
COPIES OF THESE
REGULATIONS AND TECHNICAL GUIDELINES FOR
STORMWATER MANAGEMENT

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CITY OF HUNTSVILLE
STORMWATER MANAGEMENT MANUAL

Prepared for

CITY OF HUNTSVILLE
URBAN DEVELOPMENT DIVISION
ENGINEERING DEPARTMENT

2020 EDITION

PREFACE

The Huntsville Stormwater Management Manual has been prepared to establish regulations and technical guidelines on stormwater management for developers, landowners, builders, architects, engineers, and others involved in development activities. The manual is made available ONLINE from the City of Huntsville website at www.huntsvilleal.gov

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ABBREVIATIONS

AASHTO	American Association of State Highway Transportation Officials
AMC	Antecedent MoistureCondition
cfs	Cubic feet per second
CM	corrugated metal
P	Pipe
csm/inch	cubic feet per second per square mile per inch
FEMA	Federal Emergency ManagementAgency
FHWA	Federal Highway Administration
IDF	intensity duration-frequency
NAVD	National AmericanVertical Datum
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Soil Conservation Service
SMM	Stormwater Management Manual
TVA	Tennessee Valley Authority
USDA	United States Department of Agriculture
USDOT	United States Department of Transportation
USGS	United States Geological Survey
WSO	Weather Service Office

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SUMMARY

The Huntsville Stormwater Management Manual (SMM) establishes policy, permitting requirements, design regulations, and technical information for stormwater activities in the City of Huntsville. The manual contains ten chapters, presented in two parts as listed below:

PART 1 POLICY, PERMITTING, AND REGULATIONS

1. Introduction
2. Policy and Permitting
3. Regulations for Facility Design and Floodplains

PART 2 TECHNICAL INFORMATION

4. Hydrology
5. Open Channel Hydraulics
6. Culvert Hydraulics
7. Gutter and Inlet Hydraulics
8. Storm Sewer Hydraulics
9. Storage System Hydraulics
10. Erosion and Sediment Control

The regulatory chapters in Part 1 are of particular interest to developers, builders, realtors, landowners, architects, and engineers, while the technical chapters in Part 2 are oriented to the concerns of designers and builders.

Tables and figures are located at the end of each chapter, with tables preceding figures. Each chapter is introduced by a table of contents that includes table and figure titles. In the text, equation terms and symbols are defined immediately following each equation. All references are listed at the end of the manual.

Example problems are provided at the end of selected sections in the manual to demonstrate the application of procedures. A summary list of the example problems follows:

CHAPTER 1
INTRODUCTION

PART 1
PART I – POLICY, PERMITTING, AND REGULATIONS

PART 1 – POLICY, PERMITTING, AND REGULATIONS

Part 1 of the Huntsville Stormwater Management Manual will be of interest to developers, builders, realtors, landowners, architects, and engineers who need to be familiar with the regulations associated with land development or alterations to existing facilities. The following chapters are included in Part 1:

1. Introduction
2. Policy and Permitting
3. Regulations for Facility Design and Floodplains

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INTRODUCTION

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CHAPTER 1
INTRODUCTION

1.1 PURPOSE

The purpose of the Stormwater Management Manual (SMM) is to establish regulations and provide stormwater design information for local agencies, engineers, developers, or others whose activities concern stormwater management in the City of Huntsville. The manual has been prepared to document the following items:

1. Uniform stormwater design criteria and procedures

2. Regulations for grading and building permits

3. Regulations and policies for land disturbance activities

4. Technical information to assist with the design of stormwater drainage systems

5. Published data applicable to the City of Huntsville

This manual should be used to guide persons involved in the activities mentioned above through the required procedures to obtain approval from the Director of City Engineering for grading, building, and construction permits, and subdivision development. Zoning and Building Code requirements are the responsibility of the City of Huntsville Planning Department and Inspection Department, as appropriate. The requirements contained in this manual are in addition to Zoning and Building Code requirements. Any conflicts arising from the implementation of the requirements of the manual with enforcement of Zoning and Building Code requirements shall be resolved by appropriate methods and procedures within the City of Huntsville.

1.2 SOURCE OF PROCEDURES

Procedures identified in this manual are generally found in readily available government publications. Information from these publications has been duplicated as required to supplement contents of the manual. Complete details and additional information on the procedures can be obtained from information on the procedures can be obtained from the original references.

1.3 LIMITATIONS

This manual neither replaces the need for engineering judgement nor precludes the use of information not presented. The emphasis is on desktop procedures that can be performed manually. To ensure that procedures are properly applied and adapted, the engineer must consider actual site conditions and project requirements.

1.4 UPDATING

The Huntsville SMM will be updated and revised, as necessary, to reflect up-to-date engineering practices and information applicable to Huntsville. The status and details of any updates or revisions can be obtained from the Engineering Division. Questions and any suggestions for changes should also be addressed to the Engineering Division.

1.5 DEFINITIONS

1. When used in this manual, the following words, terms, and phrases, whether used in the singular or plural, or their derivations, shall be defined as follows, unless the context clearly indicates a different meaning (should any conflict exist, final determination will be made by the Director of City Engineering):

   ADEM means the Alabama Department of Environmental Management.

   BMPs and *Best Management Practices* mean schedules of activities, prohibitions of practices, maintenance procedures and other management practices to prevent or reduce the discharge of pollutants to the municipal separate storm sewer system. BMPs also include treatment requirements, operating procedures and practices to control facility site runoff, spillage or leaks, sludge or waste disposal, or drainage from raw materials storage.

   Channel means a natural or artificial watercourse of perceptible extent with definite bed and banks to confine and conduct continuously or periodically flowing water. Channel flow is that water which is flowing within the limits of the defined channel.

   Director of City Engineering or *Director* means the Director of Engineering of the City of Huntsville or his/her authorized representatives, which includes, but is not necessarily limited to, his/her subordinates and designees.

   Cut means a portion of land surface or area from which earth has been removed or will be removed by excavation; the depth below original ground surface to the excavated surface.

   Detention means the temporary delay of storm runoff prior to discharge into receiving waters.

   Developer means any individual, firm, corporation, association, partnership or trust involved in commencing proceedings to effect development of land for himself or others.

   Drainage basin means a part of the surface of the earth that is occupied by and provides surface water runoff into a drainage system, which consists of a surface stream or a body of impounded surface water together with all tributary surface streams and bodies of impounded surface water.

   Erosion means the disintegration or wearing away of soil by the action of water.

   Excavation means the same as the term "cut."

Fill means or portion of land surface or area to which soil, rock or other materials have been or will be added; the height above original ground surface after the material has been or will be added.

Grading means any operation or occurrence by which the existing site elevations are changed; or where any ground cover, natural or manmade, is removed; or any watercourse or body of water, either natural or manmade, is relocated on any site, thereby creating an unprotected area. This includes stripping, cutting, filling, stockpiling or any combination, and shall apply to the land in its cut or filled condition.

Impervious surface means any ground or structural surface that water cannot penetrate or through which water penetrates with great difficulty.

Retention means the prevention of storm runoff from direct discharge into receiving waters. Examples include systems which discharge through percolation, exfiltration, filtered bleed-down and evaporation processes.

Sediment means solid material, both mineral and organic, that is in suspension, is being transported, or has been moved from its site of origin by air, water or gravity as a product of erosion.

Site means all contiguous land and bodies of water in one ownership, graded or proposed for grading or development as a unit, although not necessarily at one time.

Slope means degree of deviation of a surface from the horizontal, usually expressed in percent or ratio.

Stripping means any activity that removes or significantly disturbs the vegetative surface cover, including clearing and grubbing operations.

Stormwater means stormwater runoff, snow melt runoff, and surface runoff and drainage.

2. The definitions set forth in Article 62, Section 62.2 of the 1989 Huntsville Zoning Ordinance, as amended from time to time, are incorporated herein by reference as if fully set forth, and shall apply to those provisions of this manual that relate to the floodplain, unless the context or the terms of the provision require otherwise. Where there is a conflict between the definitions set forth in Section 1.5.1 and Section 62.2, the definitions set forth in Section 62.2 shall apply.

CHAPTER 2

POLICY AND PERMITTING

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CHAPTER 2
POLICY AND PERMITTING

2.1 OBJECTIVES

The City of Huntsville's stormwater management program has the following objectives:

1. Protect human life and health.

2. Protect water quality and reduce nonpoint source pollution to waterbodies, wetlands, and ground water within and immediately downstream of the City of Huntsville.

3. Support compliance with the current Municipal Separate Storm Sewer System (MS4) permit and the Huntsville Storm Water Management Program Plan (SWMPP).

4. Reduce expenditure of public money for flood control projects.

5. Reduce the need for rescue and relief efforts associated with flooding.

6. Provide for the sound use and development of flood-prone areas so as to maximize beneficial use without increasing flood hazard potential.

7. Reduce damage to public facilities and utilities such as water and sewer lines; electric, telephone, and gas facilities; and streets and bridges located in floodplains.

8. Ensure a functional stormwater drainage system that will not result in excessive maintenance costs.

9. Encourage the use of natural and aesthetically pleasing design.

10. Encourage the improvement of existing flooding problems in conjunction with new development and redevelopment.

11. Reduce the impact on public and private property caused by the accumulation of mud, dirt, water, debris, and other construction materials.

2.1.1 STANDARDS

To implement the objectives presented above, the following standards shall apply:

1. The calculated peak rate of stormwater runoff resulting from a ten-year return period, twenty-four (24) hour duration, type II storm distribution (as defined by the Natural

Resources Conservation Services (NRCS), formerly the Soil Conservation Services, shall be no greater after development of a site than before development of a site.

2. Stormwater quality protection shall be provided to manage 100% of the water quality treatment volume on-site. The treatment volume is based on the runoff from a 1.1-inch, 24-hour storm, preceded by a 72-hour antecedent dry period.

3. No construction, whether by private or public action, shall be performed in such a manner as to materially increase the degree (e.g. frequency and/or depth) of flooding in its vicinity or in other areas except that construction fill within the floodway fringe areas and along unstudied streams shall be accomplished according to the floodplain regulations of the City of Huntsville Zoning Ordinance.

2.2 PERMIT REQUIREMENTS

A grading permit issued by the Director of City Engineering will be required before any land disturbance activity can begin, unless the disturbance is exempted by the provisions in Section 2.3. For the purposes of this chapter the term “land disturbance activity” or “land disturbance” means grading, excavating, clearing, filling, and/or the construction or paving of any parking lot or existing parking area exceeding 10,000 square feet of impervious area.

2.2.1 BUILIDING PERMITS

Building permits are issued by the City of Huntsville Inspection Department. All improvements, repairs, or construction with respect to structures within the City of Huntsville that require a permit from the Inspection Department shall be approved by the Director of City Engineering prior to issuance of the permit, unless the structure falls into one of the following categories:

1. Residential dwellings on single lots within an approved subdivision with an approved grading plan that establish the stormwater drainage pattern for the site, and the proposed land disturbance activity, if any, does not alter or impair the approved drainage pattern.

2. Remodeling within the walls of an existing structure and no land disturbance activity is to occur.

The information and data that shall be furnished to the Director of City Engineering is listed in Section 2.5.2.

Retaining walls greater than 48 inches tall (measured from the top of the footing to the top of the wall) require a stamped engineering plan submitted to the Engineering Division for approval prior to construction of the wall.

Discharge of sump pumps and roof drains must be routed to the right of way or drainage easement and controlled within the property limits.

