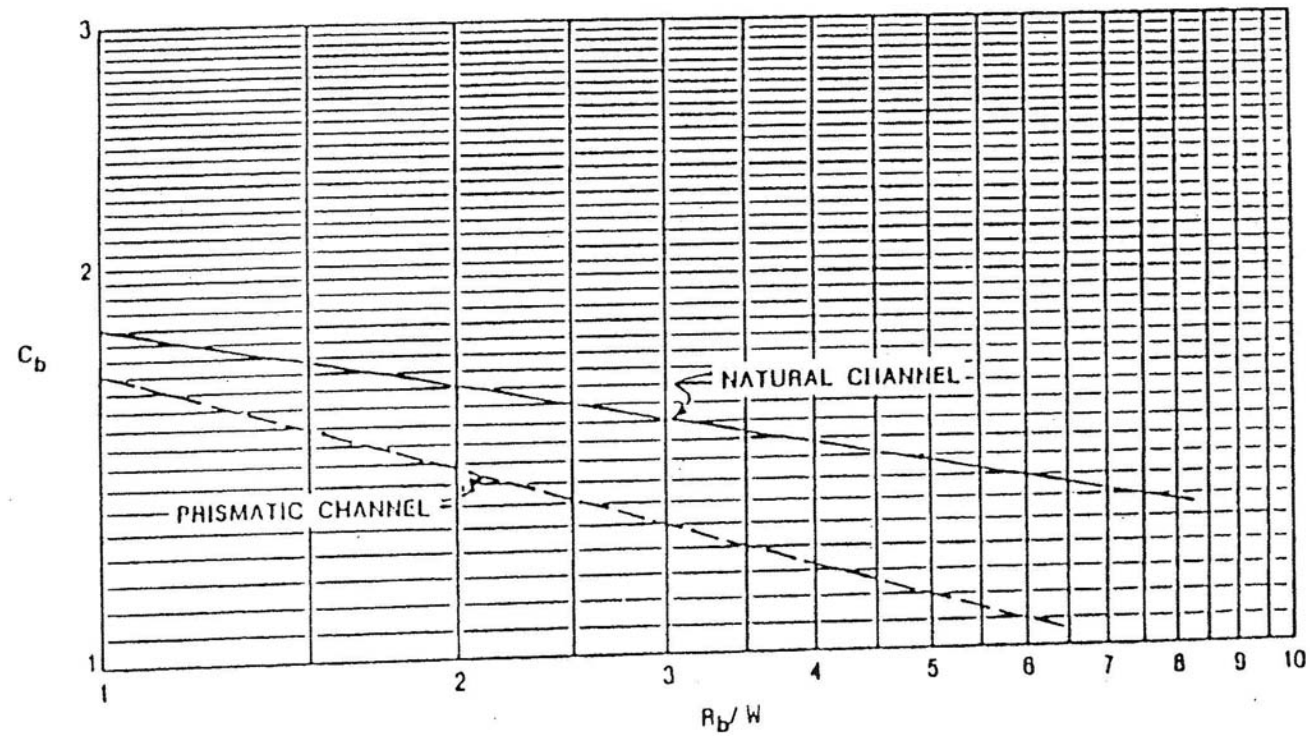
$$\mathbf{n = 0.0395\ (d_{50})^{1/6}} \qquad \textbf{(4.17)}$$

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:

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



To obtain effective velocity, multiply known velocity by C .

W = Channel Top Width

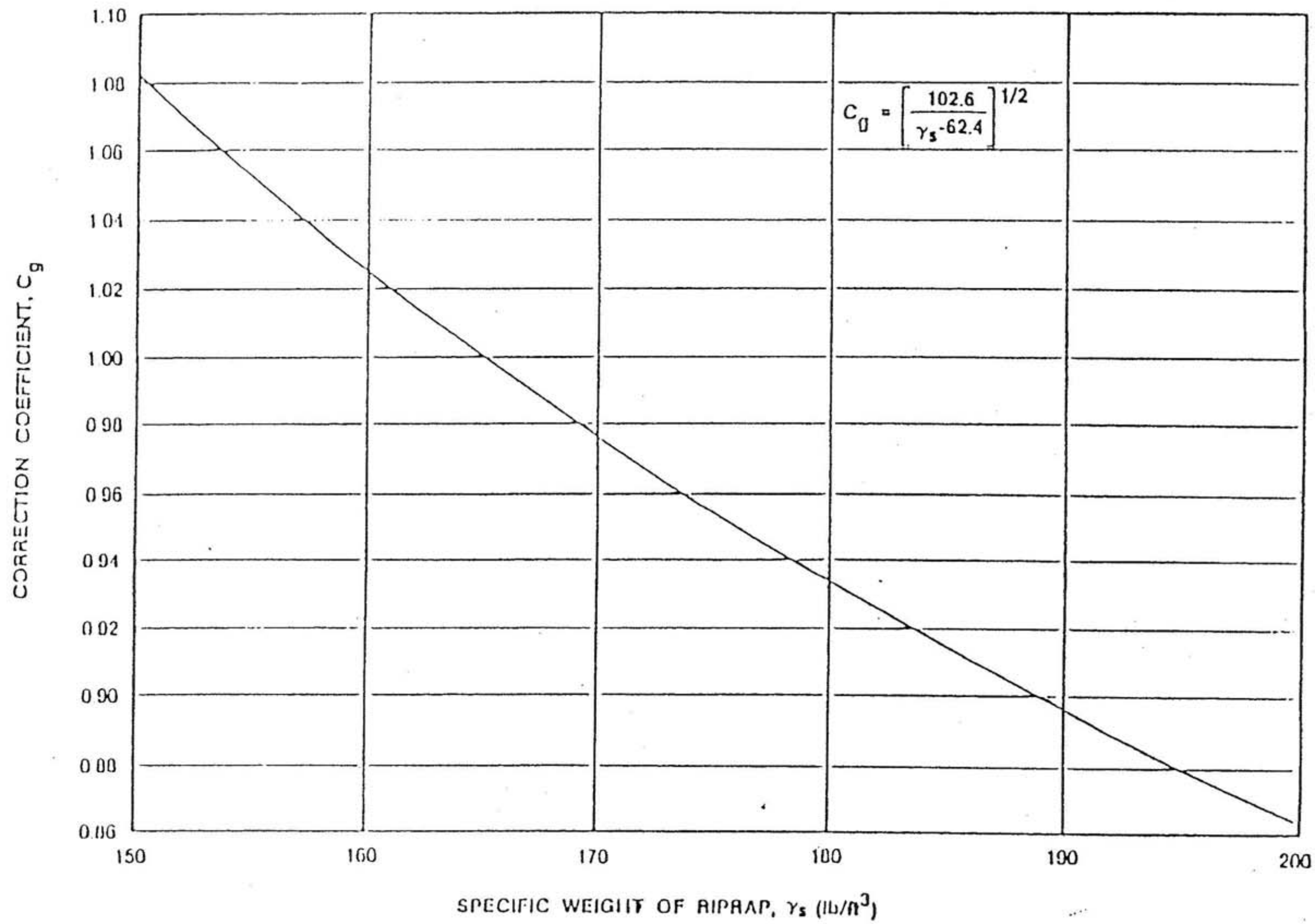
R_b= Centerline Bend Radius

C_b= Correction Coefficient

Reference: 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 (continued)

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)

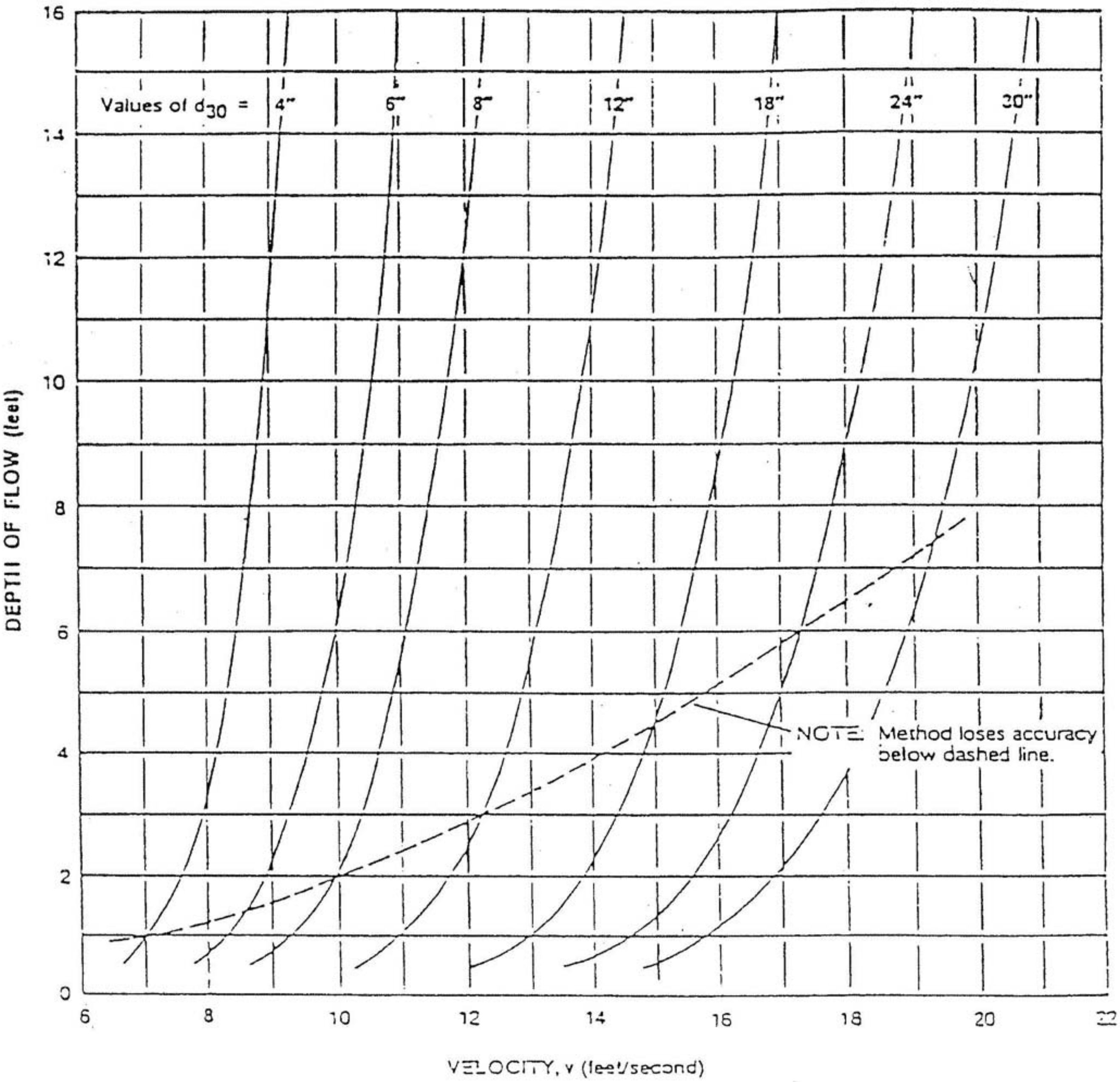
5. Determine available riprap gradations. A well graded riprap is preferable to uniform size or gap graded. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50}, size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines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

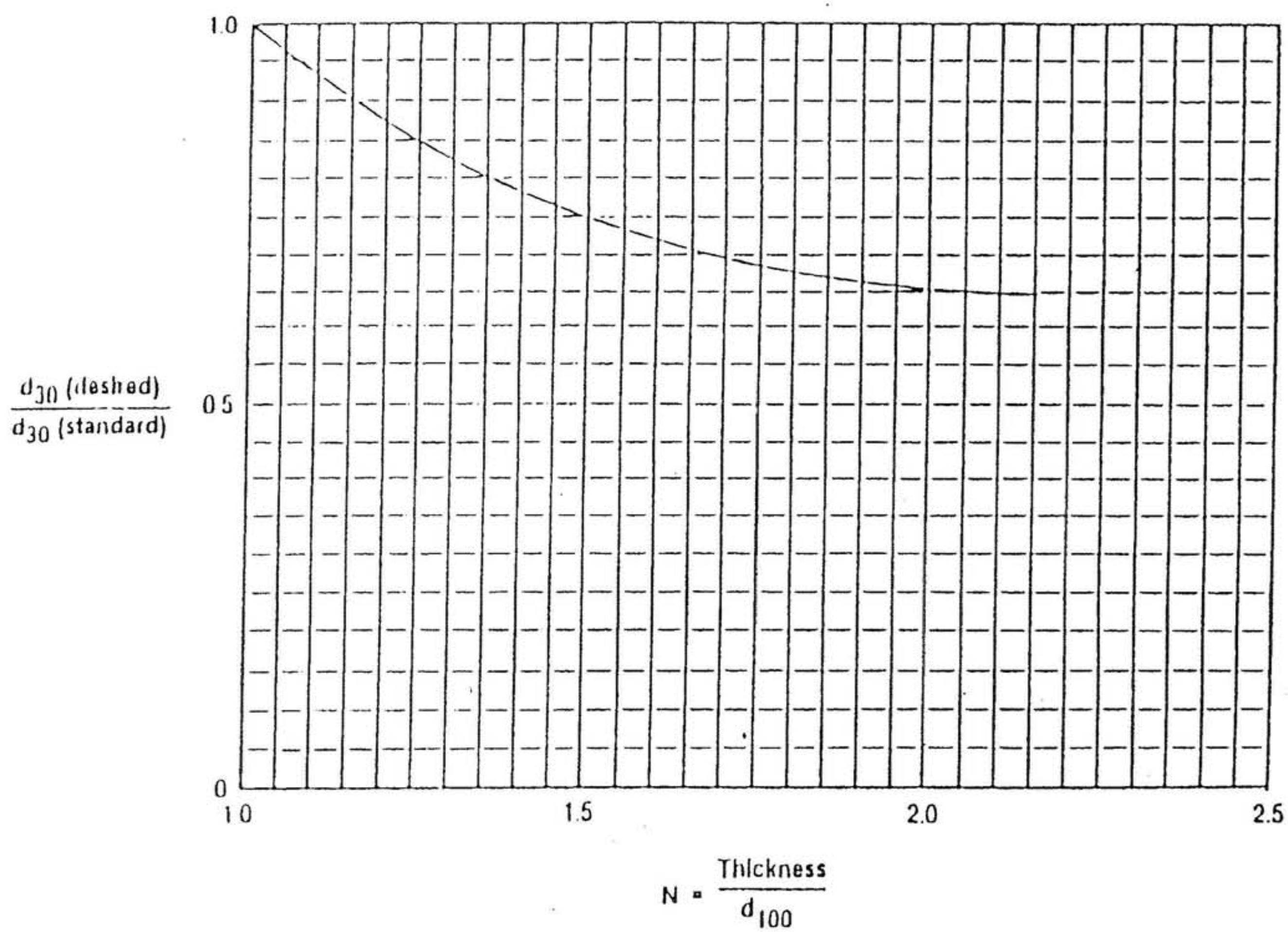
7. Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.



Reference: Reese (1983).

Figure 4–10

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



Reference: Maynord (1987)

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 non-uniform 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 = ________ α = ________ 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, Δ E, 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, Δ E, 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:

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

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:

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

Where: Δ x = length of channel between consecutive depths of flow (ft)

Δ E = 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) $h_f = \Delta x\,\bar{S}_f$

(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:

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

Where: h_e = eddy head loss (ft)

k_e = eddy head loss coefficient (ft) (for prismatic and regular channels, k_e = 0; for gradually converging 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 $by+My^2$ (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)

DIRECT STEP METHOD
Table 4-7

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)

STANDARD STEP METHOD
Table 4-8

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, Δ x, 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. This 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-22 (USDOT, FHWA, September 2009*, Revised August 2013).* 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 sag. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sag" condition exists when the inlet is located at a low point and water enters from both directions.

Criteria
5.1.3

The following criteria shall be used for drainage system design.

Design Frequencies

	Roadway Ditch/Pipe	Cross Drain	Gutter
Thoroughfares	10 year 24 hour storm	50 year 24 hour storm	4 inches/hour
Minor Roadways	10 year 24 hour storm	25 year 24 hour 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 based on a rainfall intensity of 4 inches per hour.
- For a street with a standard 2 feet 6 inch curb and gutter, an additional 2 feet is allowed with a total maximum spread of 8 feet from the face of the curb based on a rainfall intensity of 4 inches per hour.
- See Section 3.5.1 for additional criteria on design storm frequencies.

Other Design Criteria:

- In a sag condition (a vertical sag with no available overflow) inlet capacity and storm drains must be designed for the 25-year 24-hour 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 24-hour storm.
- No concentrated runoff flowing over Town sidewalks except at driveways.
- For local roads the pipes system hydraulic grade line shall not surcharge sag inlets in the 25-year 24-hour storm. For thoroughfare roads the pipe system hydraulic grade line shall not surcharge in the 50-year 24-hour 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 Definitions

	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	A drainage inlet composed of a curb-opening and a grate.
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 Engineering Design and Construction Standards Procedures Manual.

Inlet Types
and Spacing
5.4.3

Inlet types shall be selected from the Engineering Design and Construction Standards Procedures 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 Engineering Design and Construction Standards Procedures Manual.

Curb and
5.4.6

Gutter Curb and gutter installation shall be designed in accordance with the Engineering Design and Construction Standards Procedures 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 24-hour 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 5ft/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 many cases, deck drainage must be carried several spans to the bridge end for disposal. Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 1 percent. When bridges are placed at a vertical curve and the longitudinal slope is less than 1 percent, the gutter spread should be checked to ensure a safe, reasonable design.

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

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

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 inlet.

   Q = CiA

   Where: C = runoff coefficient for the sub-drainage area
   i = 4.0 inches/hour
   A = Area determined in #2 (acres)

4. Surface 'Q' Total – $Q_{total} = Q_{sub} + \sum Q_{bypass}$

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.

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.

Procedure (continued)

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

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

11. Bypass to Inlet # = List Inlet # directly downstream (bypass destination).

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 – S_T (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:

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

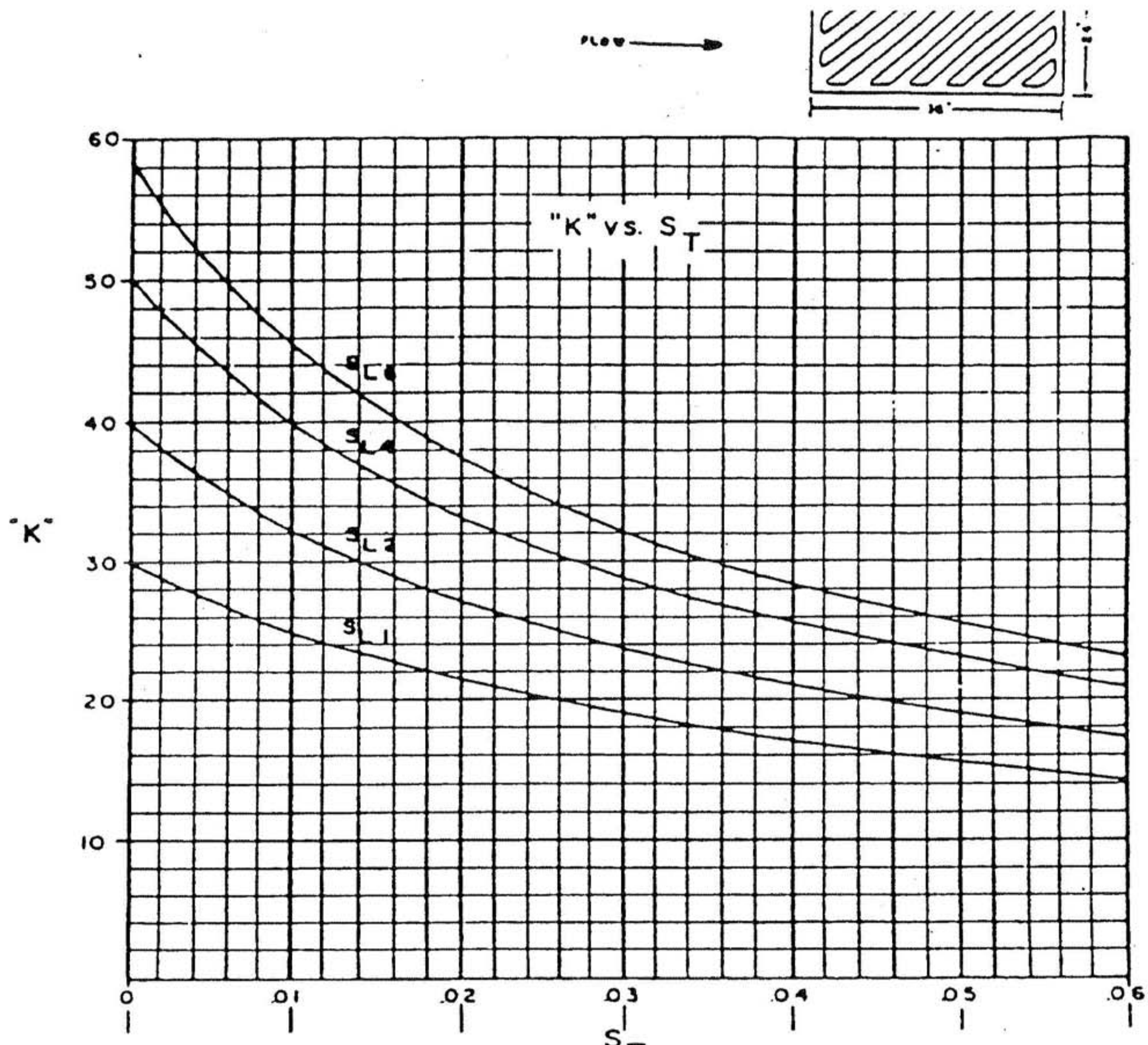
Since $\mathbf{D = T(S_L)}$ **(5.3)**

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

T can be derived:

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

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 25-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 sag condition is subject to clogging, a curb opening is required as a supplemental inlet. The capacity of a grate in a sag depends upon the area of the openings and the depth of water at the grate. Figure 5-2 can be used to calculate the head or flow for Type ‘E’ grates in sag 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:

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

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)

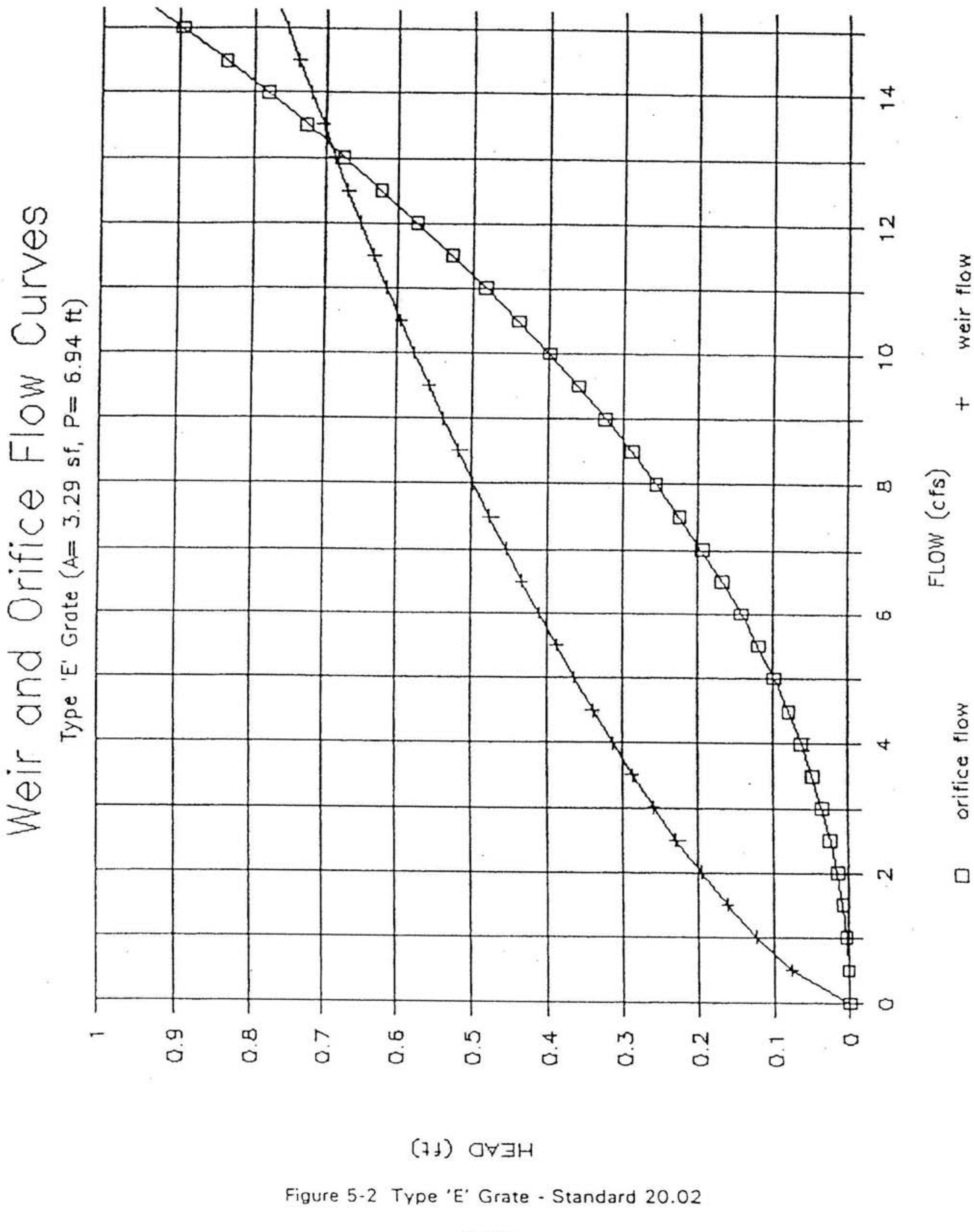


Figure 5-2 Type 'E' Grate - Standard 20.02

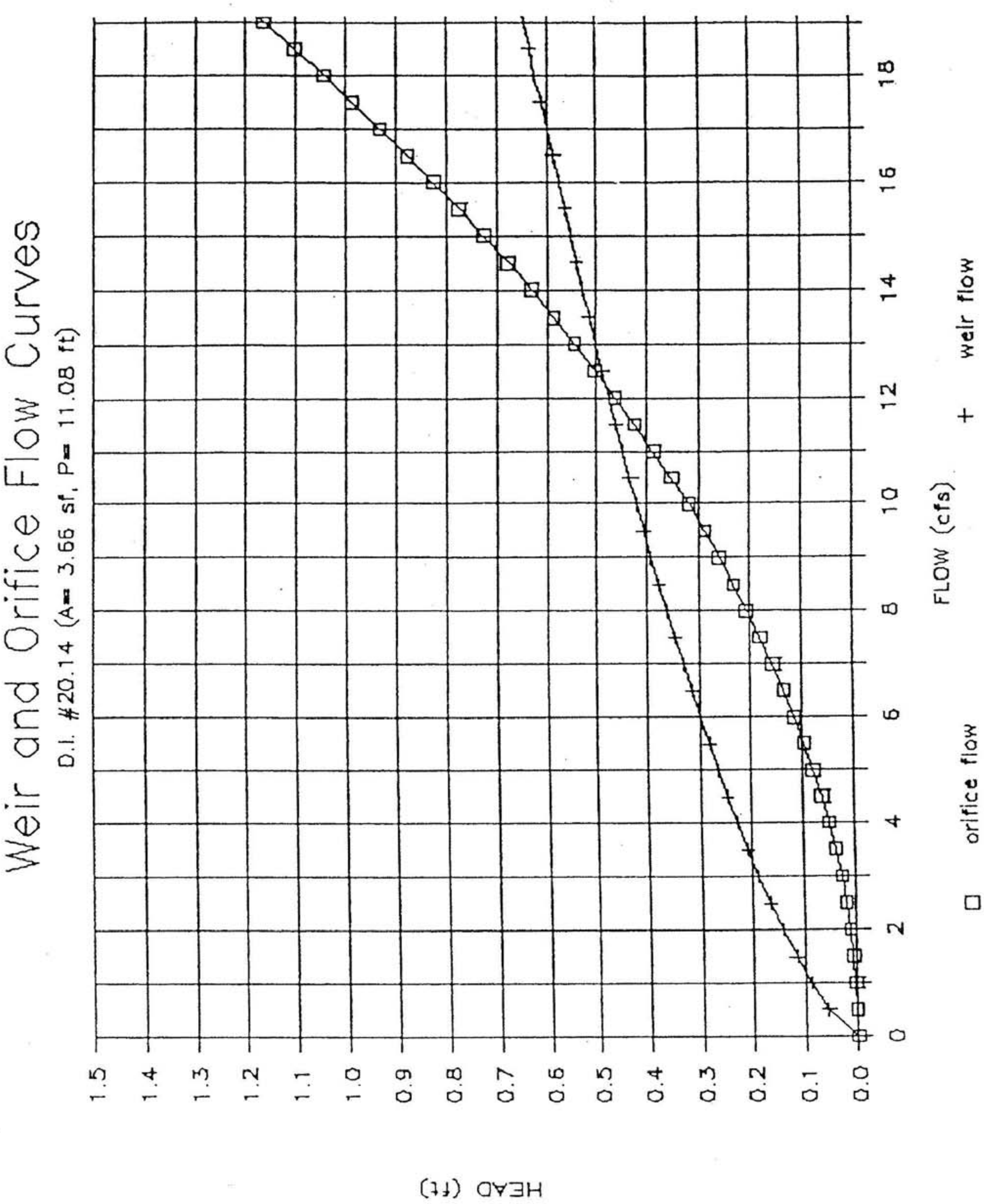


Figure 5-3 Standard Drop Inlet Grate - Standard 20.14

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)

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

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

(weir)

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

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

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

3. Check the spread

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

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

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

Check the spread

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

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.

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

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

$$H_f = S_f\ L \qquad (5.9)$$

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.

$$H = KV^2\ /2g \qquad (5.10)$$

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.

$$H_{tm} = V^2\ /2g \qquad (5.11)$$

$$H_e = KV^2\ /2g \qquad (5.12)$$

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.

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

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.
*Junction angles should not exceed 90 whenever possible

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:

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

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.

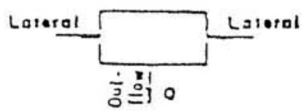
In Figure 5-4 on page 5-21 is a summary of energy losses which should be considered. Following this in Figure 5-5 is a sketch showing the proper and improper use of energy losses in developing a storm drain system.

SUMMARY OF ENERGY LOSSES

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

TERMINAL JUNCTION LOSSES
(at beginning of run)

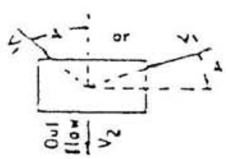
Where g = gravitational constant, 32.2 feet per second per second.



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

JUNCTION LOSSES

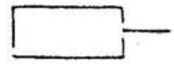
Use only where flows are identical to above, otherwise use H_{j2} Equation.



$$H_b = \frac{K V_1^2}{2g}$$

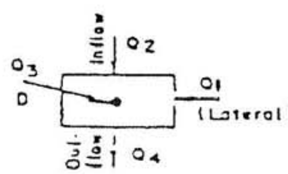
BEND LOSSES
(changes in direction of flow)

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



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

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



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

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

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

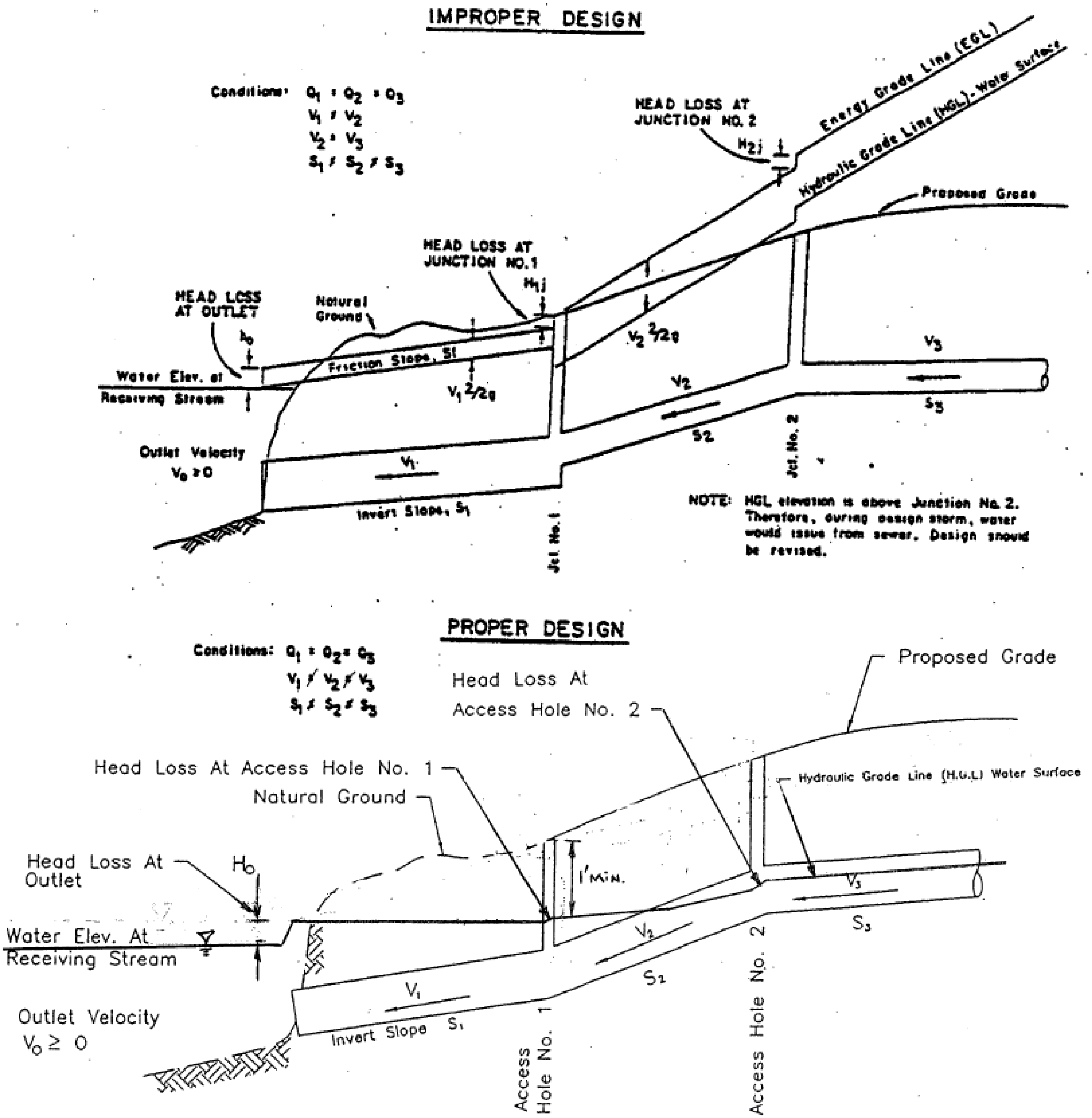


Figure 5-5 Energy and Hydraulic Grade Lines for Storm Sewer Under Constant Discharge

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:

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

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:

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

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

For pipes flowing full, the above equations become:

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

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

Where: D = diameter of pipe (ft)

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

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

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

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.