Equipment (e.g. heating, ventilating, and air conditioning units) pads and related equipment are not permitted in utility and drainage easements.

2.2.2 GRADING PERMITS

1. Subject to Subsection 2., grading permits are required from the Director of City Engineering before the commencement of any land disturbance activity within the City of Huntsville, unless exempted by the provisions of Section 2.3 GRADING PERMIT EXEMPTIONS.

2. No grading permit is required for land disturbance activity performed in connection with the construction, improvement, and/or repair of a structure that conforms with a grading, drainage, and/or erosion control plan that has been previously approved by the Engineering Division. Land disturbance activity which does not require a grading permit shall comply with the requirements of the previously approved grading, drainage, and/or erosion control plan without exception, including, but not limited to, disturbance of the natural ground cover.

3. The foregoing exception to the grading permit requirement and the following enumerated exemptions to the grading permit requirement shall not be construed as relieving the responsible party from complying with other applicable requirements of this manual, including, but not limited to, the floodplain requirements set forth herein; and/or making on-site drainage improvements that may be required.

4. Nothing in this manual, including, but not limited to, the approval of or exemption from grading permit requirements, shall be construed as relieving the responsible party from the requirements of any other applicable federal, state or local laws, including, but not limited to, on-site drainage improvements that may be required in accordance with adopted building and construction codes or from compliance with the Zoning Ordinance of the City of Huntsville or any other applicable laws or regulations.

2.2.3 ALTERATIONS TO EXISTING DRAINAGE; DISCHARGE PATTERN

1. Public Systems. A letter of approval from the Director of City Engineering shall be required for any alterations to existing drainage facilities maintained or proposed from maintenance by the City of Huntsville involving the removal or installation of underground pipe, including culverts, and construction or alteration of drainage ditches, swales or berms.

2. Private Systems. No letter of approval will be required for alterations to private drainage systems with tributary areas less than 5,000 square feet; however, alterations to those private drainage systems with tributary area equal or greater than 5,000 square feet shall meet the requirements of this manual. The exception regarding alteration to private drainage systems with tributary are less than 5,000 square feet does not imply that the work has the approval of the City of Huntsville, and does not relieve the responsible party from the requirement of any other applicable federal, state or local laws. In the case of single family attached and detached residential structures, stormwater may be discharged

onto flat areas such as streets or lawns if drainage is provided so that the stormwater will flow away from the building and not adversely affect adjacent properties.

3. Existing Discharge Pattern. Development of a site shall conform to the stormwater discharge pattern established in an existing grading plan that includes the site, unless a deviation from such plan is required or allowed by the Director of City Engineering in accordance with this manual. Deviations from the established drainage pattern shall require the approval of the Director before the issuance of a building permit in accordance with this manual. It shall be a violation of this manual for the holder of a building permit or grading permit to fail to follow the established drainage pattern for the site or the approved deviations. The Director may withhold approval of the issuance of a certificate of occupancy until either the established drainage pattern or the approved deviations are met.

4. Discharge Pattern. For construction that includes grading and where there is no grading plan in place that establishes the stormwater discharge pattern for the site, the Director of City Engineering may condition the building permit to include requirements designed to carry stormwater discharge to public drainage easements or facilities, or to private drainage easements or facilities that are designed and authorized to accommodate said drainage. The building permit holder shall grade the site to conform to such required conditions. The Director may withhold approval of the issuance of a certificate of occupancy until such required conditions have been met.

2.3 GRADING PERMIT EXEMPTIONS

2.3.1 EXCAVATION OR FILL

No grading permit is required for an excavation or fill that satisfies all the following criteria:

1. Is less than 4 feet in vertical depth at its deepest point as measured from the natural ground.

2. Will not result in a total quantity of more than 200 cubic yards of material being removed from, deposited on, or disturbed on any lot, parcel, or subdivision thereof.

3. Will not impair or alter existing surface drainage, constitute a potential erosion hazard, or act as a source of sedimentation to any adjacent land or watercourse. This condition requires that vegetative cover be re-established immediately on all disturbed areas, and to otherwise comply with Section 2.4 concerning surface stabilization measures.

4. Will have no fill placed on a surface having a slope steeper than 5 feet horizontal to 1 foot vertical.

5. Will have no final slopes cut steeper than 3 feet horizontal to 1 foot vertical.

6. Will not contain hazardous substances.

In addition, the following activities do not require a grading permit:

7. Burial within approved human or animal cemeteries.

8. Construction of accessory structures (as defined by the Zoning Ordinance of the City of Huntsville) related to single family dwellings or two-family dwellings (duplex) authorized by a building permit, provided the disturbed material or fill is handled in such a manner as to conform to the approved erosion control plan for the area or, where no such erosion control plan is in effect, that such work is performed in conformance with the basic principles of erosion and sedimentation control as contained in Chapter 10 of this manual.

9. Paving of existing parking areas that cover less than 10,000 square feet and that otherwise satisfy all the requirements for exemption set forth in this Section 2.3.1.

10. Construction and paving of new parking areas less than 2,500 square feet and that otherwise satisfy all the requirements for exemption set forth in this Section 2.3.1.

11. Resurfacing existing paved (impervious) areas.

2.3.2 AGRICULTURAL PRACTICES

No grading permit is required for agricultural land management practices approved by the U.S. Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS), as long as the conditions of Section 2.3.1 are satisfied and existing drainage is not rerouted. Typical accepted agricultural practices include plowing, cultivation, nursery operations such as the removal of or transplanting of cultivated sod and trees, and tree cuttings at or above existing ground level that leave the stump, ground cover, and root intact.

2.3.3 MAINTENANCE GRADING

No grading permit is required for grading as a maintenance measure, or for landscaping on developed lots or parcels with existing structures, provided all of the following criteria are met:

1. The aggregate area affected or stripped at any one time does not exceed 10,000 square feet and is not within a natural drainageway (e.g., designated floodplain).

2. The grade change does not exceed 18 inches at any point and does not alter the direction of the drainage flow path.

3. Proper vegetative cover is re-established as soon as possible on all disturbed areas, and otherwise conforms with Section 2.4 concerning surface stabilization measures.

4. The grading does not involve a quantity of material in excess of 200 cubic yards.

2.3.4 PUBLIC UTILITIES

1. No grading permit is required for installation of lateral sewer lines, telephone lines, cable television lines, electrical lines, gas lines, or other linear public service facilities, whether

publicly or privately owned or operated. This exemption does not include the construction of non-linear public service facilities or infrastructure, whether publicly or privately owned or operated, including, but not limited to, compounds, sites, buildings, huts, transformers, boxes and towers.

2. Agencies exempted from the grading permit requirements of this Section 2.3.4 shall submit documents to the Director of City Engineering for consistency reviews and to allow coordination with other activities.

3. Notwithstanding the foregoing. The exemption in this Section 2.3.4 does not include compliance with the Alabama Construction General NPDES Permit ALR100000. Public utilities project disturbing over 1 acre must apply for and comply with NPDES General Permit ALR100000. Public utilities projects disturbing over 1 acre may not commence until a complete Construction Best Management Practices Plan has been prepared and submitted to the Director of City Engineering and NPDES permit coverage has been issued by ADEM.

2.3.5 DEPARTMENT OF PUBLIC WORKS PROJECTS

1. No grading permit is required for projects performed by or for the City of Huntsville associated with public infrastructure improvements, including but not limited to, facilities maintenance, right-of-way projects, and storm sewer or street construction. This exemption does not include the construction of sites or buildings used by Public Works nor construction projects being performed by a private contractor on behalf of the City of Huntsville. The City of Huntsville shall design, construct and maintain all appropriate public infrastructure projects in accordance with the applicable standards contained in this manual.

2. Notwithstanding the foregoing, the exemption in this Section 2.3.5 does not include compliance with the Alabama Construction General NPDES Permit ALR100000. Public Works project disturbing over 1 acre must apply for and comply with NPDES General Permit ALR100000. Public Works projects disturbing over 1 acre may not commence until a complete Construction Best Management Practices Plan has been prepared and submitted to the Director of City Engineering and NPDES permit coverage has been issued by ADEM.

2.4 SURFACE STABILIZATION MEASURES ON SITES NOT REGULATED UNDER A GRADING PARMIT

2.4.1 DEFINITIONS FOR SECTION

When used in this Section 2.4, the following words, terms, and phrases shall have the following meanings:

Graded site means: (1) a site with grading that is under an active building permit, (2) a site being graded in preparation for construction prior to the issuance of a building permit, or (3) a site with grading where the building permit has expired prior to completion of construction. The term includes public and private easements and rights-of-way located on or immediately adjacent to the site.

Inclement weather period means a prolonged period of heavy rain, high winds, frost or similar weather conditions that render installing permanent stabilization measures unattainable due to such conditions.

Permanent stabilization measures mean maintaining a graded site with a full stand of grass, allowed impervious surface, or other approved permanent stabilization measures.

Person in control means, where there is no active building permit, the owner or person in possession or control of the graded site, and, where there is an active building permit, the holder of a building permit issued for construction on the graded site.

Surface stabilization measures mean maintaining a graded site with either permanent stabilization measures in place or temporary stabilization measures in place, or both.

Temporary stabilization measures mean maintaining a graded site with straw, seed, silt fencing, or other approved temporary stabilization measures.

2.4.1 COVERAGE OF SECTION

This Section 2.4 is intended to govern graded sites that are not being regulated under an active grading permit. Nothing in this Section 2.4 shall be construed to relieve any person from complying with other applicable federal, state, and local laws, including, but not limited to, landscaping requirements under the Zoning Ordinance, foundation drainage requirements under the Building Code, and federal and state environmental regulations, including NPDES permitting requirements.

2.4.2 SURFACE STABILIZATION FOR GRADED SITES

The person in control of a graded site shall maintain, in good condition, order, and repair, surface stabilization measures in place on the site that prevent mud, dirt, water, debris and other construction materials from being washed, blown or otherwise deposited on adjacent public or private property, including, without limitation, public rights-of-way.

2.4.3 SURFACE STABILIZATION PREREQUISTE TO CERTIFICATE OF OCCUPANCY

1. Subject to Subsection 2., as a condition to the issuance of a certificate of occupancy after completion of construction on a graded site, the building permit holder shall have permanent stabilization measures in place on the graded site as may be approved by the Director of City Engineering.

2. During an inclement weather period the building permit holder may apply with the Director for a waiver from the foregoing permanent stabilization requirement. The Director, in his/her sole discretion, may approve the waiver, subject to each of the following conditions:

a. Permanent stabilization measures shall be in place on that portion of the graded site that has public easements and rights-of-way.

b. Temporary stabilization measures shall be in place on the remainder of the graded site.

c. Within three months from the date of the issuance of the certificate of occupancy the applicant for the waiver shall be responsible for having full stabilization measures in place on the site.

2.5 GRADING AND BUILDING PERMIT APPLICATION PROCEDURES

A flow chart for processing of a grading permit application is presented in Figure 2-1. Additional details of application procedures are presented below.

2.5.1 APPLICATION PROCEDURE

Each application for a grading or building permit for which approval of the Director of City Engineering is required shall include the information required under Section 2.5.2 and be submitted for review to the City's E-Plans submittal portal for review and approval. Plans shall be certified by a licensed professional engineer, landscape architect, or land surveyor, as appropriate. Once the plans have been approved for permitting by the appropriate departments, the general contractor shall provide one (1) full size copy of the approved drawings to the Director of City Engineering.

2.5.2 REQUIRED INFORMATION

A complete application package shall include the following items:

1. A complete plan of the proposed development at a scale no less than 1 inch = 100 feet. This plan is to include existing and proposed contours at intervals no greater than one foot with at least one benchmark required, including the elevation relative to North American Vertical Datum of 1988 (NAVD 88). Contours must be drawn showing true elevations referenced to the site benchmark shown on the plans. Contours shall extend to the centerline of all roads/streets bordering the site.

2. Existing and proposed buildings, impervious surfaces (all parking areas will be considered impervious), easements, right-of-way land, and drainage structures, including inlets, catch basins, manholes, junction boxes, driveway pipes, culverts, cross drains, headwalls, outlet facilities, and sanitary sewers, with size, type, slope, invert elevations, and quantity indicated.

3. If a project will be constructed in phases, common facilities and phasing schedules.

4. Hydrologic and hydraulic calculations for appropriate design conditions and facilities, including pollution prevention/detention/retention facilities and reservoir routing calculations as required by Chapter 9 of this manual.

5. Any proposed ditches or channel improvements, with typical section and proposed limits of change indicated.

6. Any high-water marks of flood lines, either calculated or observed in the vicinity of the proposed development, and the source of said line or elevation indicated.

7. All fill areas indicated as such, with the limits and elevation indicated.

8. Drainage arrows indicating the existing and proposed direction of runoff throughout the plan.

9. Locations of existing and proposed locations where stormwater runoff will exit the site (stormwater outfalls).

10. Invert and top of grate elevations on all catch basins and inlets in addition to flow line elevations, stations, and percent grades of all cross drains and pipe between inlets and catch basins.

11. For floodplain areas, designated floodway along with regulatory flood elevations, cut and fill cross sections and calculations, and lowest floor elevations for buildings in the floodplain. Hydraulic calculations should be submitted, when appropriate.

12. An Erosion and Sediment Control Plan (ESC Plan) detailing temporary erosion and sedimentation control measures to be implemented during construction, as required by Chapter 10 of this manual.

13. Final stabilization measures proposed for all disturbed areas on the property. Areas with slopes equal to or greater than two feet horizontal to one-foot vertical shall be stabilized with riprap or another equally effective stabilization measure. Stabilization for each ditch shall be shown.

14. For construction activity that will result in land disturbance equal to or greater than one (1) acre, or from construction activities involving less than one (1) acre and which are part of a common plan of development or sale equal to or greater than one (1) acre, a copy of the Construction Best Management Practices Plan (CBMPP) required by Alabama General NPDES Permit ALR100000 and proof of NPDES permit coverage.

15. A Post-Construction Stormwater Management Plan detailing structural and non-structural post-construction stormwater management and pollution prevention measures developed in accordance with the requirements outlined in Chapter 9 of this manual.

16. Where special structures such as box culverts, bridges, or junction boxes are proposed, detail plans showing dimensions, reinforcement, spacing, sections, elevations, and other pertinent information.

17. Plans and calculations signed and sealed by a registered engineer, landscape architect, or land surveyor, if appropriate, if application is for a grading permit. If application is for a building permit, signature and seal of a registered engineer or architect is required unless otherwise exempted.

Exemptions from the requirement to furnish existing and proposed contours, site benchmark, and the stamp of a registered engineer, architect, or land surveyor, shall be given in following specific case:

1. Construction of, or an addition to, a non-residential structure that has less than 2,500 square feet of floor space and less than 5,000 square feet (including structure) of additional impervious area;

2. Construction of, or an addition to, any single family dwelling or two-family dwelling (duplex), not located in an approved subdivision, of less than 5,000 square feet of impervious area; or

3. Construction of a swimming pool related to single family dwelling or two-family dwelling (duplex), provided that the activities exempted herein satisfy the excavation or fill criteria of Section 2.3.1 (Swimming pools are exempt from the requirements of paragraph (I) and (2) under the excavation or fill criteria of Section 2. 3. 1 except for the requirement under paragraph (2) that no more than 200 cubic yards of material may be deposited on any such lot or parcel without an approved grading plan).

Applications that do not require contour elevations and an on-site benchmark as provided above shall include existing and proposed spot elevations along with a site plan showing distances of any new structures or construction from the property lines.

Additional information required for various facility components is in Chapter 3, Section 3.2.

2.5.3 POSTING

Work requiring a building or grading permit shall not be commenced until the permit holder or his agent has posted the permit in a conspicuous place on the front of the premises. The permit shall be protected from the weather and be placed to allow easy access for recording entries. The building permit shall remain posted by the permit holder until the certificate of occupancy has been issued by the Inspection Department. The grading permit shall remain posted by the permit holder until the work has been approved by the Director of City Engineering.

Work requiring coverage under the Alabama Construction General NPDES Permit ALR100000 shall not be commenced until the information required by the NPDES permit has been posted in a conspicuous place on the front of the premises.

2.5.4 EFFECT

Grading permits issued by the Director of City Engineering and building permits approved by the Director of City Engineering and issued by the City of Huntsville Inspection Department shall be construed to be a license to proceed with the work governed by this manual and shall not be construed as authority to violate, cancel, alter, or set aside any of the provisions of the permit, nor shall issuance of a permit prevent the Director of City Engineering from thereafter requiring a correction of errors in plans, errors in construction or violations of these regulations. A certificate of occupancy shall not be issued until the Director of City Engineering has determined compliance with this manual.

2.5.5 TIME LIMITS

Unless the work governed by this manual and authorized by a grading or building permit is commenced within 6 months after the date the permit was issued, the permit shall become invalid and a new permit shall be required. If the work authorized by such permit is not completed in accordance with approved timing schedules, the permit, with respect to these regulations shall be invalid; however, for just and reasonable cause, one extension for a period not exceeding ninety (90) days may be allowed. Requests for such a time extension shall be submitted in writing to the Director of City Engineering, and authorization for such a time extension shall be in writing. This requirement does not replace, alter, or change any other requirements or regulations of the Inspection Department.

2.5.6 OTHER PERMITS

Approval by the Director of City Engineering does not relieve the applicant of responsibility for obtaining any other required local, state, or federal permits.

2.6 ENFORCEMENT

2.6.1 RIGHT OF ENTRY

The Director of City Engineering, or any duly authorized representatives, may enter upon the premises of any land for which an application for either a grading or building permit has been filed, or said permits issued, for the purpose of inspecting the site before, during, and upon conclusion of construction to or other land disturbance activity to determine compliance with the requirements of this manual and the grading or building permit. The application for and acceptance of a grading or building permit shall be construed as permission by the applicant for entry upon the premises for such inspections.

2.6.2 REVOCATION AND APPEALS

1. Revocation. The Director of City Engineering may revoke an approved grading permit when informed of any false statement or misrepresentation of facts in the application or plans on which the approval is based. The Director of City Engineering may request the revocation of an approved building permit by the Inspection Department when informed

of any false statement or misrepresentation of facts in the application or plans on which approval is based.

2. Appeals. Except for appeals brought pursuant to Article 62 (Flood Hazard District Regulations) of the city's Zoning Ordinance, which shall be governed by the applicable provisions of the Zoning Ordinance, the Director of Urban Development("Urban Development Director"), or such other city official as the Mayor may designate, (hereinafter referred to as "hearing officer") is authorized to hear and decide appeals where it is alleged that there is an error in any requirement, decision or determination made by the Director of City Engineering in the enforcement of this manual. A notice of appeal specifying the grounds therefor must be submitted in writing to the hearing officer within 15 days after such decision by a person aggrieved by the decision. The appeal will be heard within a reasonable time. At least five days prior thereto notice of the hearing will be given in writing to the appellant and Director of City Engineering who shall both have the opportunity to be heard on the matter either in person or through a duly authorized representative. Failure of the appellant to attend the hearing either in person or through a duly authorized representative, shall be deemed to be a withdrawal of the appeal. The decision of the hearing officer is final, subject to such remedy as may be provided at law or in equity.

2.6.3 VERBAL WARNING

Following identification of deficiencies during an inspection, the inspector will issue a Verbal Warning to the owner or operator. Educational materials may also be supplied at this time to facilitate understanding of the stormwater program, regulations, and purpose.

Following issuance of the Verbal Warning:

1. If the deficiencies can be corrected at the time of the inspection, no further action is required.

2. If the deficiencies cannot be corrected at the time of the inspection but could result in a potential illicit discharge, the site will be scheduled for re-inspection. A Notice of Violation may be issued at the inspector's discretion.

3. If the deficiencies have resulted in an illicit discharge of sediment, trash, or other pollutants, a Notice of Violation (NOV) will be issued by the Engineering Division.

Issuance of a Verbal Warning will be noted on the BMP Inspection Form.

A verbal warning shall not be provided in the case of an emergency.

2.6.4 NOTICE OF VIOLATION

A written Notice of Violation (NOV) will be issued by the Engineering Division to the owner or operator of a noncompliant construction site under the following conditions:

1. Construction sites with observed deficiencies that have resulted in a discharge of sediment or other pollutants;

2. Construction sites that have been re-inspected following a Verbal Warning and the deficiencies noted at the time of the Verbal Warning have not been corrected; or

3. A Notice of Violation may be issued at the inspector's discretion to construction sites with observed deficiencies that could potentially result in a non-compliant discharge, but have not yet caused a release of sediment or other pollutants.

The NOV will include the following information, at a minimum:

1. Date and time the deficiencies were observed

2. Description of the identified deficiencies

3. Remediation schedule

4. Re-inspection date

2.6.5 STOP WORK ORDER

Upon notice from the Director of City Engineering, work governed by this manual which is being performed on any site within the City of Huntsville without an approved grading or building permit, in violation of the conditions for an approved grading or building permit, in violation of this manual, or in a dangerous or unsafe manner, shall be stopped immediately. Such notice shall be in writing, shall be delivered to the owner of the property, his agent, or the person doing the work, and shall state the conditions under which the work may be resumed.

2.6.6 CORRECTIVE MEASURES

Any non-permitted drainage system, construction, or fill located within a floodplain shall, upon written notice from the Director of City Engineering, be removed at the property owner's expense. Where any violation is discovered, the responsible party shall restore the land to its state prior to the violation.

2.7 CONSTRUCTION INSPECTIONS

The Director of City Engineering, at his/her discretion, may inspect or cause to be inspected, at various intervals, any or all construction or land disturbance activity for which a grading permit has been issued or a building permit has been approved. The Director of City Engineering may direct that additional testing and inspection be performed. If such additional testing and inspection reveal that the construction and materials conform to the permit requirements, the costs of such additional testing and inspection shall be paid by the City of Huntsville. If such additional testing and inspection reveal that the construction and materials do not conform to the permit requirements, the costs of such additional testing and inspection shall be paid by the permit holder. All nonconforming construction and material shall be removed and replaced with conforming construction and material at no cost or obligation to the City of Huntsville.