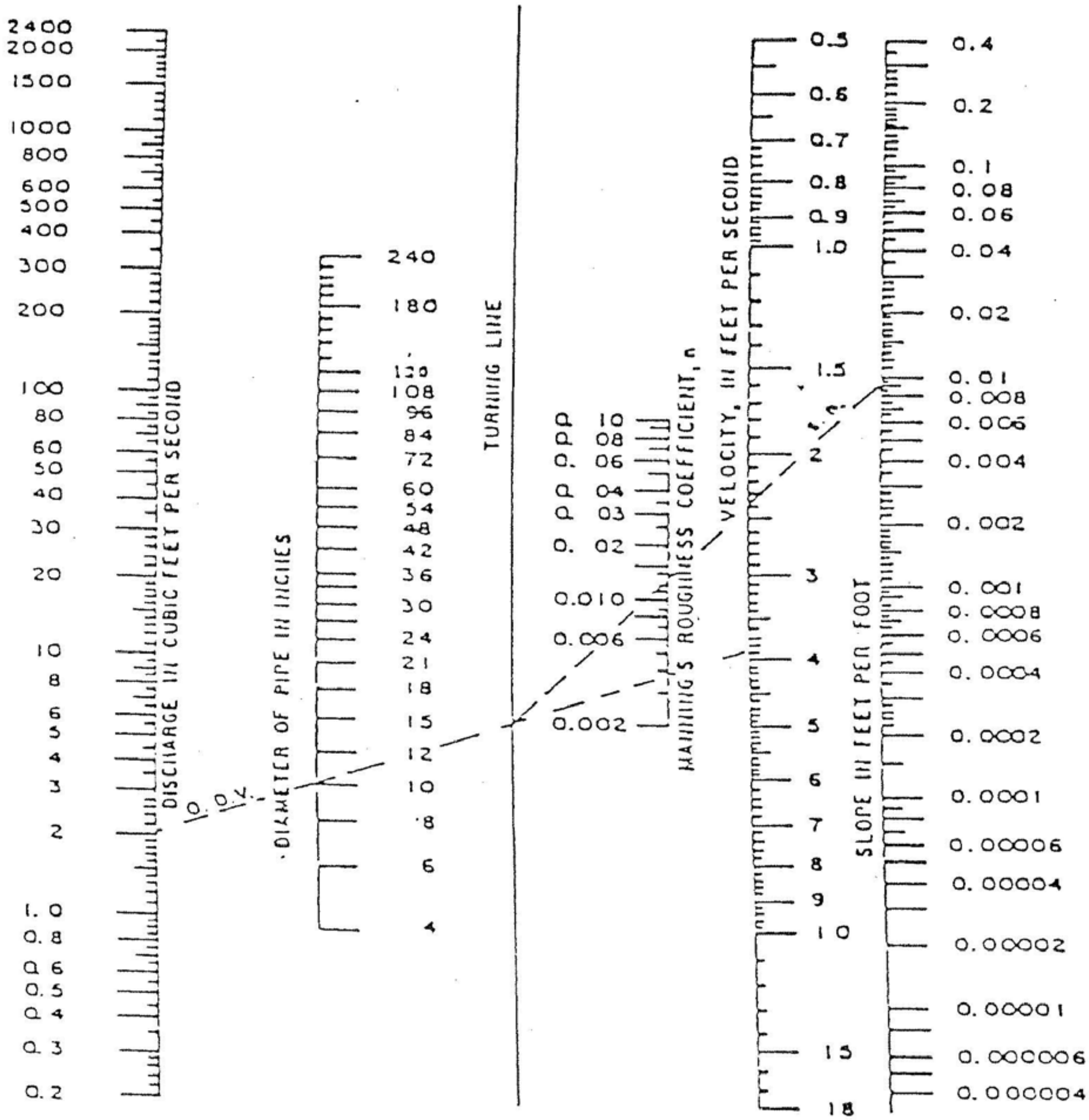


Figure 5-6 Nomograph for Solution of Manning's Formula for Flow in Storm Sewers

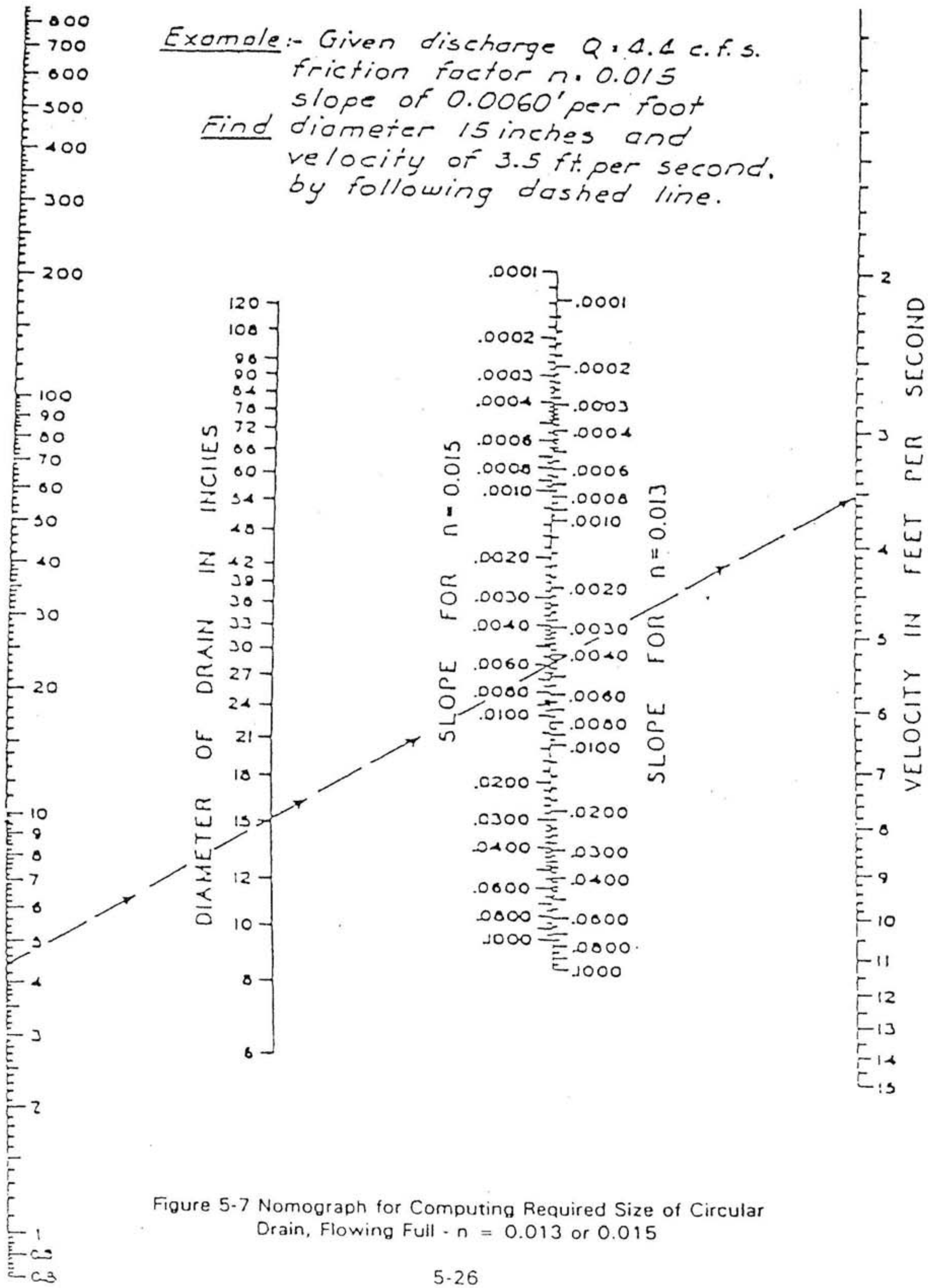


Figure 5-7 Nomograph for Computing Required Size of Circular Drain, Flowing Full - n = 0.013 or 0.015

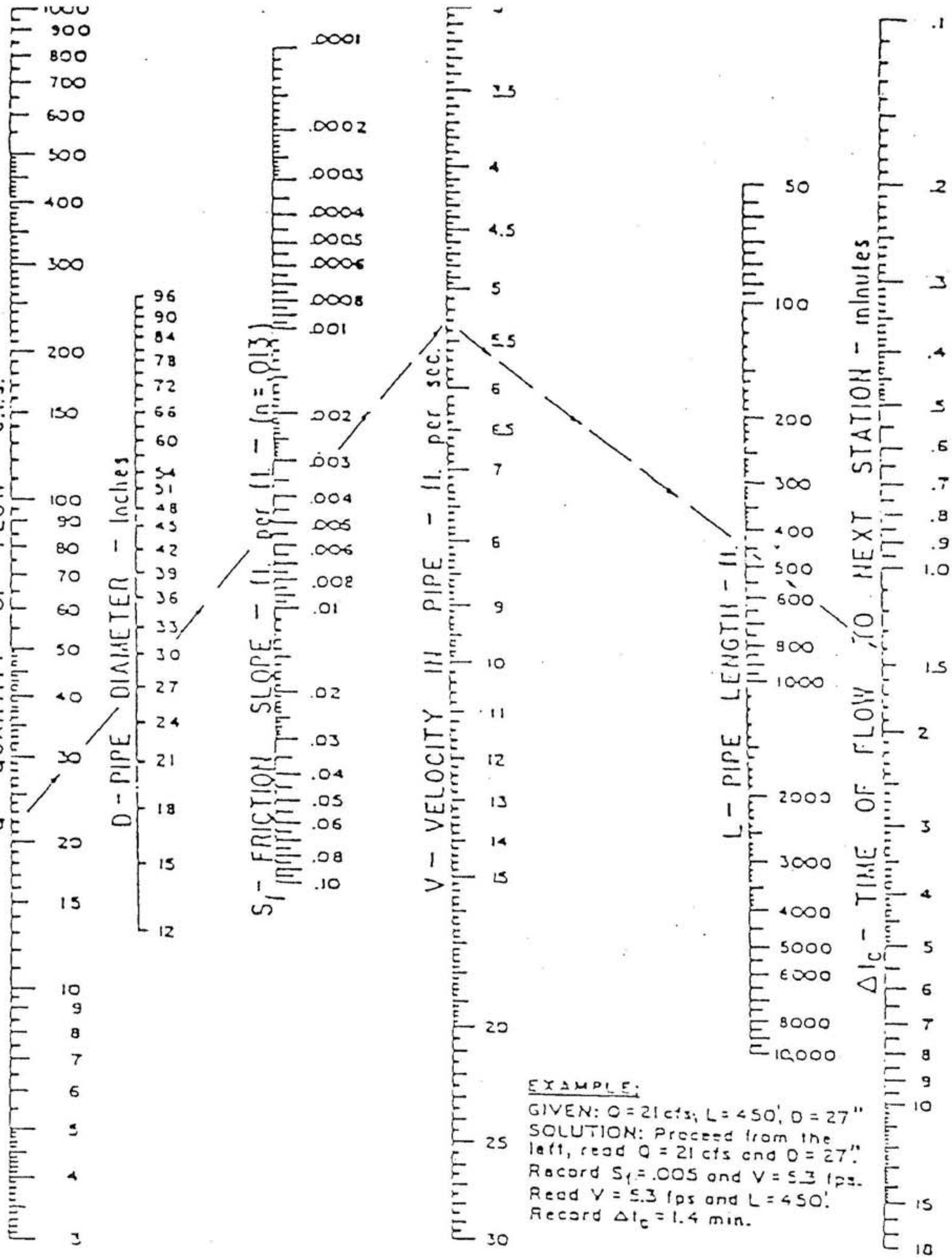
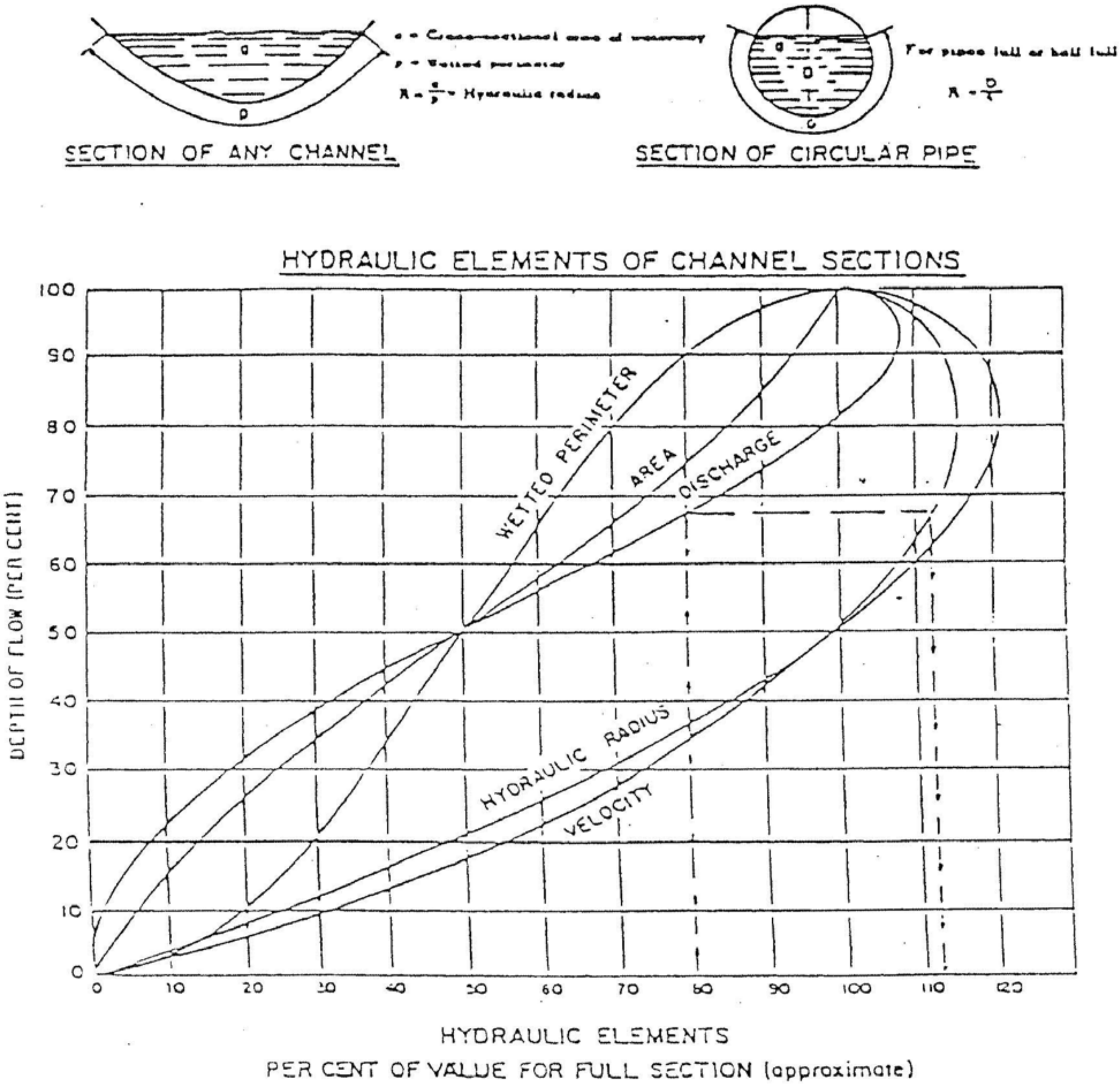


Figure 5-8 Concrete Pipe Flow Nomograph



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:

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

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:

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

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:

$$\mathbf{H\Delta = KV_j^2/2g} \qquad \textbf{(5.24)}$$

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:

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

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 V_o	H_o	Q_i	V_i	$Q_i V_i$	$\frac{V_i^2}{2g}$	H_i	Angle	H_Δ	H_t	$1.3\ H_t$	$0.5\ H_t$	Final H	Inlet Water Surface Elev	Rim Elev
1	2	3	4	5	6	7	8	9	10	11	12		13	14	15	16	17	18	19	20	21

Final $H = H_f + H_t$

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

90° K = 0.70
70° K = 0.64
60° K = 0.55

45° K = 0.47
30° K = 0.35
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

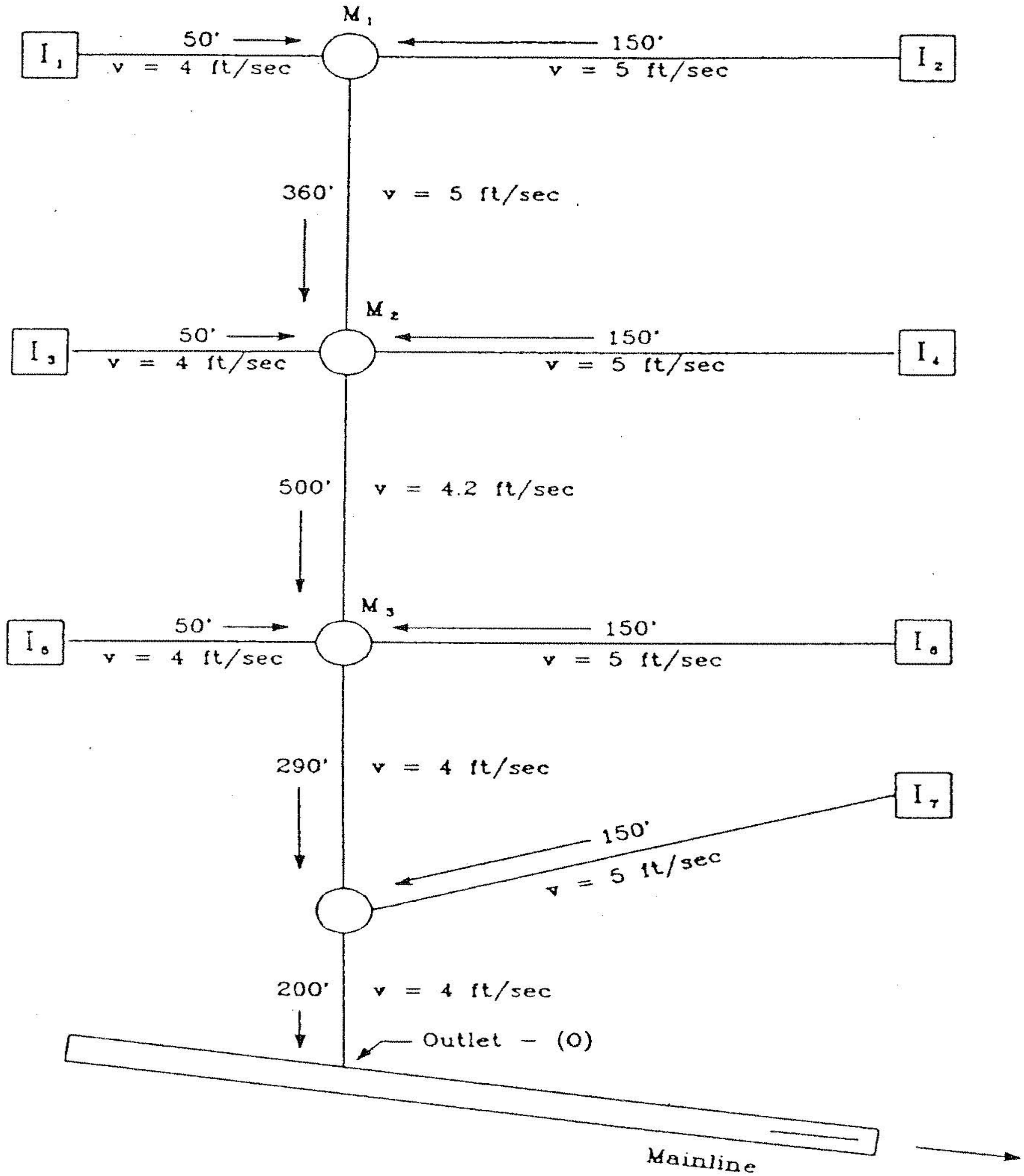


Figure 5-12 Hypothetical Storm Drain System Layout

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 has 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.

5.11 Flood Study

Flood Study

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 floodproofed 6.5.3.1, 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 the lowest floor is completed, 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

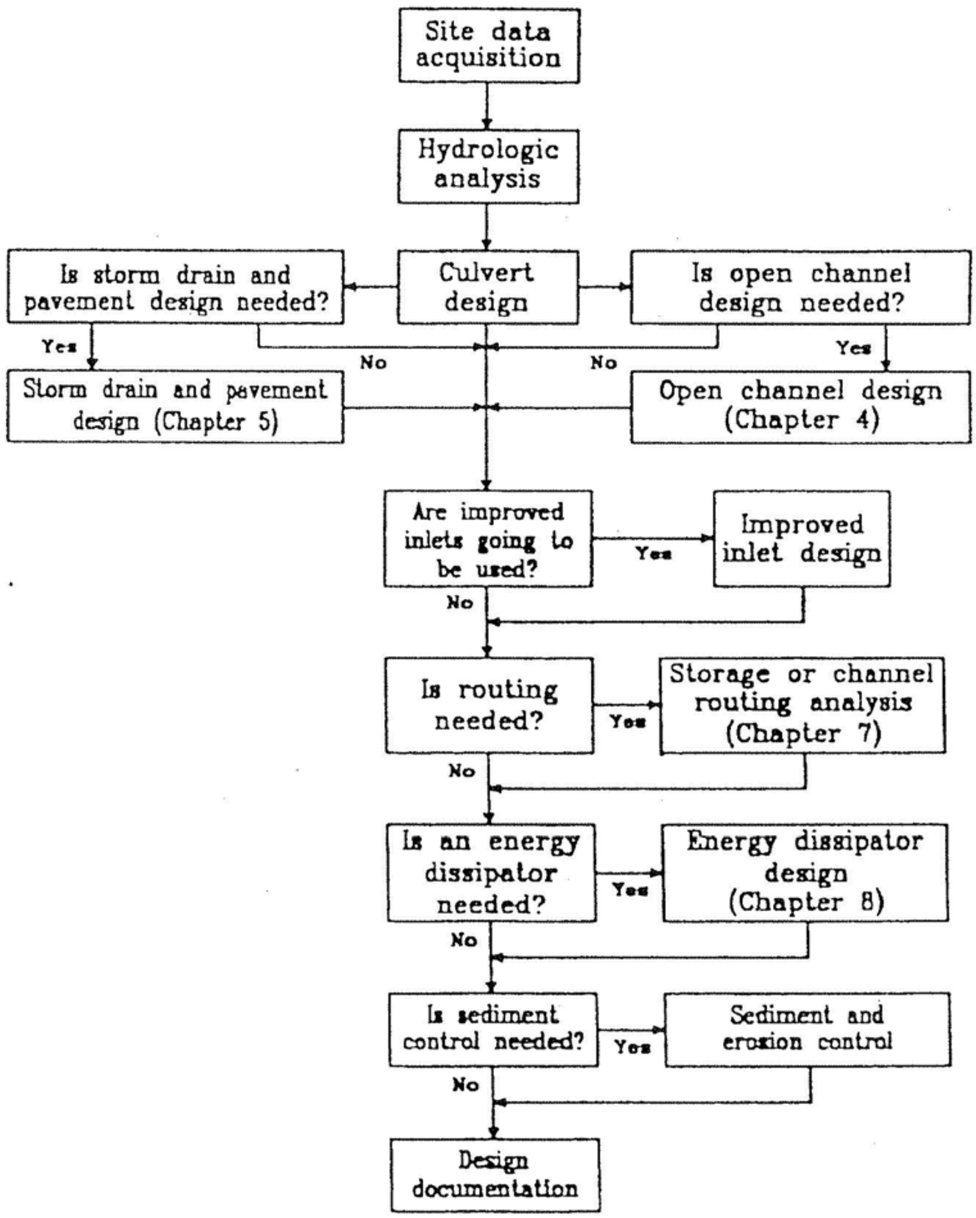
Symbol	Definition	Units
A	Area of cross section of flow	sq. ft.
B	Barrel width	ft
C_d	Overtopping discharge coefficient	--
D	Culvert diameter or barrel depth	in/ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration due to gravity	ft/s^2
H	Total energy loss	ft
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.

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. If 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 ≤ 1.2

* The headwater should be checked for the 100-year flood to ensure compliance with the locally adopted floodway ordinance and 100+1 criteria.

* The maximum acceptable outlet velocity should be identified. Either the headwater should be set to produce acceptable velocities, or stabilization or energy dissipation should be provided.

* 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)

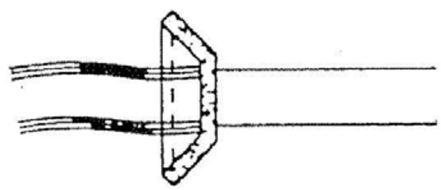
The relative efficiency of the inlet depends on the conduit. Headwalls are required for all metal culverts. The figure below illustrates the use of headwalls and wingwalls. Corrugated metal pipe in a headwall is essentially square-edged with an inlet coefficient of about 0.5.

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

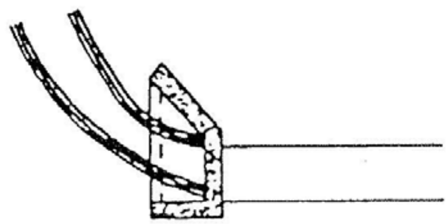
Wingwalls And Aprons 6.5.10

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.

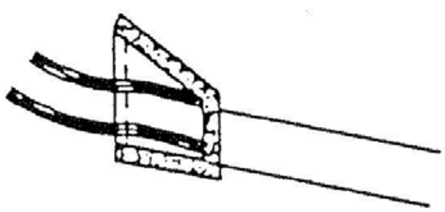
Little increase in hydraulic efficiency is realized with the use of wingwalls, regardless of the pipe material used and, therefore, the use should be justified for other reasons. Wingwalls can be used to increase hydraulic efficiency if designed as a side-tapered inlet (See Section 6.10.6.2 for more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



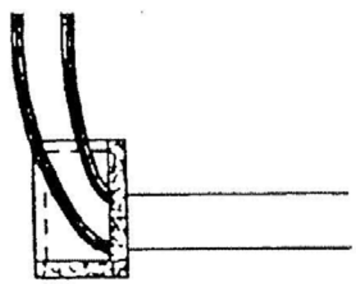
Flow Normal To Embankment – Preferred



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 corrugations	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

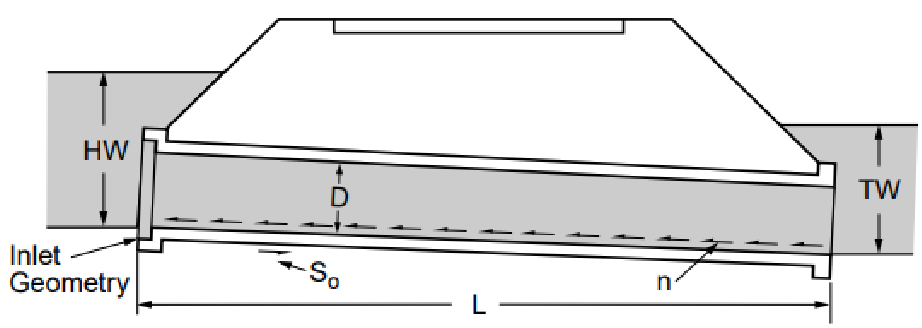
Inlet Control – If the culvert is operating on a steep slope, it is likely that the entrance geometry will control the headwater and the culvert will be on inlet control.

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, 2012. (Third Edition)

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

D = Inside diameter for circular pipe
HW = Headwater depth at culvert entrance
L = Length of culvert
n = Surface roughness of the pipe wall, usually expressed in terms of Manning's n
S_o = Slope of the culvert pipe
TW = Tailwater depth at culvert outlet



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

<u>Critical Depth – Outlet Control</u>- 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.

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

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

<u>Tailwater Depth – Outlet Control</u>- 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.

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

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

<u>Tailwater Depth > Barrel Depth</u> – 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)

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

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

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

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

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

Steep Slope 6.6.3.2

Tailwater Insignificant – Inlet Control- The entrance may be submerged or unsubmerged, the critical depth is greater than uniform depth at the design discharge ($d_c > d_u$), TW depth is less than SL (tailwater elevation is lower than the upstream flowline). Tailwater depth with respect to the diameter of the culvert is inconsequential as long as the above conditions are met. This condition is a common occurrence for culverts in rolling or hilly country. The control is critical depth at the entrance for HW values up to about 1.5D. Control is the entrance geometry for HW values over about 1.5D. HW is determined 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

Inlet or Outlet Control- For “slug” flow operation, the entrance may be submerged or unsubmerged, critical depth is greater than uniform depth at the design discharge ($d_c > d_u$), TW depth is greater than ($SL + d_c$) (TW elevation is above the critical depth at the entrance), and TW depth is less than SL + D (TW elevation is below the upstream crown). TW depth with respect to D alone is inconsequential as long as the above conditions are met. This condition is a common occurrence for culverts in rolling or 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

and the higher of the two determinations be used. Entrance control HW is (continued) 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 6.7.1

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

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 6.7.2

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 6.7.3

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.

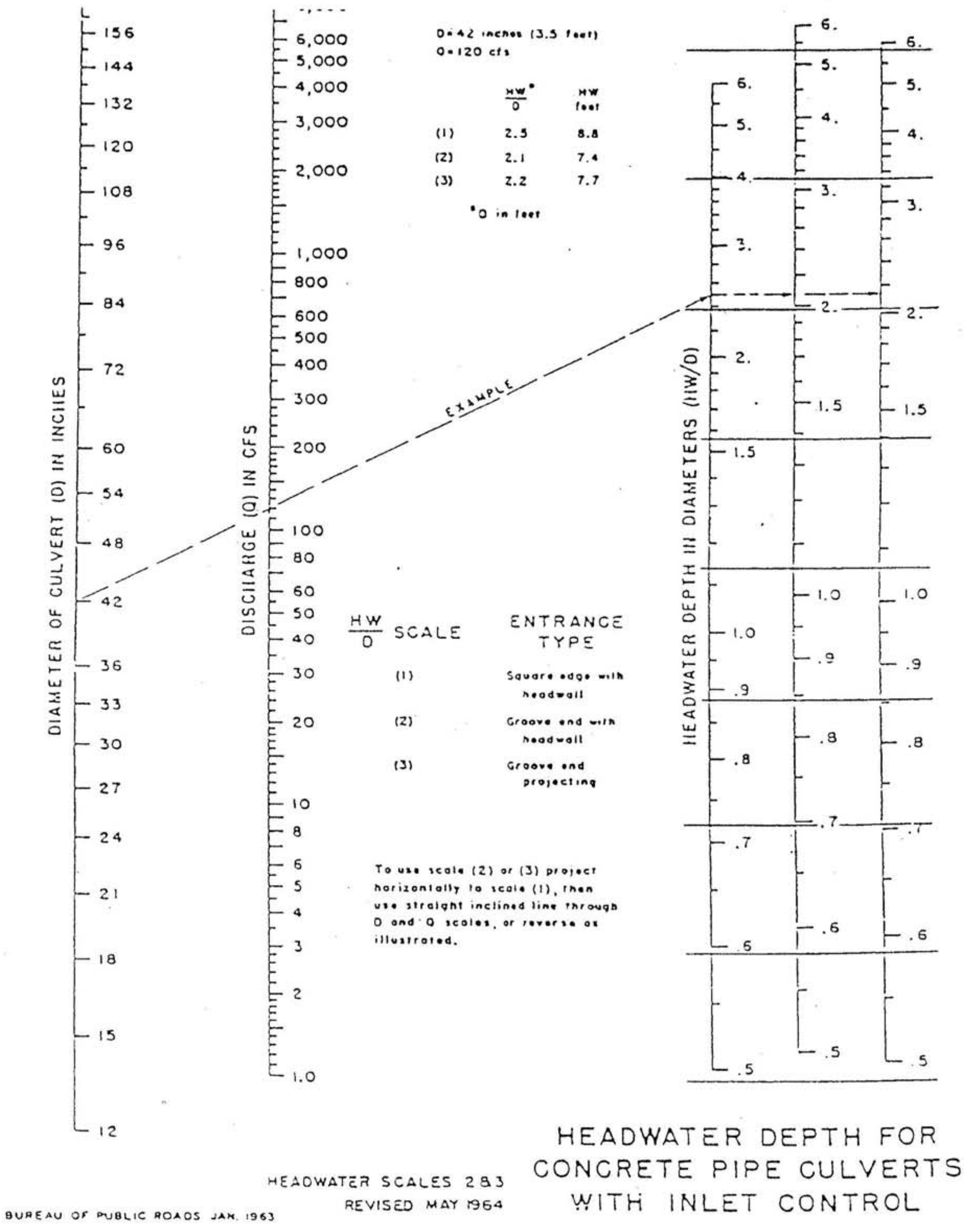
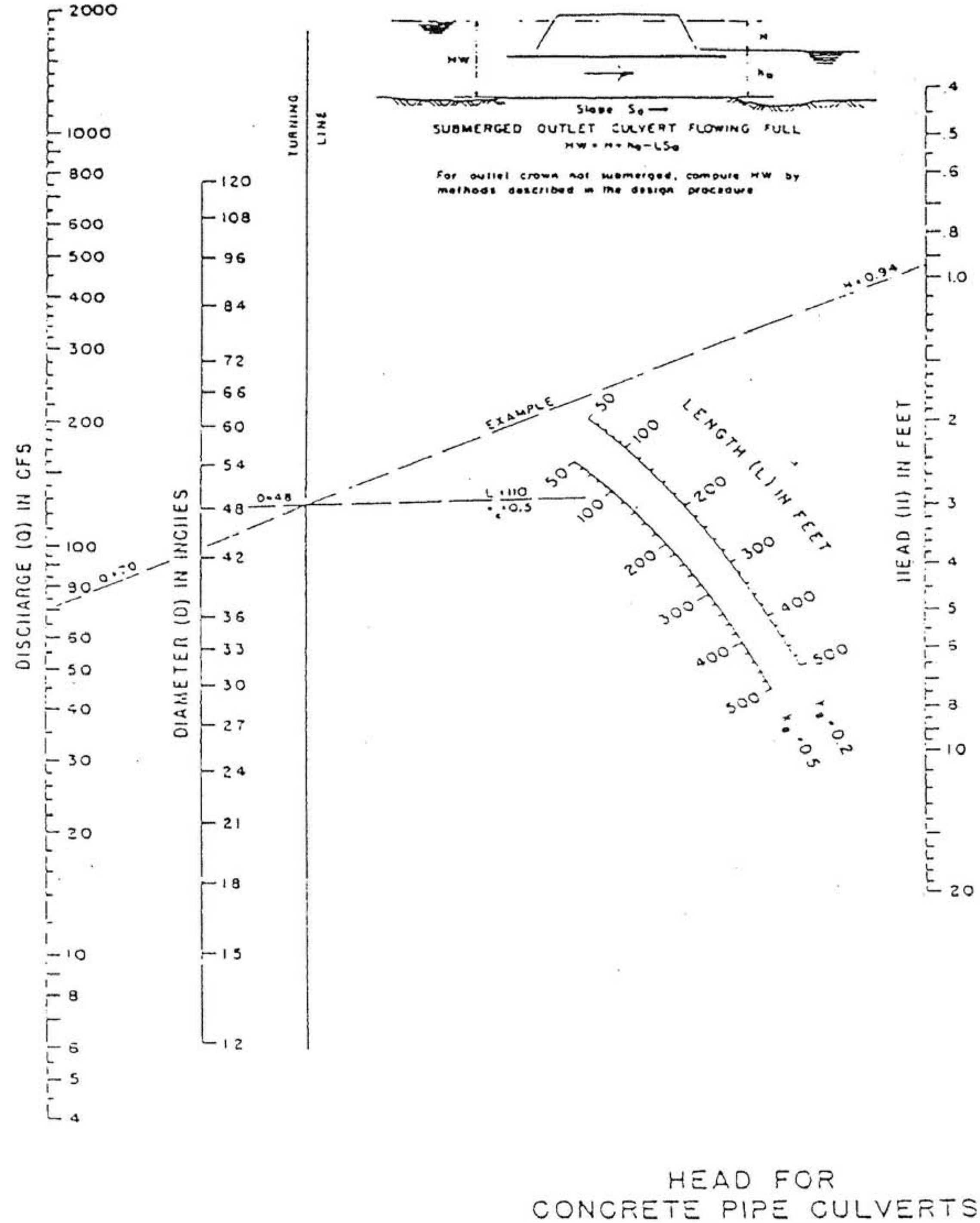