2.8 CONSTRUCTION STORMWATER BEST MANAGEMENT PRACTICES INSPECTIONS

The Engineering Division will conduct regular inspections of all construction or grading for which a grading permit has been issued or a building permit has been approved to evaluate erosion and sediment control Best Management Practices (BMPs). Each site will be inspected at least **quarterly** until permit termination or project completion, whichever is later. Construction sites located within Priority Areas, as defined by Alabama Construction NPDES General Permit, will be inspected **monthly**. The inspector will evaluate the following:

1. Condition of the site entrances/exits
2. Effectiveness of erosion controls (e.g., diversion channels, vegetation, outlet protection, etc.)
3. Condition of erosion controls
4. Condition of discharge points from the site
5. Evidence of off-site sediment deposition
6. Condition of sediment controls (e.g., silt fence, wattles, check dams, inlet protection, etc.)
7. Condition of any sediment basins
8. Concrete washout management
9. Fueling area management
10. Sanitary waste management
11. Construction debris and trash management
12. Whether an Erosion and Sediment Control (ESC) plan meeting the City's requirements is present on the site

2.9 AS-BUILT CERTIFICATION

The final step in the permit process under this manual involves as-built certification. As-built certification from a registered engineer, landscape architect, or land surveyor, as appropriate, shall be required and shall include the following items, as appropriate:

1. A letter certifying that the stormwater management system and stormwater control measures (both structural and non-structural), erosion and sedimentation control measures,

and other improvements are complete and functional as required by the appropriate regulations and the approved permit.

2. Record drawings showing the final grades, elevations, stormwater outfalls, and location of facilities and post-construction stormwater management practices. Deviations from the approved plans shall be identified on the record drawings.

3. An inspection report prepared by a registered engineer, landscape architect, or land surveyor, as appropriate verifying proper installation of any post-construction stormwater management practices will become the responsibility of the City to maintain.

4. If as-built grades, elevations, location or size of post construction stormwater controls deviate from the design plans, the Director of Engineering may require calculations documenting the performance of the as-built system.

2.10 ANNUAL INSPECTIONS OF POST-CONSTRUCTION STORMWATER CONTROLS

The Engineering Division will require or conduct inspections of post-construction stormwater controls to confirm that the controls are functioning as designed.

<u>Privately-Owned/Maintained Controls</u>

Owners of post-construction stormwater controls will be required to inspect all owned post-construction stormwater controls annually, at a minimum, and document that the controls are properly maintained and functioning as designed. Inspections will be conducted by individuals familiar with the operation of said controls.

Inspections will be documented and are required to be maintained by the owner/responsible party for a minimum period of three years from the date of inspection or maintenance, and will be made available to the City and/or ADEM upon request.

<u>City-Owned/Maintained Controls</u>

The Engineering Division will inspect City-owned or managed post-construction stormwater controls annually. The City may also inspect non City-owned post-construction stormwater controls periodically at its discretion. Inspection records will be maintained for a minimum period of three years from the date of inspection or maintenance, and will be made available to ADEM upon request.

2.11 OPERATION AND MAINTENANCE AGREEMENT

Owners of post-construction stormwater controls are required to perform, or provide for, long-term maintenance in accordance with the provisions of an agreement between the owner(s) and the City of Huntsville, which agreement may include, but not necessarily be limited to, the Detention Facilities Maintenance Agreement and/or Common Area Maintenance Agreement (hereinafter referred to in this section as “Agreement”). The Agreement will be a condition of final or boundary plat approval and/or grading or building permit approval.

The Agreement is comprised, in part, of the following elements:

1. Inspection and Maintenance provisions, with easement(s) being granted in favor of the City of Huntsville for access, inspection, and/or other performance contemplated in the Agreement.

2. A Long-term Maintenance Plan with a description of each post-construction stormwater control and its components, inspection priorities, the inspection schedule for each component, and a schematic for each stormwater control.

3. Drawings clearly identifying the locations of the stormwater controls.

4. Designation of the owner(s) responsible for compliance with the provisions of the Agreement, including, but not limited to, the stormwater controls; together with the ability to impose assessments and/or liens, including, but not necessarily limited to, those associated with reimbursement of costs incurred by the City of Huntsville if the city performs maintenance responsibilities upon the default of the owner(s).

Should the location, design, or configuration of any approved post-construction control change following final plat approval and/or the issuance of the grading or building permit, the Director of City Engineering may require that the Agreement and/or plan be revised accordingly, and the revised Agreement shall be provided with the as-built drawings.

2.12 AUTHORITY OF DIRECTOR; MODIFICATIONS; ALTERNATIVES

1. Authority. The Director of City Engineering is hereby authorized to administer and enforce the provisions of this manual and to act through duly authorized representatives. The Director of City Engineering shall have the authority to render interpretations of this manual and to adopt policies and procedures in order to clarify the application of its provisions. Such interpretations, policies, and procedures shall be in compliance with the intent and purpose of this manual. Such policies and procedures shall not have the effect of waiving requirements specifically provided for in this manual.

2. Modifications. Where there are practical difficulties involved in carrying out the provisions of this manual, the Director of City Engineering shall have the authority to grant modifications for individual cases, provided that the Director shall first find that special individual reason makes the strict letter of this manual impractical and the modification is in compliance with the intent and purpose of this manual and that such modification does not lessen health, life and safety requirements. In considering whether a modification should be granted, the Director may require the person requesting the modification to provide, at no cost to the city, proof that a modification is warranted. Such proof may include, but not necessarily be limited to, such technical and/or engineering reports, studies, and analysis, duly stamped by a registered engineer, deemed necessary by the Director. The details of action granting modifications shall be recorded and entered in the files of the department. Notwithstanding the foregoing, this provision shall not apply in cases where a variance is being sought pursuant to Article 62 (Food Hazard District Regulations) of the Zoning Ordinance.

3. Alternative materials, design and methods of construction. The provisions of this manual are not intended to preclude the installation of any material or to prohibit any design or method of construction not specifically prescribed by this manual, provided that any such alternative has been approved by the Director of City Engineering. Proposed alternatives shall be evaluated based on compliance with the intent of the provisions of this manual, and prudent technical and engineering standards and practice. In evaluating the alternative, the Director may require that the person proposing the alternative provide, at no costs to the city, proof that the alternative is appropriate. Such proof may include, but not necessarily be limited to, such technical and/or engineering reports, studies, and analysis, duly stamped by a registered engineer, deemed necessary by the director. Where the alternative material, design or method of construction is not approved, the Director of City Engineering shall respond in writing, stating the reasons why the alternative was not approved.

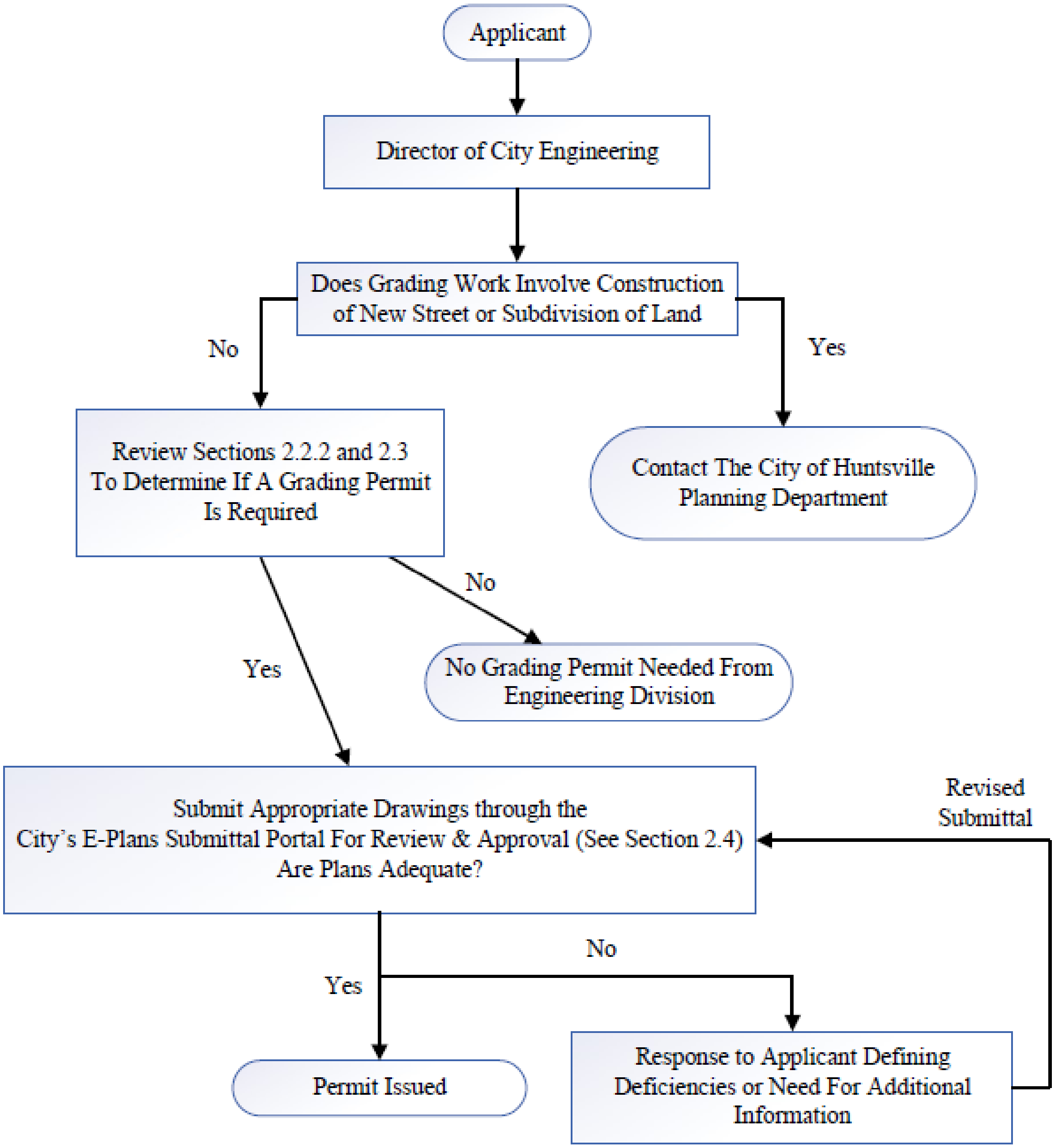

FIGURE 2-1
Grading Permit Application Procedure

CHAPTER 3

REGULATIONS FOR FACILITY DESIGN AND FLOODPLAINS

CHAPTER 3
REGULATIONS FOR FACILITY DESIGN AND FLOODPLAINS

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CHAPTER 3
REGULATIONS FOR FACILITY DESIGN AND FLOODPLAINS

3.1 DESIGN CRITERIA SUMMARY

Design criteria presented throughout the manual are summarized below. Additional details on these criteria can be found in the appropriate chapter.

3.1.1 RETURN PERIODS

Design storm return periods for components of stormwater management facilities shall be as follows:

Facility Component	Return Period	Section
Open Channels	25-year	5.2.1

Check channels, using the 100-year return period, to avoid flooding of existing buildings or exceeding the required minimum floor elevations.

Culverts and Bridges for Roads, Streets and Alleys		
1. Minor and Major Arterials	25-year	6.2.1
2. Major Collectors	25-year	6.2.1
3. Minor Collectors	10-year	6.2.1
4. Residential and Local	10-year	6.2.1
5. Roadside Ditches	10-year	6.2.1

Culvert and bridge capacities should be checked, using the 100-year return period, to ensure that overtopping flood conditions do not exceed 1 foot above the top of the curb.

Pavement Drainage for Roads, Streets and Alleys		
1. Minor and Major Arterials	25-year	7.2.1
2. Major Collectors	25-year	7.2.1
3. Minor Collectors	10-year	7.2.1
4. Residential and Local	10-year	7.2.1

Pavement flooding spread limitations are summarized in Section 7.1.2. In addition, the pavement surface shall be placed above the predicted surface elevation for the 25-year return period flood of nearby streams or channel systems, and rivers. Future flood elevations shall be used where future flood information is available.

Storm Sewers for Roads, Streets and Alleys		
1. Minor and Major Arterials	25-year	8.1.1
2. Major Collectors	25-year	8.1.1
3. Minor Collectors	10-year	8.1.1
4. Residential and Local	10-year	8.1.1

The impact of a 100-year return period flood shall be analyzed considering adjacent property and buildings, emergency access, and public safety.

3.1.2 MANNING'S n VALUES

Manning's n values for components of stormwater management facilities shall be as follows:

Facility Component	Value	Section
Open Channels		
1. Vegetative Class C	See Equa. 5-1	5.2.3
2. Vegetative Class D	See Equa. 5-2	5.2.3
3. Vegetative Class E	See Equa. 5-3	5.2.3
4. Natural Channels	See Tab. 5-5	5.2.3
5. Rigid, Bare Soil, Rock Cut, Temporary, and Riprap	See Tab. 5-3	5.2.3
Culverts		
1. Circular Concrete	0.012	6.2.4
2. Circular Corrugated Metal Pipe (CMP)	0.024	6.2.4
3. Structural Plate CMP	0.0328-0.0302	6.2.4
4. Box Concrete	0.012	6.2.4
5. Oval Concrete	0.012	6.2.4
6. Arch CMP	0.024	6.2.4
7. Structural Plate Arch CMP	0.0327-0.0306	6.2.4
8. Arch Concrete	0.012	6.2.4
Pavement Drainage		
1. Curb and Gutter	0.016	7.1.3
Storm Sewers		
1. Circular Concrete	0.012	8.1.2
2. Box Concrete	0.012	8.1.2
3. Arch	0.012	8.1.2
4. Oval	0.012	8.1.2
5. Circular CMP	0.024	8.1.2
6. Arch CMP	0.024	8.1.2

3.2 SUBMITTAL REQUIREMENTS

All data, drainage computations, and plans shall be submitted to the Director of City Engineering and shall be signed and sealed by a registered professional engineer. If deficiencies are identified by the Director of City Engineering after reviewing data, computations, and plans, additional information shall be submitted, as requested.

3.2.1 HYDROLOGIC PROCEDURES

Hydrologic data and computations used to determine peak discharges and/or flood hydrographs for the design of drainage facilities shall be consistent with the information presented in Chapter 4. The guidelines summarized in Table 4-1 shall be used for selecting appropriate hydrologic procedures. Hydrologic calculations shall be based on existing conditions and development under current zoning classifications within the watershed. Design discharge values available to the Director of City Engineering from other projects or studies may be used in the design and analysis of selected stormwater management facilities, as appropriate.

The following minimum information shall be submitted to the Director of City Engineering:

1. Drainage area maps on a scale of 1 inch = 200 feet or less.

2. Soil maps

3. Runoff coefficients, rainfall intensities, curve numbers, design storms, rainfall excess, overland flow calculations, time of concentration, and other pertinent data in determining peak run-off rates.

4. Peak design discharges, included on plans to allow easy determination of the design discharges for each component of the drainage system or stream; the return period shall be indicated by subscript on each peak design discharge.

5. Calculations, along with hard copies of computer input and output files, as applicable.

6. Post-Construction Stormwater Management Plan consisting of drawings detailing structural and non-structural post-construction stormwater management and pollution prevention measures.

3.2.2 OPEN CHANNELS

This section addresses the regulations for proposed and existing open channels and watercourses within and downstream of the proposed development. The required section and lining material for open channels shall be consistent with the criteria and procedures presented in Chapter 5.

Evaluation of Existing Upstream and Downstream Facilities

Improvements to existing open channels may include, but are not limited to, widening, deepening, paving, grassing, incorporating pollution prevention and erosion control measures, and piping the existing watercourse. Improvements may not be required for existing City maintained open drainage systems if it is determined by hydraulic calculations that the proposed development does not cause an increase in flooding or erosion potential for the design storm return period. The Director of City Engineering may require the submittal of data to support any consideration for exempting existing open channel facilities from improvement.

Existing open drainage courses shall not be altered, except when the following conditions requiring improvements occur:

1. The existing drainage course has insufficient capacity to carry the design flow (see Section 3.1.1 and Chapter 5).

2. The existing drainage course requires rerouting because of the development design and layout.

3. The existing drainage course is erratic in alignment, which may create drainage and maintenance problems.

4. At street crossings, the existing drainage course must be improved as required to tie into the proposed drainage structures.

5. The existing drainage course is contributing to degradation of downstream water quality and pollution prevention measures are necessary to mitigate erosion or stabilize the channel.

Public utility and drainage easements shall be dedicated along existing drainage courses within the proposed development. Off-site public utility and drainage easements shall be required along existing drainage courses. More detailed information is contained in the section on easements found later in this section.

Existing drainage courses that collect and convey stormwater runoff to and from the proposed development shall be analyzed upstream and downstream to ensure that the proposed development does not increase the potential for flooding, erosion, and sedimentation along the drainage course (see Section 3.5). The limits of downstream analysis shall be established by the Director of City Engineering.

Any drainage construction off-site from the proposed development shall be accomplished within dedicated public utility and drainage easements.

<u>Alterations to Existing Facilities</u>

If the drainage course lies within a Floodway or Special Flood Hazard Area (SFHA) as shown on a Flood Insurance Rate Map (FIRM) Madison County, Alabama and Incorporated Areas, any alterations to the drainage course shall comply with all the requirements of the Federal Emergency Management Agency (FEMA) and the City of Huntsville Ordinance No. 87-269, as amended (see Appendix A).

No alteration will be permitted that increases the Regulatory Profile Elevations along the existing drainage course.

Hydraulic Engineering Requirements

The water surface elevation and floodway widths, as determined for a 100-year return period, shall be established along drainage courses to determine minimum floor elevations and floodway boundaries.

Proposed Facilities

The regulations below pertain to proposed open drainage channels and ditches required for transport of stormwater run-off. Existing watercourses and drainage channels that require improvements will also be covered by the following regulations:

1. The proposed open channels shall be sized to carry the design flow (see Section 3.1.1 and Chapter 5).

2. The water surface elevation and floodway widths, as determined for a 100-year return period, shall be established along drainage courses to determine minimum floor elevations and floodway boundaries. See Section 3.4 for additional standards and requirements on drainage courses.

3. Proposed open channels shall be protected from erosion by lining the side slopes and bottom with permanent materials designed to withstand anticipated flow velocities. Acceptable permanent channel lining materials are concrete, riprap, engineered channel armoring products and vegetation (see Chapter 5). Until permanent stabilization is established, a temporary lining of synthetic (e.g. geotextile) or natural (e.g., straw mulch) materials shall be provided. Maximum side slopes are as follows:

Lining Material	Maximum Side
Concrete, Grouted Riprap	1:1
Ungrouted Riprap, Rock Lined, Solid Sod in Non-Residential Subdivision	2:1
Vegetative in non-Residential Subdivision	3:1
Vegetative in Residential Subdivision, Includes Solid Sod	4:1

The lining shall extend upslope a sufficient distance to contain the design storm flow with appropriate freeboard (generally 20 percent of design storm flow depth).

4. Open channel drainage will not be permitted within any residential subdivision nor alongside lot lines within other subdivisions. Underground pipes and facilities shall be provided along the side lot lines and back-easement lines. The Director of City Engineering will consider exceptions to this requirement for the following reasons:

a. The existing or proposed easements will provide the City adequate access to the open channel.

b. The existing or proposed channel requires a 42-inch concrete pipe (or equivalent) or greater to carry the design flow and the channel is designed to prevent erosion and sedimentation problems.

c. The open channel is an existing channel or drainage course located in steep terrain and consisting primarily of natural rock.

5. In addition to those requirements, open channel drainage will not be allowed when the channel meets the following conditions:

a. Channel depth exceeds 12 feet in depth alongside or back lot lines in subdivision development.

b. Channel creates a dangerous situation for the public due to depth and/or location.

c. Channel creates access problems for City maintenance of public utilities.

d. Channel lies in close proximity to existing building structures.

e. Channel is less than 200 feet in length and can be replaced using underground pipe.

f. Channel lies along streets with curb and gutter.

<u>Easements</u>

A utility and drainage easement shall be required along all drainage courses located within a development or subdivision. The easement shall be of sufficient width to contain the entire drainage course as well as providing adequate space for access for maintenance purposes. The easement shall also provide ample width to preserve the natural integrity of the drainage course and to prevent building of structures within areas reserved for the flow of stormwater run-off. Minimum widths shall be as specified below:

1. Vegetative or flexible linings: 10 feet from top of bank on each side.

2. Rigid linings: 5 feet from top of bank on each side

Easements shall be provided of sufficient width within a development or subdivision to allow for future widening and improvements to any drainage course when required by the Director of City Engineering.

Plan Requirements

1. Design discharges along the channel at most upstream points in the subdivision or development and at points when additional pipes or ditches are flowing into the channel or changes in slope occur

2. Channel alignment

3. Channel dimensions and side slopes, and lining material

4. Profile showing flow line, slope, and natural and finished ground elevations at channel centerline

5. Cross section every 50 feet along centerline

6. Structural details

7. Water surface elevations

3.2.3 CULVERTS AND STORM SEWERS

Hydrologic data and computations shall conform to the requirements of Chapters 6 and 8 as appropriate. Hydraulic calculations shall establish capacities for storm sewer and culvert facilities.

Plans submitted to the Director of City Engineering that show proposed storm sewer and culvert facilities shall include the following information:

1. Pipe or culvert size

2. Class of pipe material

3. Pipe length and slope

4. Invert flow line elevations

5. Inlet locations

6. Design discharge

7. Existing drainage facilities

8. Profile drawing indicating finished ground elevations

9. Structural details

All drainage culverts and storm sewers shall be reinforced concrete within all rights-of-way. Class III reinforced concrete pipe shall be used with appropriate bedding requirements, unless the traffic loads dictate use of higher strength pipe. Within easements, double walled traffic bearing plastic pipe is allowable up to 30” diameter with proper bedding and backfill to the manufacturer’s specifications.

Hydraulic grade line data shall be provided. Storm sewer pipe under streets shall extend at least to the back-lot line easement unless otherwise approved by the Director of City Engineering, as allowed in Section 3.2.2.

Plastic pipe that meets ASTM F2881 in sizes from 15” to 30” may be used outside of the right of way.

Installation shall be in accordance with:

1. All pipe systems shall be installed in accordance with ASTM D2321, "standard practice for underground installation of thermoplastic pipe for sewers and other gravity flow applications", latest addition, with the exception that the initial backfill may extend to the crown of the pipe. soil classifications are per the latest version of ASTM D2321. Class ivb materials (mh, ch) as defined in previous versions of ASTM D2321 are not appropriate backfill materials.

2. Measures should be taken to prevent migration of native fines into backfill material, when required.

3. Foundation: where the trench bottom is unstable, the contractor shall excavate to a depth required by the Engineer and replace with suitable material as specified by the Engineer. As an alternative and at the discretion of the design engineer, the trench bottom may be stabilized using a geotextile material.

4. Bedding: Suitable material shall be Class I, II, III or IV. The contractor shall provide documentation for material specification to the Engineer. Compaction shall be specified by the Engineer in accordance with Table 3 for the applicable fill heights listed. Unless otherwise noted by the Engineer, minimum bedding thickness shall be 4” for 15”-24” diameter pipe; 6” for 30” diameter pipe. The middle 1/3 beneath the pipe invert shall be loosely placed. Please note, Class IV material has limited application and can be difficult to place and compact; use only with the approval of a soil expert.