D=42 inches (3.5 feet)
Q=120 cfs
HW*/D HW feet
(1) 2.5 8.8
(2) 2.1 7.4
(3) 2.2 7.7
*D in feet
DIAMETER OF CULVERT (D) IN INCHES
DISCHARGE (Q) IN CFS
HEADWATER DEPTH IN DIAMETERS (HW/D)
EXAMPLE
HW/D SCALE
ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove end with headwall
(3) Groove end projecting
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 DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL
HEADWATER SCALES 2&3 REVISED MAY 1964
BUREAU OF PUBLIC ROADS JAN. 1963

Figure 6-1



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
$n = 0.012$

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 the 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 6.7.5

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 6.7.6

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 single performance curve for the culvert.

(3) When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and equation 6.6 to calculate flow rates across the roadway.

$$\mathbf{Q = C_d\, L HW_r^{1.5}} \qquad \mathbf{(6.6)}$$

Where: Q = overtopping flow rate in ft^3/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 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 6.7.7

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 25-yr flood = 70 cfs
Discharge for 100-yr flood = 176 cfs
Allowable HW for 25-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 25-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 25-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 = (Area times 4/3.14)$^{0.5}$

   Therefore: D = [(14 sq. ft. x 4/3.14)$^{0.5}$ x 12 in/ft)]

   D = 50.7 in

3. A grooved end culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 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) x (0.0015) = 3.90 ft

6. Since HW for outlet control (3.90 ft) is greater than the HW for inlet control (3.72 ft), outlet control governs the culvert design.

Thus, the maximum headwater expected for a 25-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) x (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 25-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: $V = 176 / (\pi 4.5^2/4) = 11.1$ ft/s

With this high velocity an energy dissipater may be needed downstream from the culvert for streambank protection. It will first be necessary to compute a scour hole depth and then decide if protection is needed. See Chapter 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.

CULVERT DESIGN FORM

PROJECT: ______________________

STATION: ______________ SHEET ______ OF ______

DESIGNER / DATE: __________ / ______

REVIEWER / DATE: __________ / ______

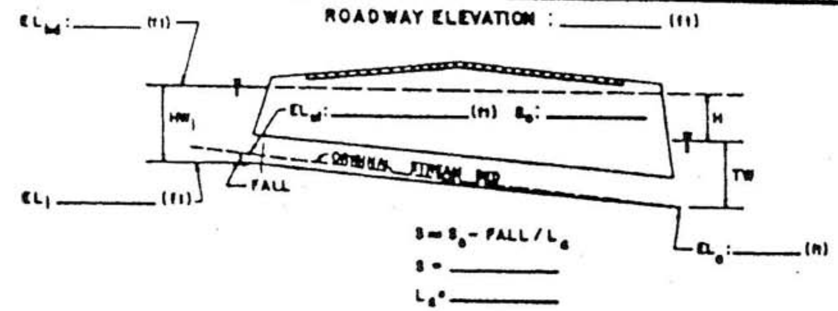
HYDROLOGICAL DATA

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

DESIGN FLOWS/TAILWATER

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

ROADWAY ELEVATION: ______ (ft)



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

(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

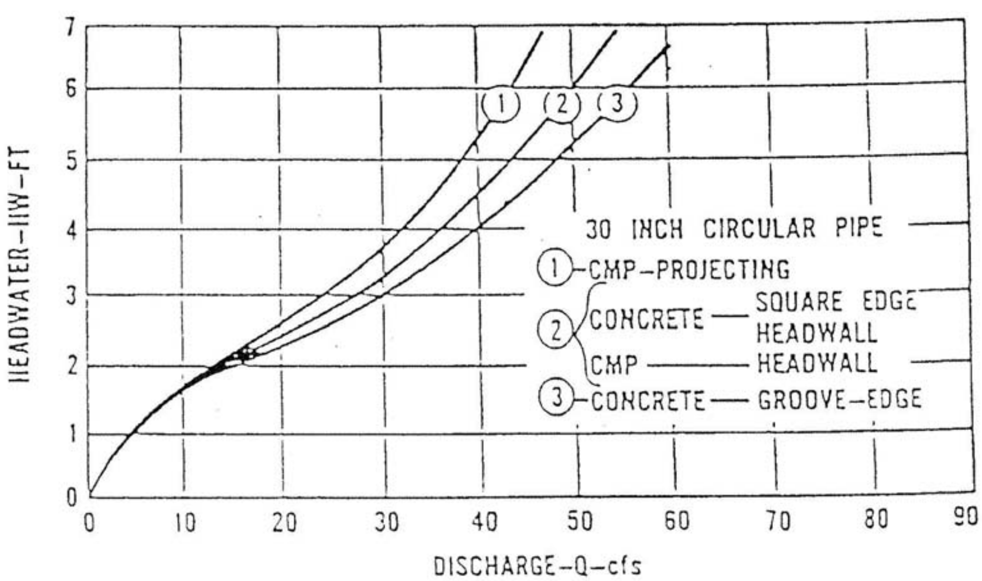
The flow condition for outlet control may be full or partly full for all or part of the culvert length. The design discharge usually results in full flow. Inlet improvements in these culverts reduce the entrance losses, which are only a small portion of the total headwater requirements. Therefore, only 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.

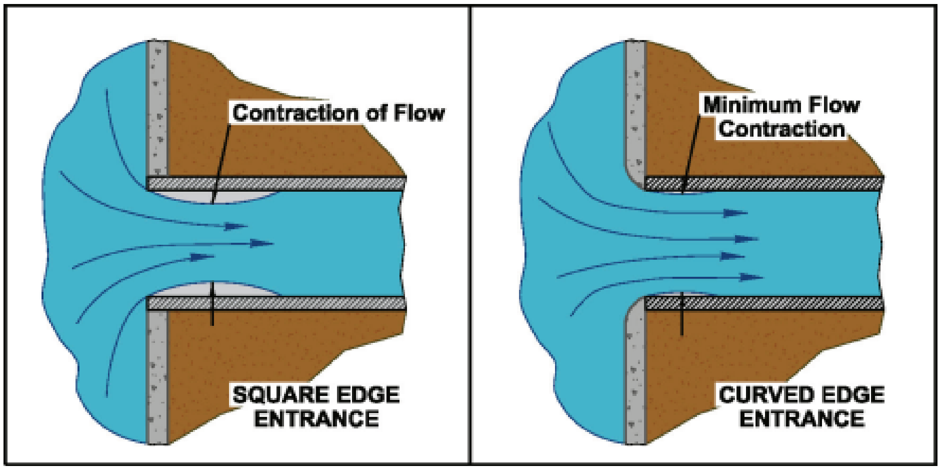


PERFORMANCE CURVES INLET CONTROL

Capacity Determinations 6.10.5

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 two inlet types.



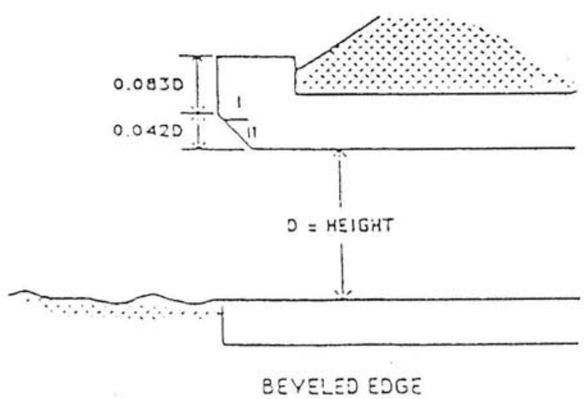
Source: FHWA-HIF-21-026 Hydraulic Design Series Number 5 (April 2012)

Improved Inlets 6.10.6

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.

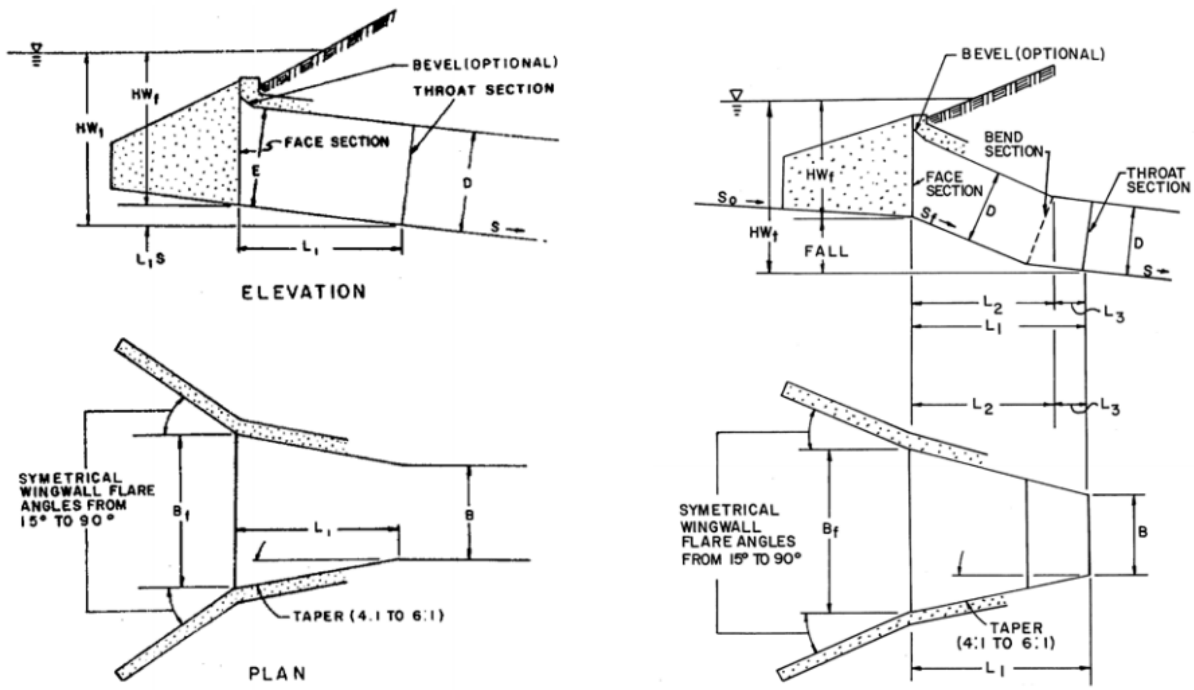
Bevel-edged Inlet 6.10.6.1

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.



Source: www.lincoln.ne.gov/city/ltu/wetershed/dcm/pdf/chapter4.pdf.

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. Beveled inlets should be used on all culverts inlet when feasible.
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. 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. FHWA (2005a) Hydraulic Design of Highway Culverts provides guidance on the design of Improved inlets.



Side-tapered and Slope-tapered improved inlets

Source: Urban Drainage and Flood Control District Urban Storm Drainage Criteria Manual Volume 2

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 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.

Federal Highway Administration. 1978. Hydraulics of Bridge Waterways. Hydraulic Design Series No. 1.

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

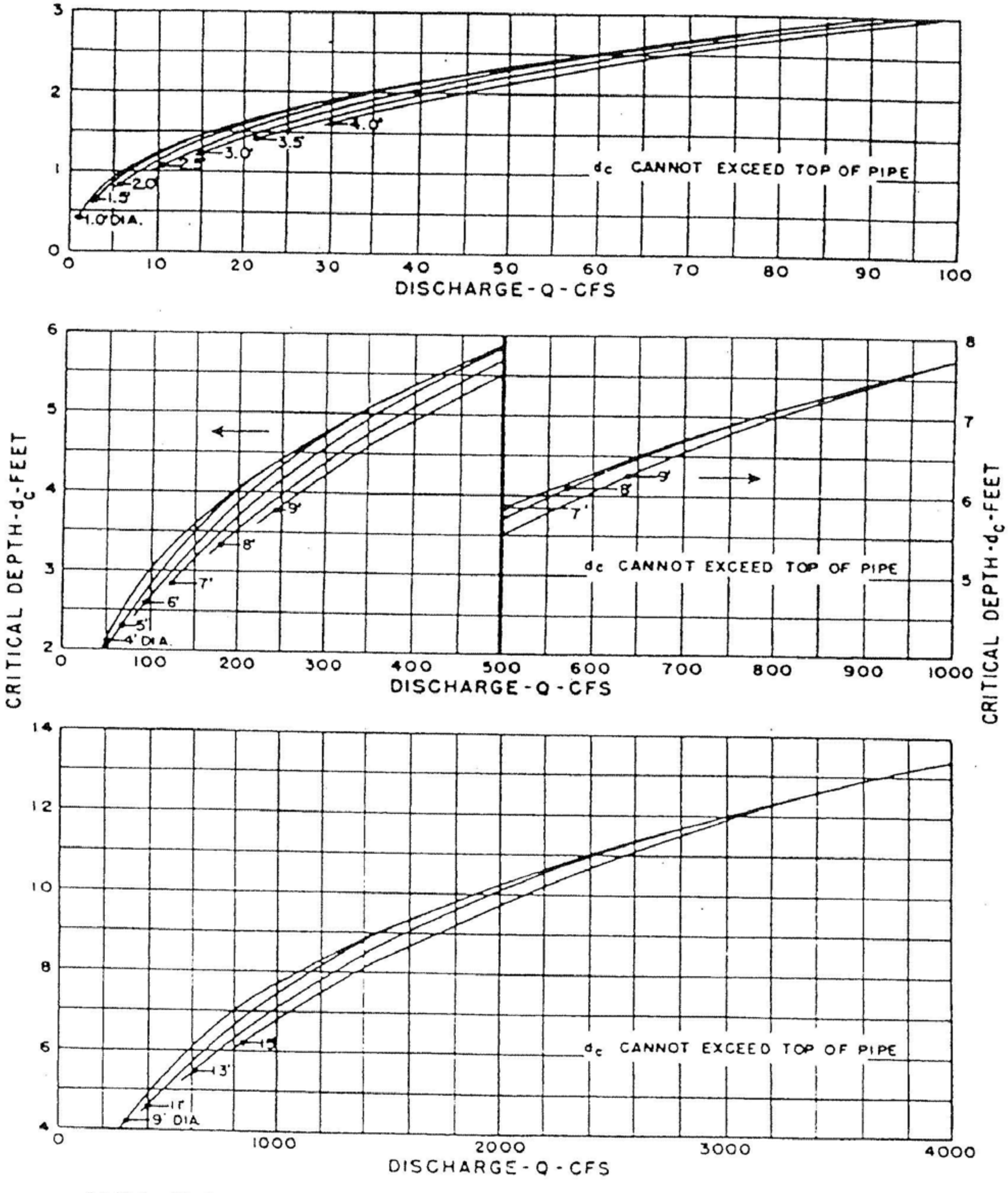
Federal Highway Administration. 1971. Debris-Control Structures. Hydraulic Engineering Circular No. 9.

Federal Highway Administration. 1987. HY8 Culvert Analysis Microcomputer Program Applications Guide. Hydraulic Microcomputer Program HY8.

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.

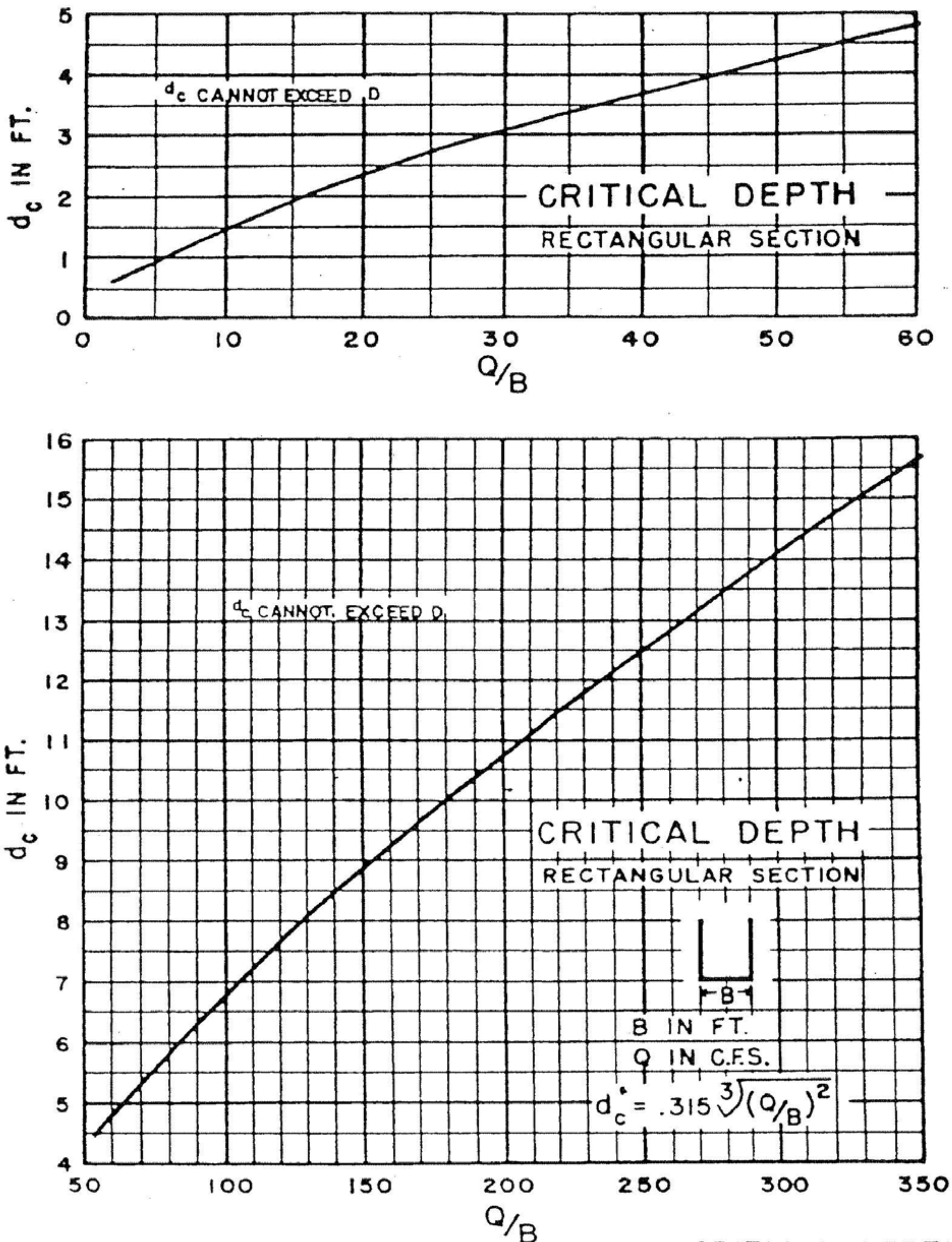
APPENDIX A – CRITICAL DEPTH CHARTS



BUREAU OF PUBLIC ROADS

JAN 1964

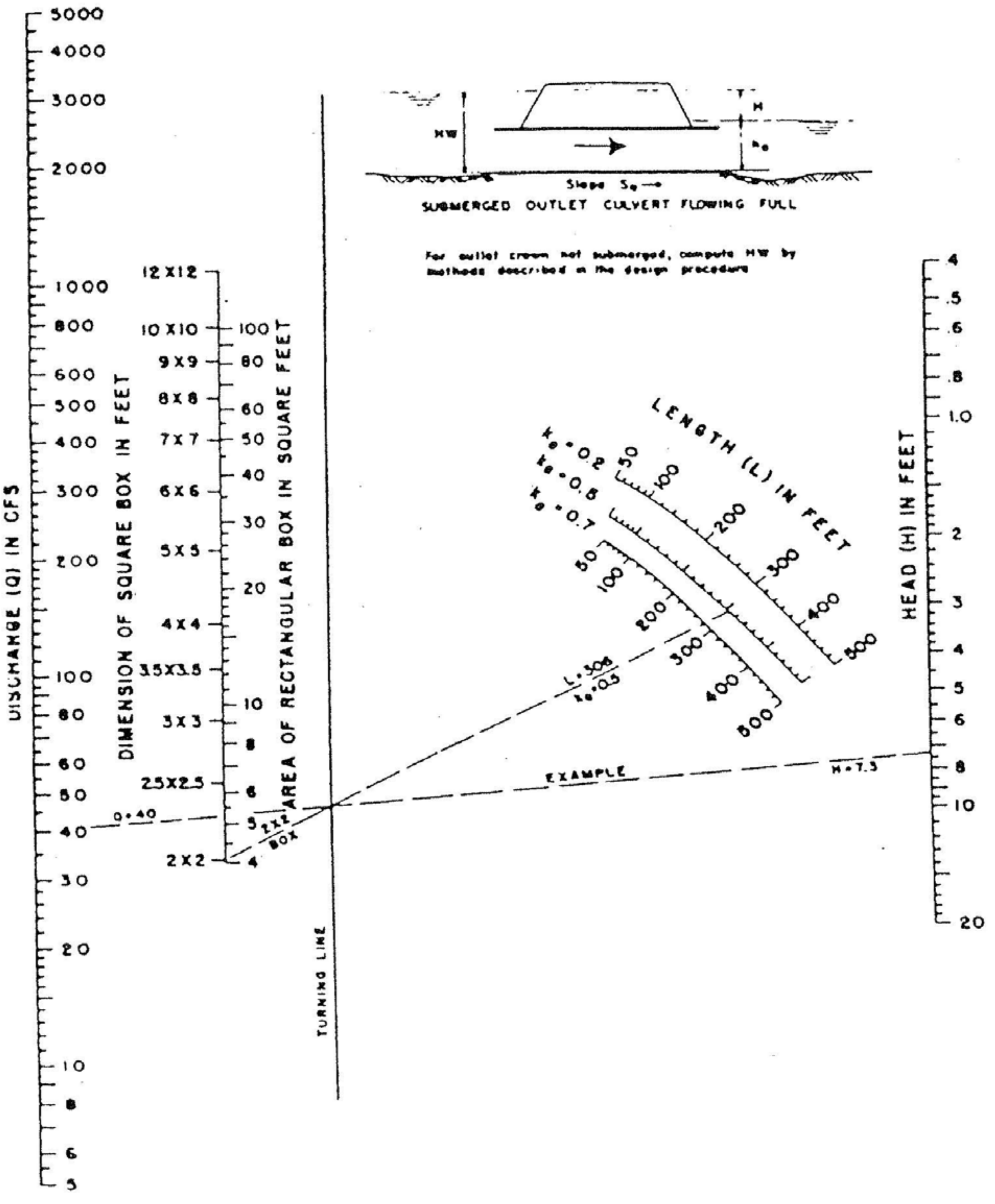
CRITICAL DEPTH
CIRCULAR PIPE



BUREAU OF PUBLIC ROADS JAN 1963

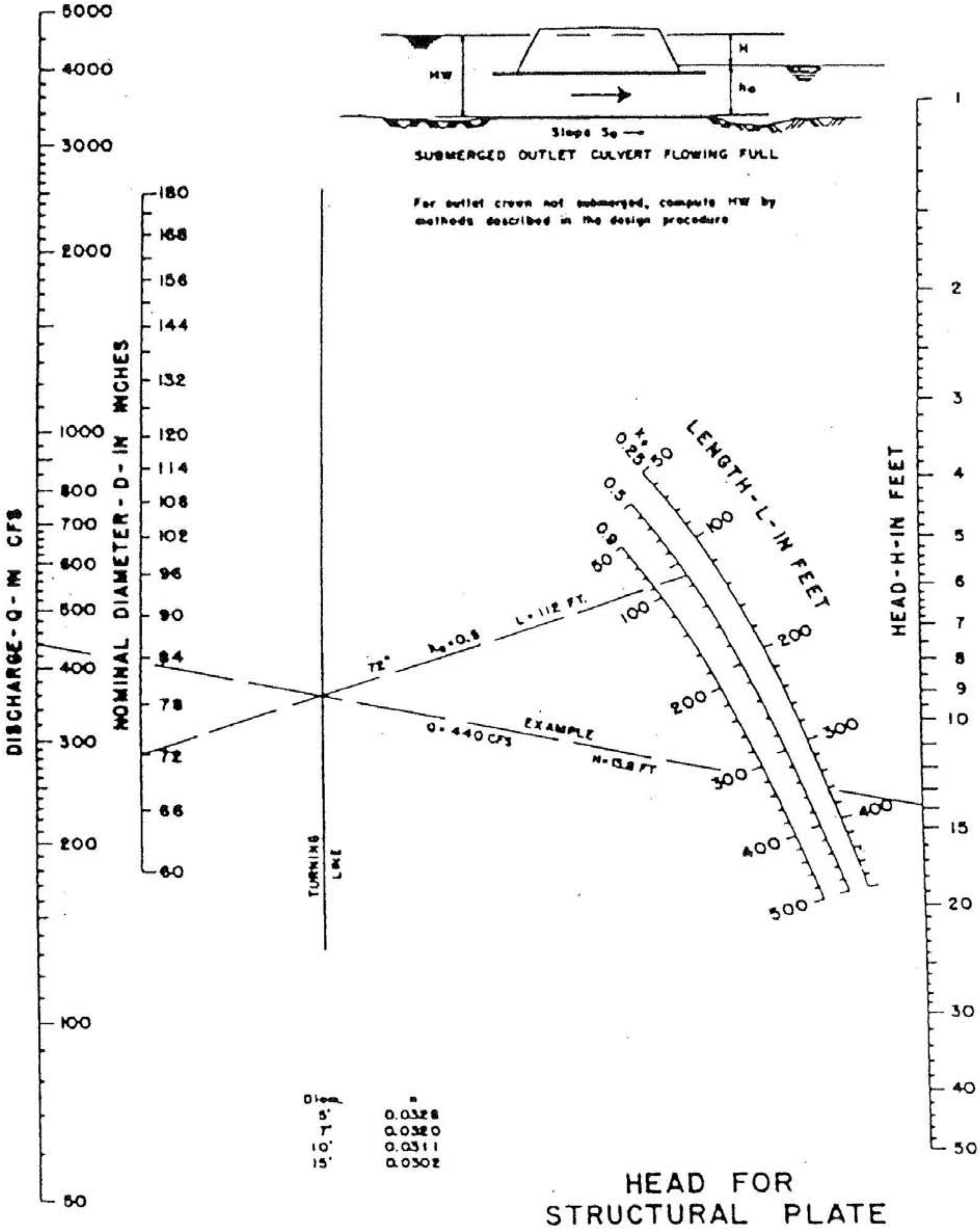
CRITICAL DEPTH
RECTANGULAR SECTION

APPENDIX B – CONVENTIONAL NOMOGRAPHS



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

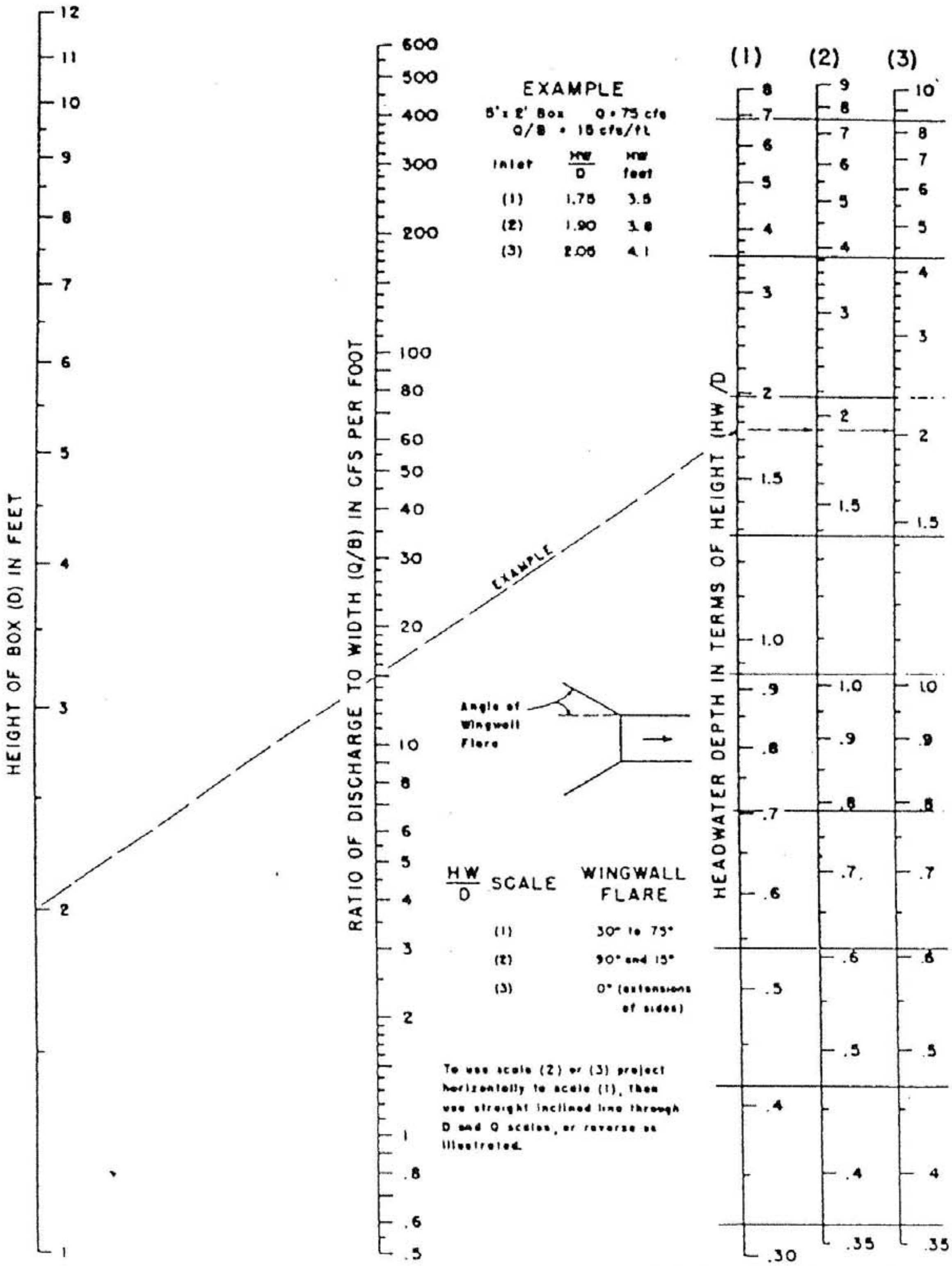
BU OF PUBLIC ROADS JAN. 1963



HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL

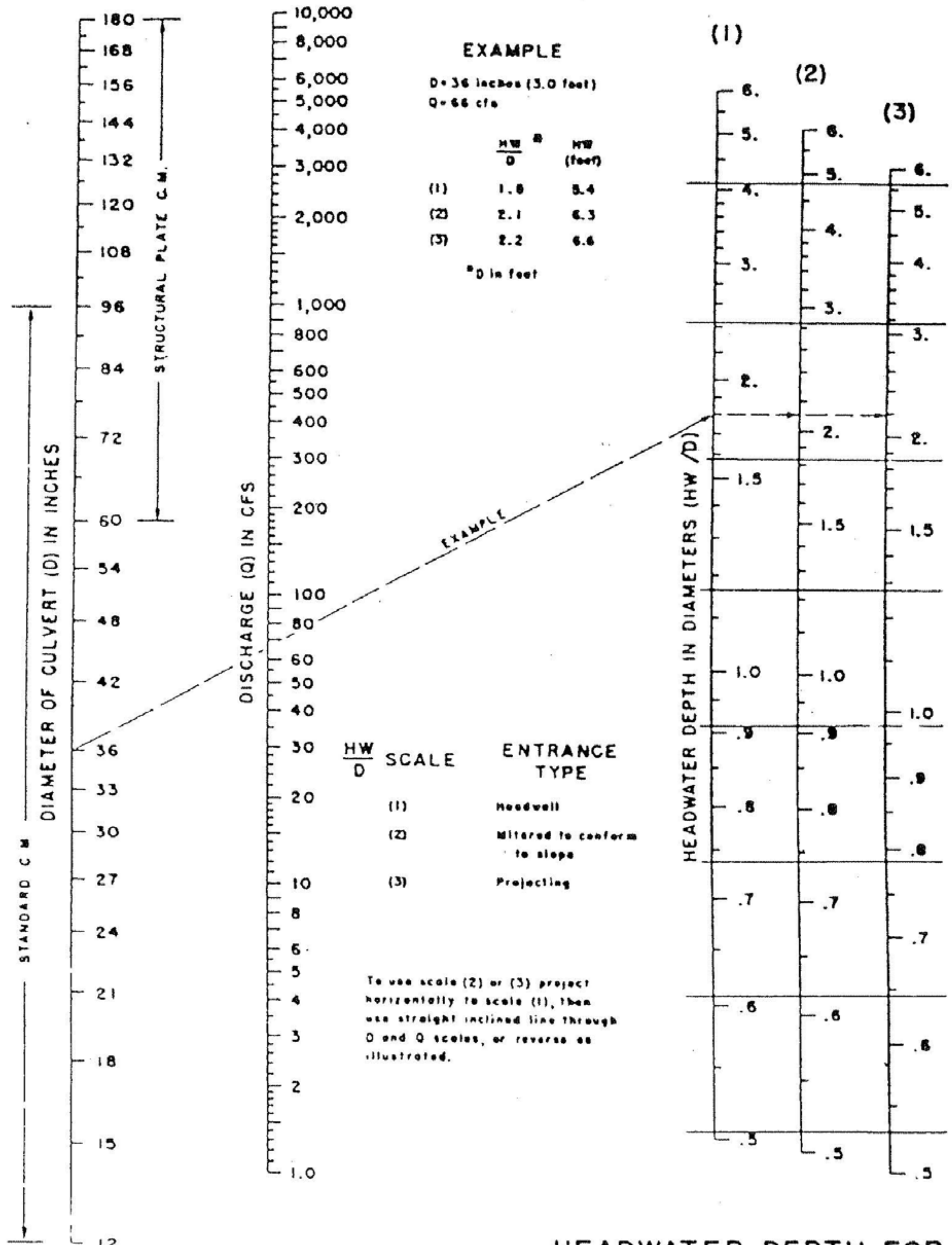
$n = 0.0328$ TO 0.0302

BUREAU OF PUBLIC ROADS JAN. 1963



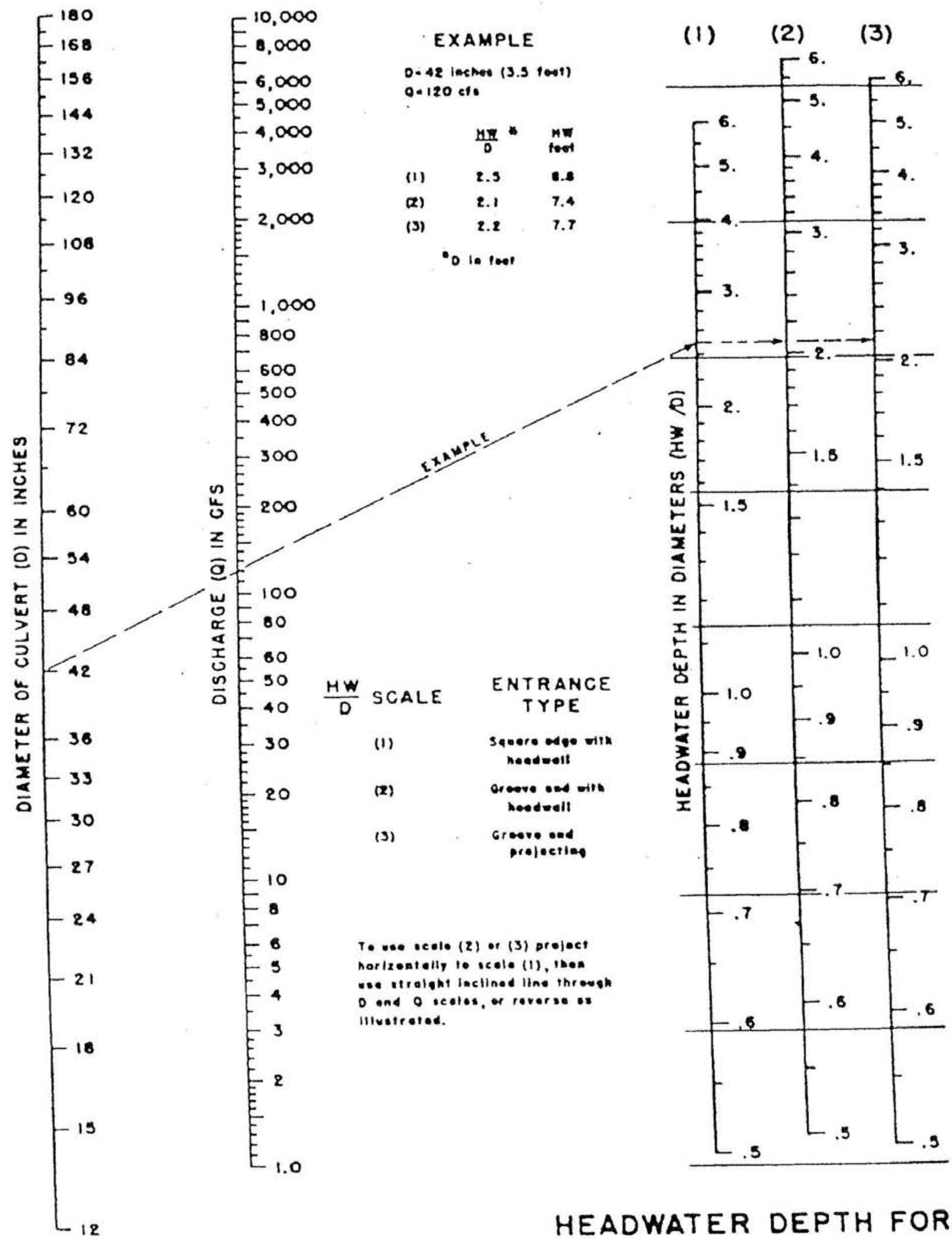
HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN 1963



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

BUREAU OF PUBLIC ROADS JAN 1963



HEADWATER SCALES 2&3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

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 and 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 24-hour 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 24-hour 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 24-hour 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 24-hour 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.

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.

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 24-hour design storm high water elevation

Grading and Depth of Earthern Storage Facility (contined)

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 24-hour storm. The sizing of a particular outlet works shall be based on results of hydrologic routing calculations. Minimum barrels through embankments are 12 inch pipes with corresponding orifice plates. Any orifice smaller than 4 inches in diameter must be protected to prevent blockage. A 2 foot by 2 foot concrete pad must be placed in front of any orifice plate at the invert of the outlet. If the spillway is in fill material then the spillway must be lined.

Dam Safety Act 7.3.6

Under the Dam Safety Act regulations, a dam is a structure ad appurtenant works erected to impound or divert water. Structures less than 25 feet in height or that have an impoundment capacity of less than 50 acre-feet, unless the Department determines that failure of the dam could result in the loss of human life or significant damage to property below the dam, are exempt. Any questions concerning a specific design or application should be addressed to the North Carolina Department of Environment, Quality, Dam Safety Section Of Energy, Mineral and Land Resources (800-858-0368).

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.

Off-site Storm Water Detention Facilities 7.3.8

When off-site storm water detention facilities (or for storm water release through recorded easements to regulated floodway in lieu of detention) the following requirement must be met:

- All pipes/channels leading from the subject site to the off-site storm water detention facility (or regulated Floodway) must be sized to carry the 10-year 24-hour storm water runoff. (25-year 24-hour stormwater if routed under and existing roadway.)
- A “Permanent Detention Easement” leading from the subject site to the off-site detention facility (or regulated floodway) must be shown on a map which has been recorded with the Union County Register of Deeds Office. This easement should be centered on the pipe or channel and must be at least 20 feet wide for pipes and 25 feet wide for channels (refer to Town Standards for easement widths). A metes and bounds description is not required for this portion of the easement.
- A “Permanent Detention Easement” which encompasses the detention facility must be shown on the recorded map. This easement must be described by metes and bounds.
- The recorded map must have a note which clearly states who is responsible for maintenance of the detention facility, pipes, and/or channels located within the Permanent Detention Easements (these easements will not be maintained by the Town).