5. Initial Backfill: Suitable material shall be Class I, II, III, or IV in the pipe zone extending to the crown of the pipe. The contractor shall provide documentation for material specification to the Engineer. Material shall be installed as required in ASTM D2321, latest edition. Compaction shall be specified by the Engineer in accordance with Table 3 for the applicable fill heights listed. Please note, Class IV material has limited application and can be difficult to place and compact; use only with the approval of a soil expert.

6. Minimum Cover: Minimum cover, H, in non-traffic applications (grass or landscape areas) is 12” (300MM) from the top of the pipe to ground surface. Additional cover may be required to prevent flotation. For traffic applications; Class I or II material compacted to 90% SPD and Class II compacted to 95% SPD is required. For traffic applications,

minimum cover, H, is 15" up to 30" diameter pipe, measured from top of pipe to bottom of flexible pavement or top of rigid pavement.

3.2.4 GUTTERS AND INLETS

Gutter and inlet capacities shall be determined when new streets are designed. The gutter and inlet hydraulics of streets and roadways shall be consistent with the criteria and procedures presented in Chapter 7.

Inlets shall be placed so that at least 80 percent of the gutter flow is intercepted (i.e., maximum bypass is 20 percent). The maximum length of longitudinal curb and gutter section without a "Type S" inlet shall be 300 feet. Calculation, as detailed in Chapter 7, shall be made to ensure that new development and construction that increases run-off to existing streets does not violate maximum spread limitations, as contained in Section 7.1.2. If the existing spread on the street equals or exceeds the maximum allowed before considering the proposed development, the run-off from the new development shall be transported by pipe to an existing storm sewer system.

The following information shall be provided in the plans:

1. Design flow along the curb and gutter

2. Inlet locations, intercepted flow, and bypass flow

3. Street profiles and typical sections

3.2.5 STORAGE AND TREATMENT SYSTEMS

The standard for determining the requirement for storage systems shall be that the calculated peak rate of stormwater runoff shall be no greater after development of a site than before development of a site. The calculations shall be made considering a ten-year return period, twenty-four hour duration, type II storm distribution as defined by the Natural Resources Conservation Service (NRCS), U.S. Department of Agriculture (USDA), for both before and after development site conditions.

The standard for determining the water quality treatment volume shall be based on the runoff from a 1.1-inch, 24-hour storm, preceded by a 72-hour antecedent dry period. Stormwater quality protection shall be provided to manage 100% of the treatment volume on-site.

Design strategies to reduce post-construction runoff rate and/or provide water quality treatment may include, but are not limited to:

1. Minimizing impervious surfaces
2. Providing vegetated buffers
3. Detention ponds
4. Retention ponds
5. Bioretention
6. Constructed stormwater wetlands

7. Grassed swales, infiltration swales, and wet swales
8. Rain gardens
9. Rainwater harvesting
10. Permeable pavement

Selection and design guidance for Low Impact Development (LID) practices is provided in the most recent *Low Impact Development Handbook for the State of Alabama* (Alabama LID Handbook).

The hydraulics and design of post-construction stormwater controls shall be consistent with the criteria and procedures presented in Chapter 9. Storage systems will be required to limit the adverse impacts of increased runoff.

Exemptions from the requirement to provide on-site storage systems may be considered in the following specific cases:

1. Commercial or industrial development which adds less than 10,000 square feet of impervious surface area.

2. Any development which results in less than a 2.5 cubic foot per second increase in the ten-year storm peak discharge from the developed site, provided that the Director of City Engineering may require an analysis of the downstream drainage facilities to determine hydraulic capacities of affected drainage systems. The Director of City Engineering will require an analysis of the downstream system when the following conditions exist:

   a. There is a history of flooding in the vicinity of the proposed development site, or there is evidence of undersized drainage facilities, or

   b. The Director of City Engineering has inspected the downstream drainage facilities and needs additional information to determine the capacities of those facilities

3. Any development or construction where, based on a competent engineering study prepared by a registered professional engineer, satisfies all the following criteria:

   a. The standard requirement in Section 2.1.1(3) is satisfied and particularly that the increase in runoff and peak discharge from the development shall not increase the degree (e.g. frequency and/or depth) of flooding in its vicinity or in other areas;

   b. The increase in runoff and peak discharge from the development shall not cause an increase in erosion or sedimentation;

   c. All downstream drainage systems and facilities are adequate to handle the increase in runoff and peak discharge from the development considering total development of the watershed. The design storm used for determining the increase in runoff and peak discharge will be in conformance with the design criteria in Chapter 3.

   If paragraph 3(a) and 3(b), are met and 3(c) is not met, then the Director of City Engineering will determine the allowable increase in peak discharge from the proposed development based on the following equation:

$$Qa = \frac{Qc - Qp}{At} (Ap)$$

Qa = allowable increase in peak discharge, cubic feet per second

Qc = hydraulic capacity of downstream system, cubic feet per second

Qp = present condition (pre- development) peak discharge, cubic feet per second

At = total watershed drainage area, acres

Ap = drainage area of proposed development, acres

4. Construction, or repair of, or addition to, any residential dwelling located on single lots within an approved subdivision.

5. Construction, or repair of, or addition to, any commercial or industrial buildings or facilities including parking areas located within an approved subdivision where an existing stormwater detention or retention facility has been designed and constructed to meet the requirements of this manual.

Requests for exemptions from the requirement to provide stormwater quality protection for 100% of the treatment volume shall be considered by the Director of City Engineering on a case by case basis upon submittal of a written request outlining reasons for the request, site specific pre and post development runoff conditions, and including alternatives considered to incorporate stormwater quality protection to the maximum extent practicable (MEP).

Storage or treatment systems that are not required may be allowed, provided such facilities have no detrimental effect on upstream, downstream, and adjacent drainage facilities and further provided that the calculated peak rate of stormwater runoff shall be no greater after development than before development.

Post-Construction Stormwater Management Plan

A Post-Construction Stormwater Management Plan must be approved by the Director of City Engineering for the following activities unless exempted. Development of a Post-Construction Stormwater Management Plan shall be consistent with criteria and procedures presented in Chapter 9.

1. Any activity requiring a grading permit shall include a Post-Construction Stormwater Management Plan.

2. Plans submitted to the Director of City Engineering for approval in the building permit process shall include a Post-Construction Stormwater Management Plan, unless no land disturbance activity is associated with the building.

3. All subdivision plans submitted to the Director of City Engineering for approval shall include a Post-Construction Stormwater Management Plan.

The following information shall be provided in the Post-Construction Stormwater Management Plan, as applicable:

1. Pre-development and post-development design flood hydrographs for the 100-year and 10-year storm return periods

2. Stage-storage data

3. Stage-discharge data

4. Construction details

5. Design criteria and calculations for sizing of stormwater controls and treatment practices

6. Structural details for spillway and outlet facilities

7. Public utility and drainage easements of adequate width shall be required to provide access to the facility for maintenance activities.

   A Long-Term Maintenance Plan approved by the Director of City Engineering will be required for any post-construction stormwater control.

3.2.6 EROSION AND SEDIMENT CONTROL

An erosion and sediment control plan must be approved by the Director of City Engineering for all land alteration activities unless exempted (see Section 2.3). Any activity requiring a grading permit shall include an erosion and sediment control plan. Development of an erosion and sediment control plan shall be consistent with criteria and procedures presented in Chapter 10.

Plans submitted to the Director of City Engineering for approval in the building permit process shall include an erosion and sediment control plan, unless no land disturbance activity is associated with the building.

All subdivision plans submitted to the Director of City Engineering for approval shall include an erosion and sediment control plan.

The Alabama Construction General NPDES Permit ALR100000 (Alabama CGP) applies to all construction activity that will result in land disturbance equal to or greater than one (1) acre, or from construction activities involving less than one (1) acre and which are part of a common plan of development or sale equal to or greater than one (1) acre. For sites requiring coverage under the Alabama CGP, a copy of the Construction Best Management Practices Plan (CBMPP) and proof of NPDES permit coverage must be submitted to the Director of City Engineering.

Erosion and sediment control plans are not required for single residential or commercial lots less than 1 acre in size and within subdivision development. This does not relieve the builder from controlling the erosion of dirt, rock, debris, and building materials onto public or private property and into drainage systems and streets. The builder or contractor is also not relieved from any fines or penalties that may be issued against him for violation of City ordinances restriction deposits of debris on any street, alley, or public grounds within the City (Section 20, City of Huntsville Codes of Ordinances).

The erosion and sediment control plan shall include a time schedule detailing the sequence of erosion and sediment control measures along with the sequence of construction. The components of an approved erosion and sediment control plan shall be implemented and maintained to ensure that the control measures are functioning as intended.

The Director of City Engineering shall require the following information on the erosion and sediment control plans:

1. A complete plan of the proposed development at a scale no less than 1 inch = 100 feet. This plan is to include existing and proposed contours at intervals no greater than 2 feet, with at least one benchmark required.

2. Existing and proposed buildings, impervious surfaces, and drainage structures, including inlets, catch basins, manholes, junction boxes, driveway pipes, culverts, cross drains, headwalls, outlet facilities, and sanitary sewers.

3. Any proposed drainage improvements such as channels, ditches, drainage pipes, and structures.

4. All fill areas indicated as such, with the limits and elevation indicated.

5. Drainage arrows indicating the existing and proposed direction of runoff throughout the plan.

6. Existing and proposed locations where stormwater runoff will exit the site (stormwater outfalls) in State Plane compatible with City of Huntsville GIS standards.

7. Temporary erosion and sediment control measures to be implemented during construction, including an implementation schedule.

8. Locations of construction entrances and exits, as well as temporary access drives or haul roads.

9. Locations of designated fueling areas and concrete washout areas, and the associated containment measures.

10. Measures the builder or contractor will use to keep construction materials, including dirt, mud and debris, off the streets.

11. Locations of proposed vegetative buffers and vegetated areas to be preserved, as well as methods to delineate such areas during construction.

12. Proposed construction phasing, with total disturbance for each phase clearly identified in acres.

3.3 FLOODPLAIN DATA

3.3.1 FEMA FLOOD INSURANCE RATE MAPS

Reference Aug. 16, 2018 FEMA Flood Insurance Studies for Madison County, Alabama and Incorporated Areas (number 01089CV001C), Limestone County, Alabama and Incorporated Areas (number 01083CV001C), and Morgan County, Alabama and Incorporated Areas (number 01103CV001B) and their referenced/associated Flood Insurance Rate Maps (FIRMs) and any subsequent Effective revisions for Floodway and Floodway Fringe (1% Annual Chance Floodplain/Special Flood Hazard Area boundaries) District boundaries, related waterways, peak discharges, Base Flood Elevation (BFE) data, etc.

3.3.2 UNPUBLISHED DATA

If a project is located in a Zone A on the FIRMs or areas not covered by the FIRMs, the applicant shall provide base flood elevation (BFE) and floodway data as documented in a Flood Study/Report when the project is greater than 5 acres or 50 Lots, whichever is less.

In addition, a Flood Study shall be required for projects if they contain, intersect, or are along an unstudied (in regards to FEMA) drainage conveyance with a drainage area of 1 square mile or greater. At the discretion of the Director of City Engineering, a Flood Study may be similarly required for drainage areas 0.5 square miles or greater.