- For off-site detention facilities the recorded map must have a note stating, “The purpose of the Permanent Detention Easement is to provide storm water detention for Lot(s) _____. The pipes and/or channels located within the Permanent Detention easement and leading to the detention facility carry unrestricted storm water flow from the developed upstream Lot(s) ___.

- For storm water released to regulated floodway through an easement, the recorded map must have a note stating “The purpose of the Permanent Detention Easement is to allow storm water release directly to regulated floodway in lieu of on-site storm water detention. The pipes and/or channels located within the Permanent Detention Easement and leading to the regulated floodway carry unrestricted storm water flow from the developed upstream Lot(s) ___.

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).

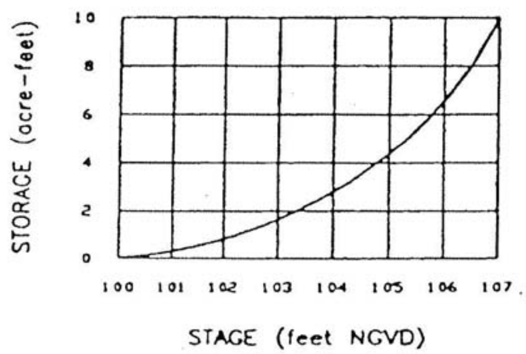


Figure 7-1 Example Stage-Storage Curve

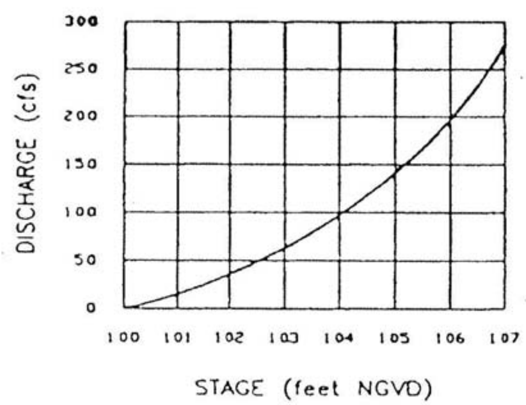


Figure 7-2 Example Stage-Discharge Curve

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 24-hour design storms using the procedures outlined in the Hydrology Chapter. Both pre and post-development hydrographs are required for the 2- and 10-year 24-hour design storms. Only the post-development hydrograph is required for the 50-year 24-hour 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 24-hour 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 or 10-year 24-hour 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. Computer based methods, such as HEC-1/HEC-HMS, are widely available perform these iterations quickly. Other computer programs can provide similar results. For guidelines for using other party software, please refer to Section 2.1.2 – Hydrologic Method.

7.5 Outlet Hydraulics

Outlets
7.5.1

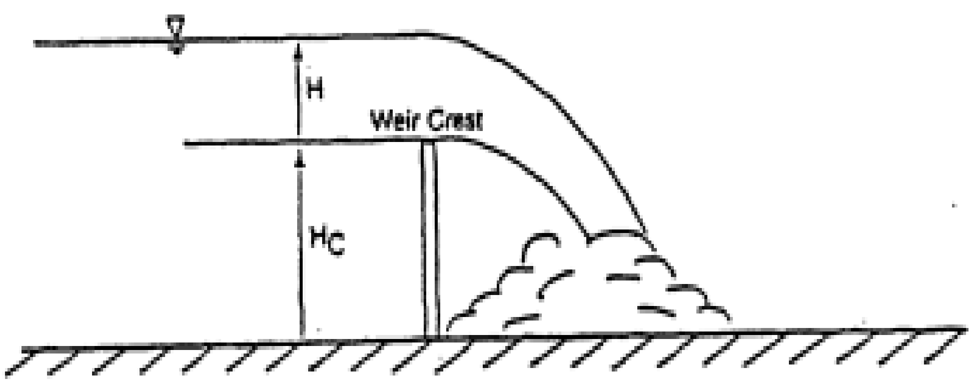
Sharp-crested weir flow equation for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlets 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):

$$\mathbf{Q = CLH^{1.5}} \qquad \textbf{(7.1)}$$

Where: Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
L = Horizontal weir length (ft)
C = sharp crested weir coefficient, use 3.3



Sharp-Crested Weir and Head

Broad-Crested Weirs
7.5.3

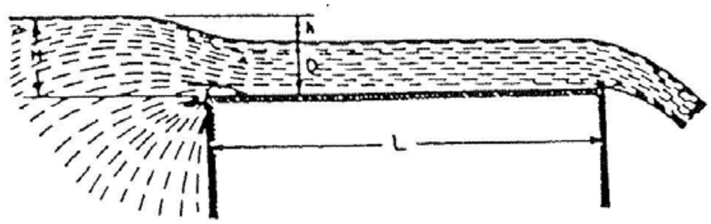
The equation generally used for the broad-crested weir (see sketch below) is (Brater and King, 1976):

$$\mathbf{Q = CLH^{1.5}} \qquad \textbf{(7.2)}$$

Where: Q = discharge (cfs)
C = broad-crested weir coefficient, use 3.0
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
Weirs
(continued)



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).

$$Q = 2.5 \tan(\theta/2) H_v^{2.5} \qquad (7.3)$$

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

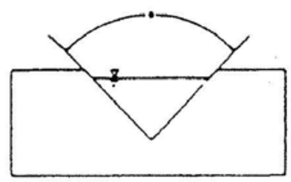


Table 7-2

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).

Orifices
7.5.5

The general equation for discharge through a submerged orifice is:

$$Q = CA(2gH)^{0.5} \quad \textbf{(7.4)}$$

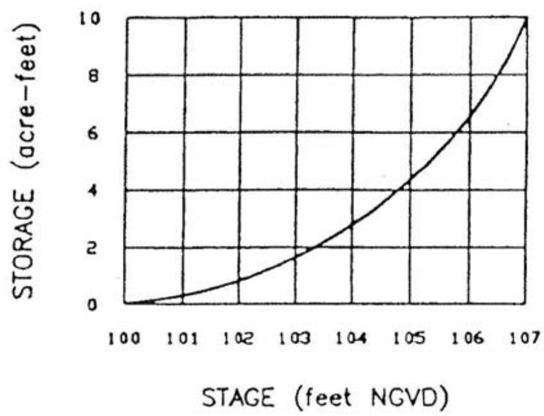
Where: Q = discharge (cfs)
A = cross-section area of smallest section (ft2)
g = acceleration due to gravity, 32.2 ft/s2
D = diameter of pipe (ft)
H = head on pipe, or the vertical distance from the center of the orifice to the upstream free-water surface
C = discharge coefficient (usually equal to 0.61-0.65,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

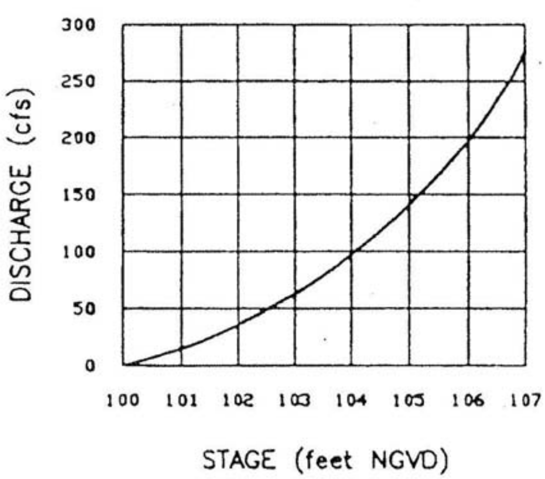
7.6 Routing Calculations

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)

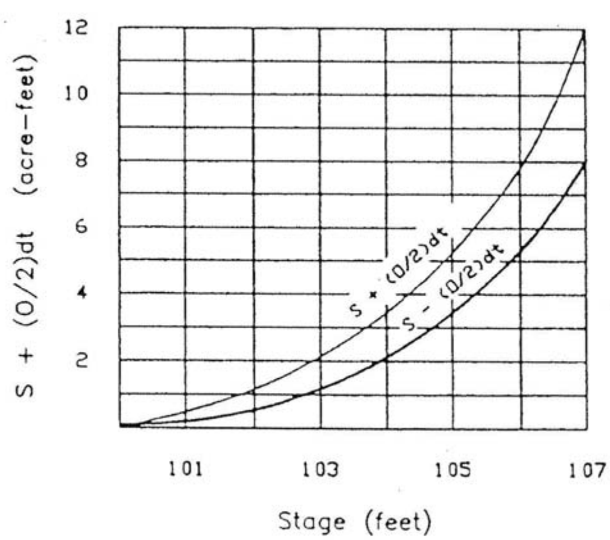
3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of S ± (O/2)dt versus stage. An example tabulation of storage characteristics curve data is shown below.

(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:

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

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 design storm. Mathematically, negative storage results on the routing calculation table, and in most spreadsheet applications this will effectively stop the calculation process. The routing may be re-initialized as follows:

   1. Set outflow equal to inflow

   2. Set stage based on outflow by referring to the already developed stage-discharge function.

   3. Set storage based on stage by referring to the already developed stage-storage function.

   4. Restart the routing and repeat steps 1-3 until the system behaves.

(Adapted from H.R. Malcom, P.E., Elements of Urban Stormwater Design, NCSU Press, 1989)

7.7 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.

7.8 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 Town of Waxhaw Engineering Design and Construction 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 dissipator becomes an integral 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 dissipators shall be employed whenever the velocity of flows leaving a storm water management facility exceeds the erosive velocity of the downstream channel system. Several standard energy dissipator designs have been documented by the U.S. Department of Transportation including impact basins, drop structures, stilling wells, and riprap.

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 306.1 of the Waxhaw Engineering Design and Construction Standards Procedures 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.

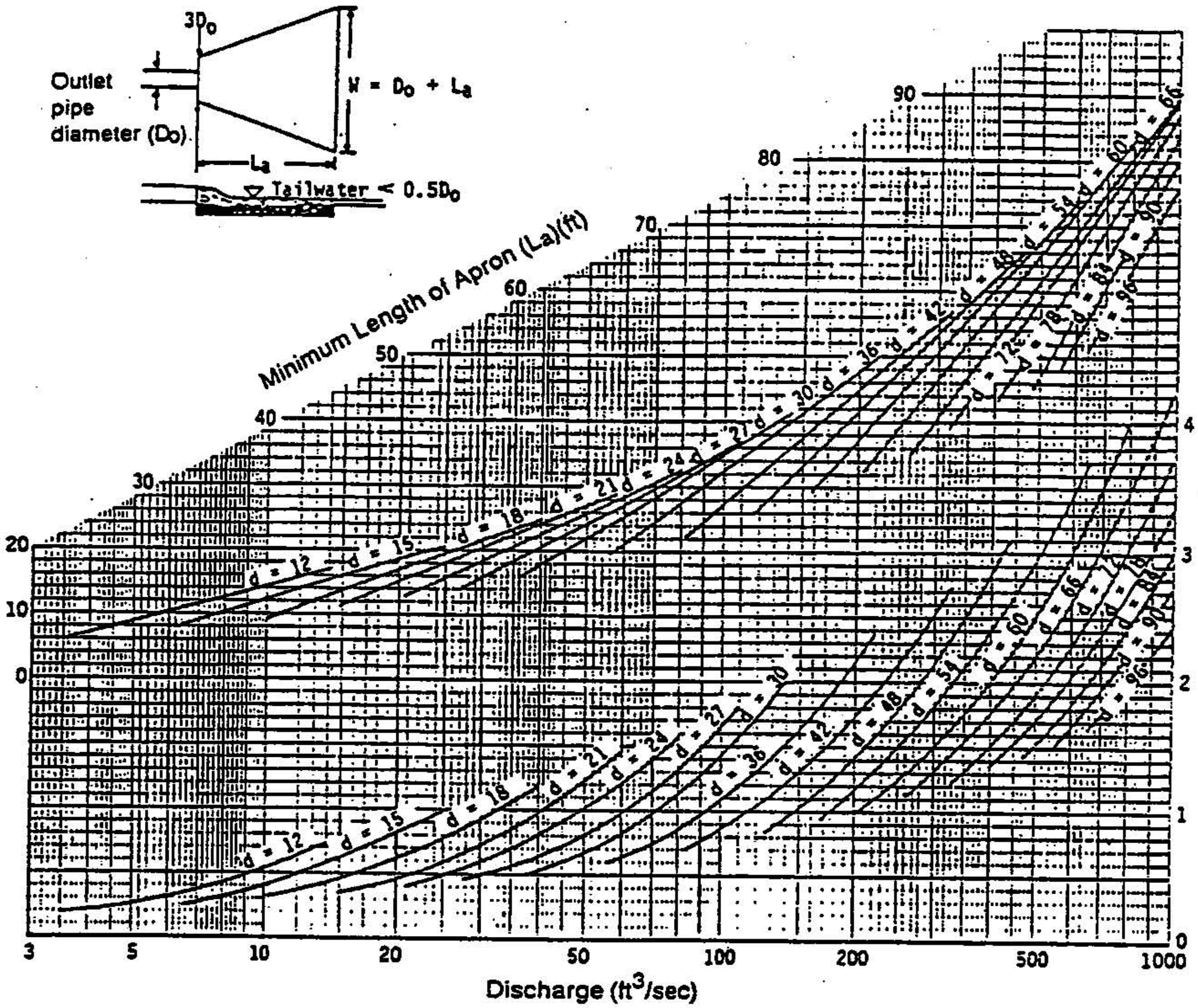
2. Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figure 8-1 and 8-2. If tailwater conditions are uncertain find the values for both minimum and maximum conditions and size the apron as shown in Figure 8-3.

   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:

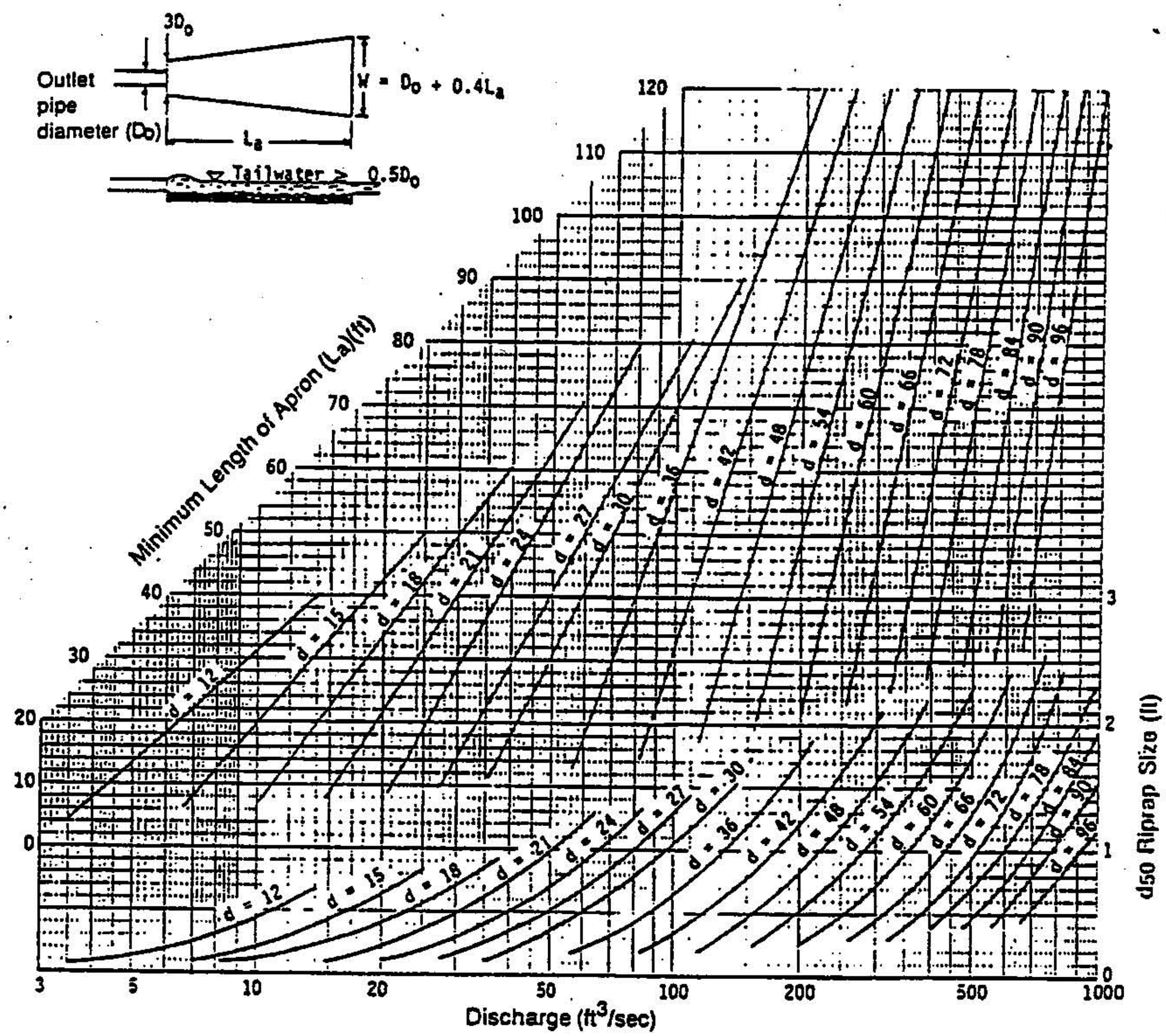
      Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the 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

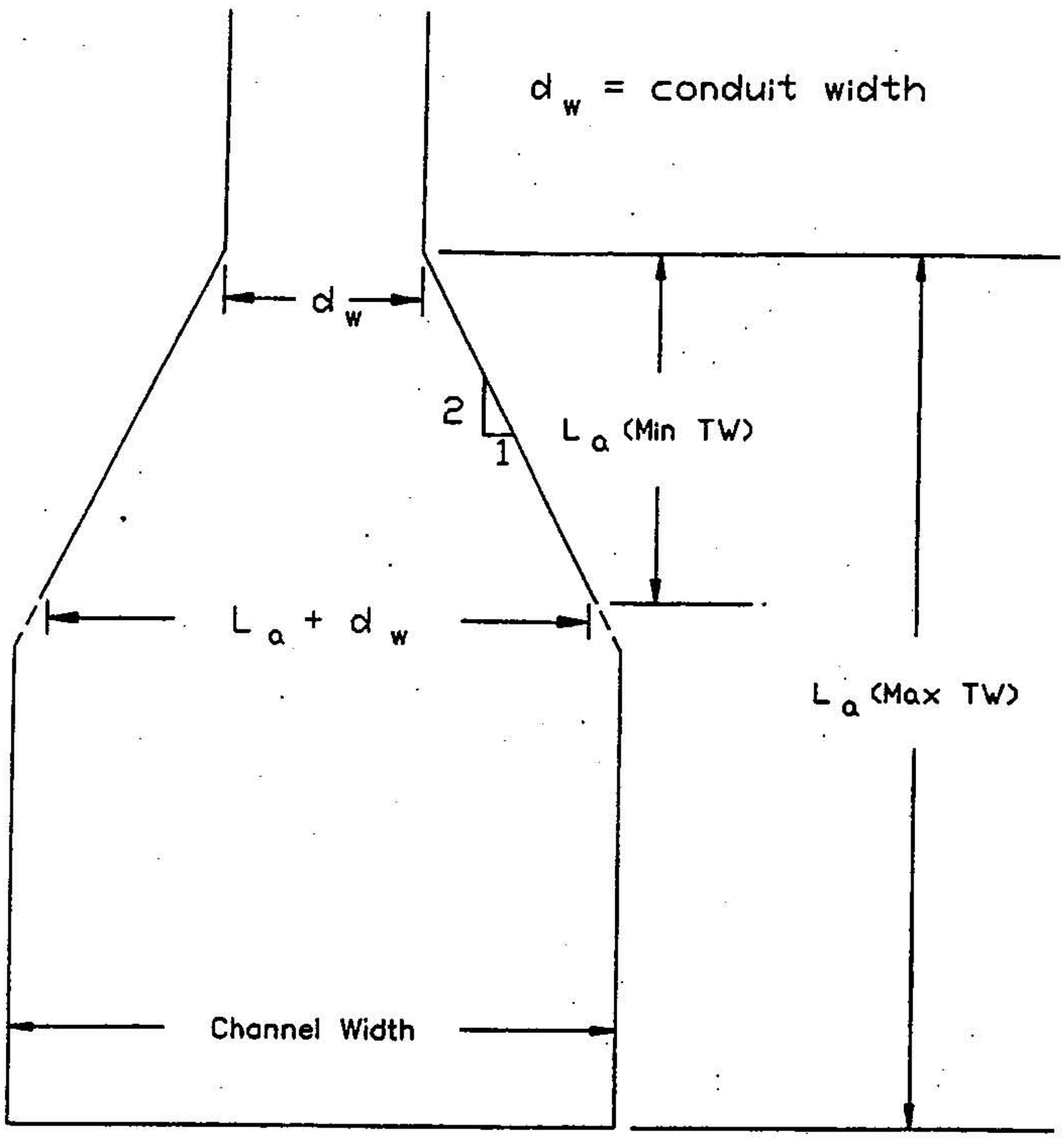
Figure 8-1



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 lower 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 that is 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 x d50

d_{50} = the median stone size in well-graded riprap apron

2. The riprap thickness should be 1.5 times the median stone diameter or 10 inches, whichever is greater.

Apron thickness = 1.5 x d_{50}

An appropriate filter fabric is required under the all riprap aprons.

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. Either the pipe or 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.

7. The apron shall be as straight as possible, aligned with channel flow. 90 degree outfalls are not permitted.

8. The approved plans shall accurately identify the apron limits, easement limits should be provided encompassing these limits.

NYDOT Method 8.4.4

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 8-5 following the steps below.

1. Estimate flow velocity V_0 at the culvert or paved channel outlet.

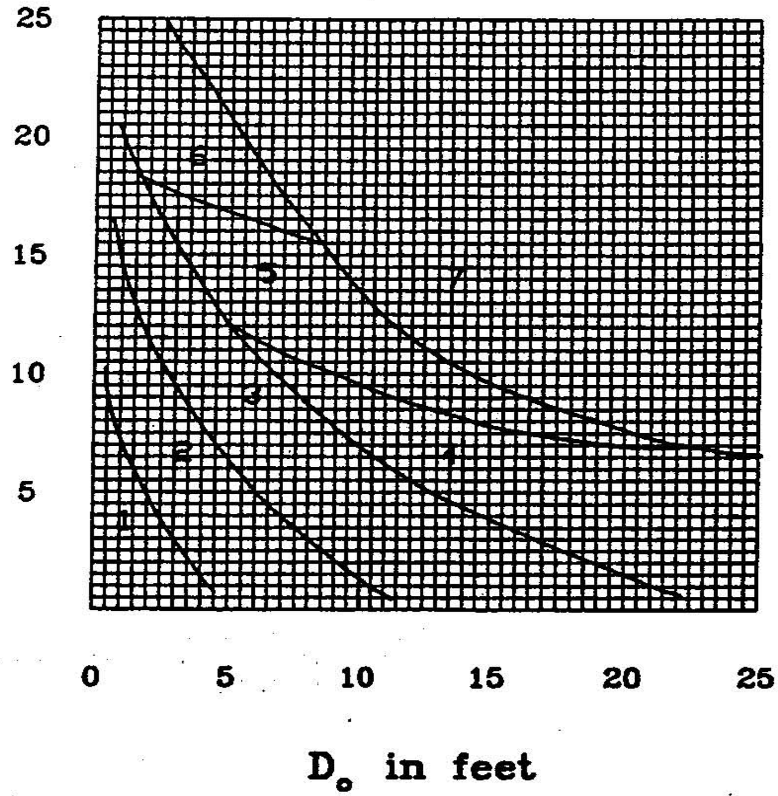
2. For pipe culverts, D is the diameter. For each pipe, box culverts, and paved channel outlets, $D_0 = A_0$ where A_0 = cross sectional area of flow at outlet.

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

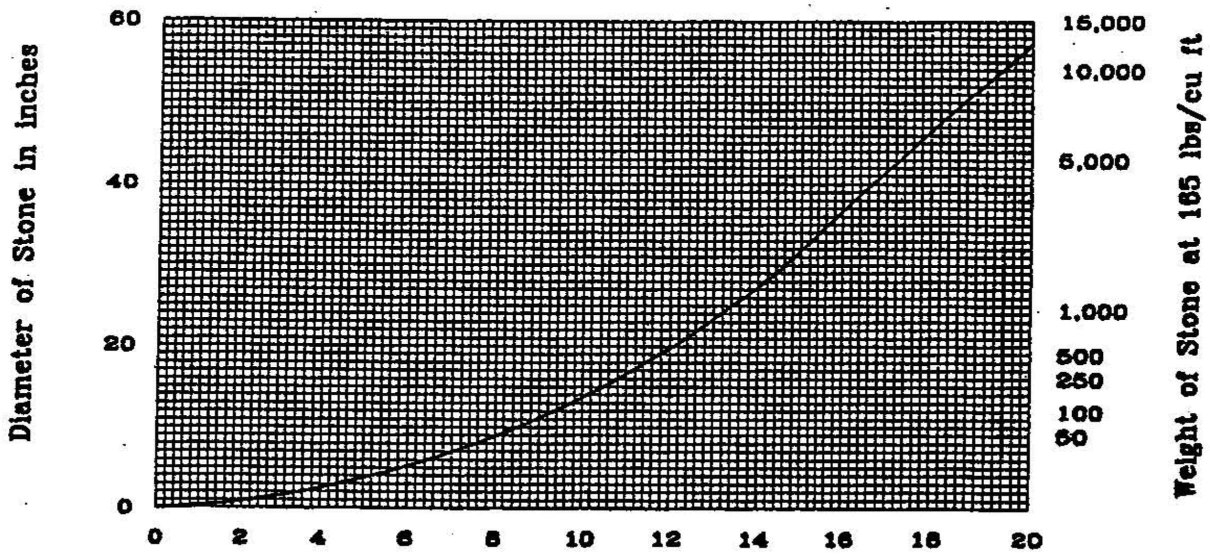
Source: "Bank and Channel Lining Procedures," New York Department of Transportation, Division of Design and Construction, 1971

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_{50}= 1.5 (1.2) 1.8 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_{50} = 1.5 (0.4) 0.6 foot. (10-inch minimum would be required)

References

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