Amongst other requirements of the Director of City Engineer, the Flood Study shall include required items for a FEMA Letter of Map Revision (LOMR) application and be reviewed by the Director of City Engineering. Following any required revisions as a result of the review, the Flood Study shall be processed for adoption as regulatory floodplain management data by the subject developer/property owner(s) through the appropriate FEMA application processes.

If the Director of City Engineering does not require a Flood Study for a drainage area of 0.5 square miles or greater but less than 1 square mile as discussed above, then the project will be considered flood prone. Subsequently at the discretion of qualified City Engineering staff, structures will be required to have their lowest finished floor elevated on fill upon building permitting. Alternately at said staff discretion, elevation on a crawl space appropriately vented for flooding may be allowed.

3.3.3 PUBLISHED FLOODWAY DATA

If a project proposes changes in/encroachment on a FEMA published floodway then a Flood Study as discussed in Section 3.3.2 shall be required. If the study demonstrates an adverse impact on flooding conditions, as determined by the Director of City Engineering, then the developer/property owner(s) shall be required to submit and receive approval of a Conditional Letter of Map Revision (CLOMR) through FEMA and post construction a follow-up LOMR.

3.4 ZONING ORDINANCE

Uses permitted within the floodplain shall be in accordance with Article 62 of the 1989 Huntsville Zoning Ordinance, as amended from time to time (the Zoning Ordinance) . The regulations and controls set forth shall be applied within the areas designated by the Zoning Ordinance. However, nothing shall prohibit the application of the Zoning Ordinance to lands that can be demonstrated by general accepted engineering methods as determined by the Director of City Engineering to lie within any floodplain/flood prone area.

Should conflict exist between this document and the Zoning Ordinance, the more restrictive of the two, as determined by the Director of City Engineering, shall apply.

Similarly, should conflict exist between the current Code of Federal Regulations (CFR) 44, which contains the FEMA National Flood Insurance Program (NFIP) minimum requirements, and this document and/or the Zoning Ordinance, the more restrictive, as interpreted by the Director of City Engineering, shall apply.

3.5 INADEQUATE FACILITIES

If it is determined that any proposed development has inadequate storm sewer facilities downstream of the proposed development outside its property lines, the Director of City Engineering shall not approve the development until the problem is resolved. “Inadequate storm sewer facilities” shall mean those insufficient to carry the design flow and meet the design criteria in this manual and those not located within public utility and drainage easements. Storm sewer facilities shall include pipes, culverts, ditches, channels, watercourses, and retention/detention ponds.

3.6 TRAFFIC-SAFE DRAINAGE FACILITIES

The City of Huntsville recognizes that design standards change constantly and become more efficient and safer with time. The standards contained herein have been adopted to ensure that construction of new roadway, reconstruction of existing roadway, and construction of driveway turnout to existing roadways is accomplished in a manner that maximizes motorists’ safety at minimum additional cost.

Clear zone distance data from the American Association of State Highway Transportation Officials AASHTO Roadside Design Guide, Current Edition

Parallel and transverse pipes and culverts and drop inlets are common drainage system elements that shall be designed, constructed, and maintained with both hydraulic efficiency and roadside safety in mind.

In general, the following options, listed in order of preference, are applicable to all drainage features:

1. Eliminate non-essential drainage structures.

2. Design or modify drainage structures so they are traversable or present a minimal hazard to an errant vehicle.

3. If a major drainage feature cannot be redesigned or relocated, shield it by a suitable traffic barrier if it is in a vulnerable location.

The rest of this section identifies the safety problems associated with pipes, culverts, and drop inlets and offers recommendation concerning the location and design of these features to improve their safety characteristics without adversely affecting their hydraulic capabilities. The information presented applies to all roadway types and projects. Typical construction details for providing traffic-safe drainage facilities are contained in the standard drawings (Appendix 3).

3.6.1 CROSS-DRAINAGE STRUCTURES

Cross-drainage structures are designed to carry water underneath the roadway embankment and vary in size from 15-inch reinforced concrete pipe to multi-barreled concrete box culverts. All cross drains shall be designed to carry at a minimum the 25 year storm event. The potential hazard is either a fixed object protruding above an otherwise traversable embankment or an opening into which a vehicle can drop. The design engineer shall be required to extend cross-drain structures to minimize roadside hazards within the right-of-way with one of the following methods:

1. Using a traversable design

2. Extending the structure so it is less likely to be hit

3. Shielding the structure

Each of these options are discussed below.

Traversable Designs

The design engineer should attempt to provide embankments as smooth or traversable as practical for a given facility. Traversable, not-recoverable slopes should be rounded at top and bottom and may provide a relatively flat runout area at the bottom. If a slope is generally traversable, the preferred treatment for any cross-drainage structure is to extend (or shorten) it to intercept the roadway embankment slope. For small culverts, no other treatment is required.

Single structures and end treatments wider than 36 inches can be traversable for passenger size vehicles by using bar grates or pipes to reduce the clear opening width. Figure 3-1 shows recommended sizes to support a full-size automobile and is based on a 30-inch bar spacing. It is important to note that the toe of the embankment slope and the ditch or streambed area immediately adjacent to the culvert must be fairly traversable if the use of a grate is to have any significant safety benefits. Normally, grading within the right-of-way limits can produce a satisfactory runout path.

For median drainage where flood debris is not a concern and mowing operations are frequently required, much smaller openings between bars can be tolerated. Figure 3-2 shows a median drainage inlet design that can be used where cover is insufficient to construct a drop inlet directly across the cross-drainage pipe.

Extension of Structures

For intermediate sized pipes and culverts for which inlets and outlets cannot readily be made traversable, the structure can be extended at or just beyond the appropriate clear zone, as defined by the required typical section. However, the extended culvert headwall may become the only significant man-made fixed object hazard immediately at the edge of the clear zone along the section of roadway under design. In that case and if the roadside is generally traversable to the right-of-way line elsewhere, extending the culvert to the edge of the clear zone may not be the best alternative. This is particularly true on freeways and other high speed, controlled access facilities. Conversely, if the roadway has numerous fixed objects, both natural and man-made, at the edge of the clear zone, extending individual structures to the same minimum distance from traffic may be appropriate. However, the preferred safety treatment is usually to redesign the inlet/outlet so it is no longer a fixed object hazard.

Shielding

Major drainage structures are costly to extend and often have end sections that cannot be made traversable. In such cases, shielding with an appropriate traffic barrier is other the most effective safety treatment. The traffic barrier will be longer and closer to the roadway than the structure opening and more likely to be hit than a shielded culvert located further from the traveled way. Nevertheless, a properly designed, installed, and maintained barrier system may provide an increased level of safety for errant motorists.

3.6.2 PARALLEL DRAINAGE FEATURES

Parallel or side drain culverts are typically used under driveways, field entrances, access ramps, intersecting side roads, and median crossovers. As with cross-drainage structures, the designer's primary concern should be to design generally traversable slopes and to match the culvert opening with adjacent slopes. Embankment slopes that can be struck at 90 degrees by vehicles run off the road should be constructed with 6:1 slopes or flatter for locations susceptible to high speed impacts (speed limit areas of 40 miles per hour or greater). On low volume roads with low speed limits, a 3:1 slope or flatter shall be used. With these guidelines, safety treatment options are similar to those for cross-drainage structures. The design engineer involved in the design parallel drainage features shall minimize the hazard of the drainage structures by using one of the following methods:

1. Eliminate the structure.

2. Use a traversable design.

3. Move the structures laterally to a less vulnerable location.

4. Shield the structure.

Eliminate the Structure

Unlike cross-drainage pipes and culverts, which are essential for proper drainage and operation of a road or street, parallel pipes can sometimes be eliminated constructing an overflow section on the field entrance, driveway, intersecting side road, or median crossover. This treatment will be appropriate only at low volume locations where it does not decrease the sight distance available to drivers entering the main road.

Traversable Designs

Embankments shall be designed with consideration given to their effect on the roadside environment. Parallel drainage structures shall match the selected side slope and shall be made traversable when they are located in a vulnerable position relative to main road traffic. The addition of pipes and bars parallel to traffic can prevent a wheel from catching in the culvert opening. The bottom bar or pipe should be set a maximum of 4 inches above the culvert invert. A possible traversable design for a parallel culvert is shown in Figure 3-3. When ditch grades permit, the inlet end may use a drop inlet type design to reduce the length of grate required.

In locations where headwater depth is limited, a larger pipe should be used or the parallel drainage structure may be positioned outside the clear zone.

Relocate the Structure

Some parallel drainage structures can be moved laterally further from the traveled way. This approach often allows the transverse embankment slope within the selected clear zone to be flattened. If the embankment at the new culvert locations in traversable and likely to be encroached upon by wither main road or side road traffic, safety treatment will be applied. The inlet or outlet shall match the embankment slope.

Shielding

In cases where the embankment cannot be made traversable or the structure is too large to be effectively safety treated and it cannot be relocated, it may be necessary to shield the hazard with a traffic barrier. Specific information is found in AASHTO Roadside Design Guidance, current edition.

3.6.3 DROP INLETS

Drop inlets can be classified as on roadway or off roadway structures. On-roadway inlets are usually located on or alongside the shoulder of a street or highway and include curb-opening inlets, grated inlets, slotted drain inlets, or combinations. Since on-roadway inlets are installed flush with the pavement surface, they must be capable of supporting vehicle wheel loads and present no hazard to pedestrians or bicyclists.

Off-roadway drop inlets are used in medians of divided roadways and sometimes in roadside ditches. They shall be designed and located to present a minimal obstacle to errant motorists. This can be accomplished by building them flush with the ditch bottom or slope on which they are located. No portion of the design shall project high enough to cause vehicular snagging, obstruction, or instability. In locations where off roadway drop inlets are raised to facilitate stormwater management, they shall be placed in the slope or shielded by a suitable traffic barrier.

The opening shall be treated to prevent a vehicle wheel from dropping into it but unless pedestrians are consideration, grates with openings as small as those used for pavement drainage are not necessary. Neither is it necessary to design for a smooth ride over the inlet. It is sufficient to prevent wheel snagging and the resulting sudden deceleration or loss of control.

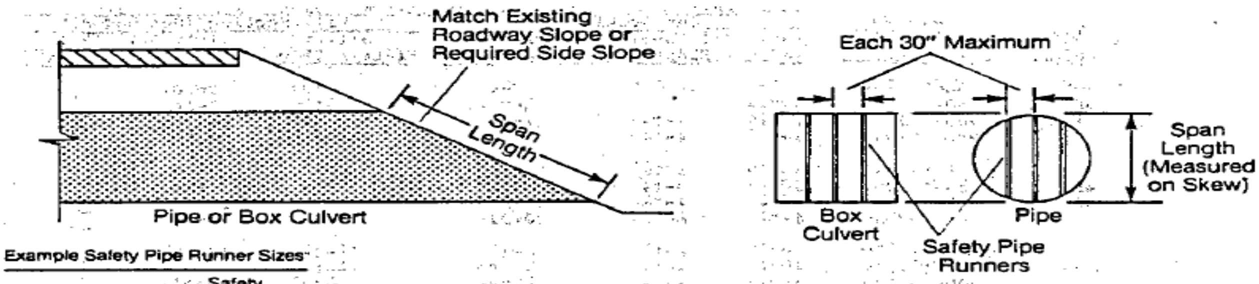
Table 3-1
CLEAR ZONE DISTANCE DATA

Speed (mph)	Design Average Daily Traffic	Clear Zone Distances (in feet from edge of traveled way) Cut Section: 3:1 Slope	Cut Section: 4:1 to 5:1 Slope	Cut Section: 6:1 or Flatter Slope	Fill Section (Unrounded): 6:1 or Flatter Slope	Fill Section (Unrounded): 4:1 to 5:1 Slope	Fill Section (Unrounded): 3:1 Slope[a]
40 or Less	Under 750	7-8	7-8	7-8	7-8	8-10	—
	750-1,000	10-12	10-12	10-12	10-12	12-14	—
	1,500-6,000	12-14	12-14	12-14	12-14	14-16	—
	Over 6,000	14-16	14-16	14-16	14-16	16-18	—
45-50	Under 750	8-10	8-10	10-12	10-12	12-14	—
	750-1,000	10-12	12-14	14-16	12-14	16-20	—
	1,500-6,000	12-14	14-16	16-18	16-18	20-24	—
	Over 6,000	14-16	18-20	20-22	18-20	24-28	—
55	Under 750	8-10	10-12	10-12	12-14	14-18	—
	750-1,000	10-12	14-16	16-18	16-18	20-24	—
	1,500-6,000	14-16	16-18	20-22	20-22	24-30	—
	Over 6,000	16-18	20-22	22-24	22-26	26-32	—

Reference: American Association of State Highway Transportation Officials (1988).

Note: Clear zone is defined as a roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. The clear zone should not be viewed as a discrete, exact distance, but as the center of a zone that should then be analyzed on a site-specific basis.

[a]Since recovery is less likely on the unshielded, transversable 3:1 slopes, fixed objects should not be present in the vicinity of the toe of those slopes.



Example Safety Pipe Runner Sizes

Span Length (feet)	Safety Pipe Diameter (inches)
up to 12	3.0
12 to 16	3.5
16 to 20	4.0
20 or less with center support	3.0

FIGURE 3-1
Recommended Cross-Drain Safety Pipe Runner Sizes for Various Span Lengths

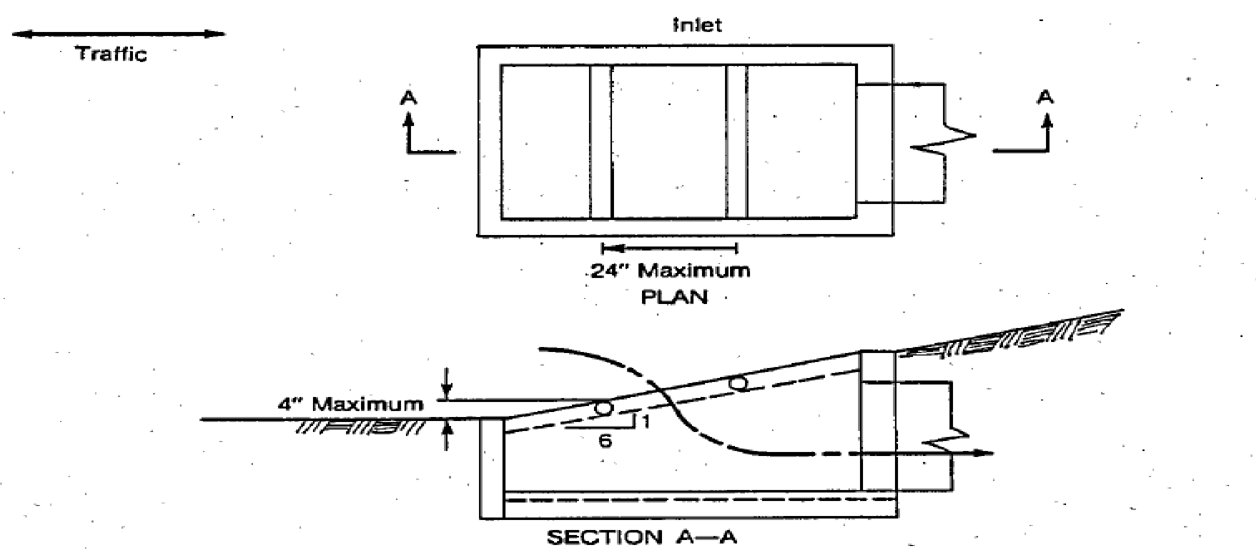
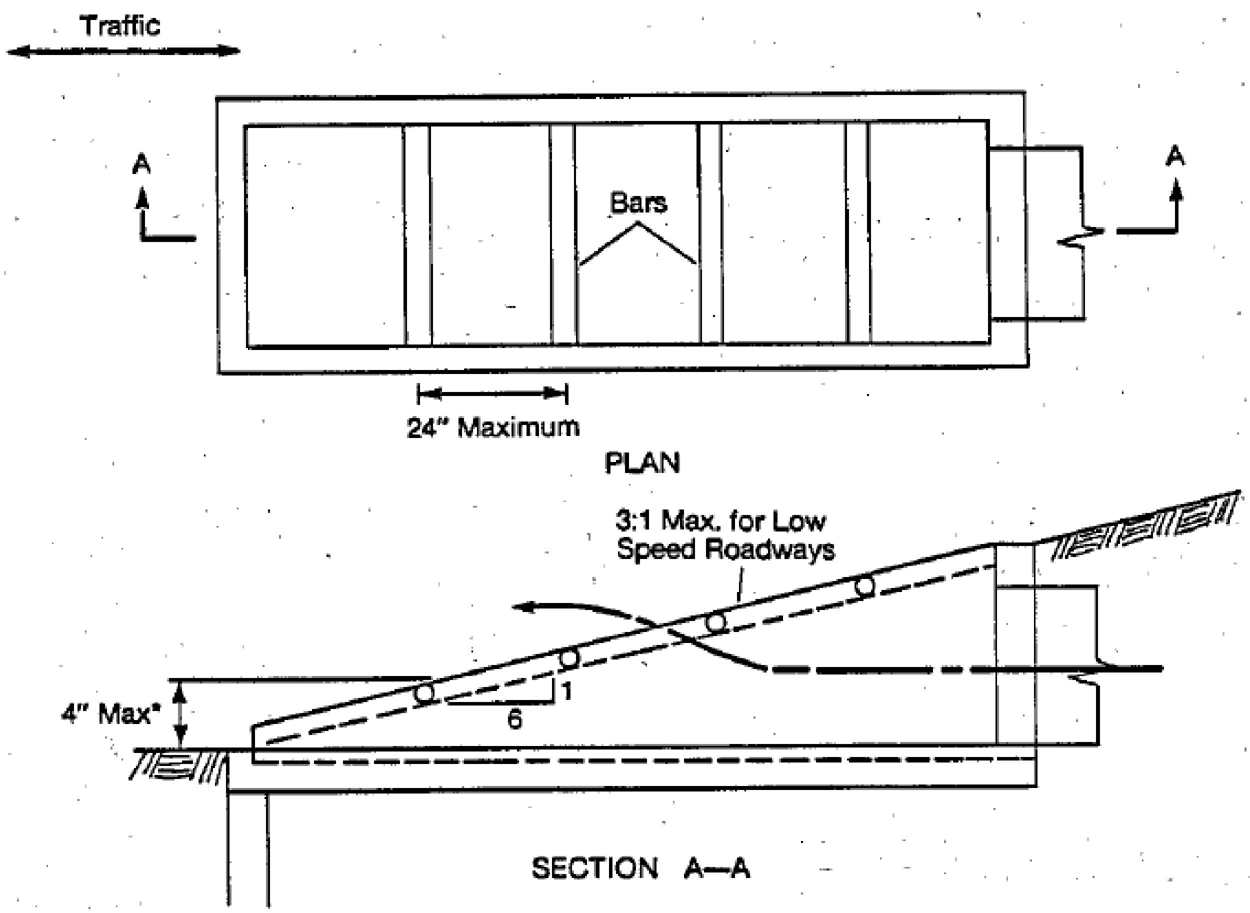


FIGURE 3-2
Transversable Median Cross Drains for Insufficient Cover



*4" max. applies to both inlet and outlet on two way roads and only to the side facing traffic on divided highways.

FIGURE 3-3

Inlet/Outlet Transversable Example for Parallel Drainage

PART 2
TECHNICAL INFORMATION

PART 2 – TECHNICAL INFORMATION

Part 2 of the Huntsville Stormwater Management Manual is oriented to the technical concerns of designers and builders. The following chapters are included in Part 2:

4. Hydrology
5. Open Channel Hydraulics
6. Culvert Hydraulics
7. Gutter and Inlet Hydraulics
8. Storm Sewer Hydraulics
9. Post-Construction Stormwater Management
10. Erosion and Sediment Control

CHAPTER 4
HYDROLOGY

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CHAPTER 4
HYDROLOGY

4.1 PROCEDURE SELECTION

The guidelines in Table 4-1 shall be followed for selecting hydrologic procedures. A consideration of peak runoff rates for design conditions is generally adequate for conveyance systems such as storm sewers or open channels. However, if the design must include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is usually required.

Because streamflow measurements for determining peak runoff rates for pre-project conditions are generally not available, accepted practice is to perform flood hydrology calculations using several methods. Results can then be compared (not averaged), and the method that best reflects project conditions selected and documented.

The Rational Method (see Section 4.5.2) is subject to the following limitations:

1. Estimates of peak design flows only.
2. Time of concentration, tc, of 5 minutes $\leq$ tc $\leq$ 30 minutes.
3. Drainage area, DA $\leq$ 50 acres.

In addition, results should be compared using other methods, and approval by the Director of City Engineering is required.

The SCS TR-55 (1986) graphical method (see Section 4.5.5) is subject to the following limitations:

1. Estimates of peak design flows only.
2. Design storm = SCS Type II 24-hour distribution.
3. Time of concentration, tc, of 0.1 hour $\leq$ tc $\leq$ 10 hours.
4. The method was developed from results of computer analyses performed using TR-20 (USDA, SCS, 1983) for a 1-square mile homogeneous (describable by one CN value) watershed.
5. Curve number, CN, of 40 $\leq$ CN $\leq$ 98.
6. Ratio of initial abstraction to precipitation, Ia/P, of 0.1 $\leq$ Ia/P $\leq$ 0.5.
7. Unit hydrograph shape factor of 484.

8. Only one mainstream channel in the watershed or, if more than one exists, nearly equal times of concentration for the branches.

9. Use of the current version of WinTR-55DB.

10. No consideration of hydrologic channel routing.

The SCS TR-55 (1986) tabular method (see Section 4.6.5) can be used to estimate flood hydrographs and to approximate the effects of hydrologic channel routing, subject to the following limitation:

1. Design storm = SCS Type II 24-hour distribution.

2. Time of concentration, tc, of 0.1 hour $\leq$ tc $\leq$ 2 hours.

3. DAs of individual subareas do not differ by a factor of 5 or more. The procedure was developed for DA of 1 square mile.

4. Curve number, CN, of 40 $\leq$ CN $\leq$ 98.

5. Ratio of initial abstraction to precipitation, Ia/P, of 0.1 $\leq$ Ia/P $\leq$ 0.5.

6. Unit hydrograph shape factor = 484.

7. Reach travel time, t_T, of 0 to 3 hours.

8. Use of the 1986 version of TR-55 in place of the 1975 procedures.

TVA regression equations (see Section 4.5.3) have been prepared for small, ungagged, urban watersheds in Huntsville. The TVA regression equations are subject to the following limitations:

1. Estimates of peak design flows only.

2. DA between 60 and 3,200 acres.

3. Imperviousness between 3 to 50 percent.

4. Inapplicable when extensive channelization or regulation is incorporated into the watershed.

TVA graphs (see Section 4.5.4) provide flow frequency estimates for modeled streams that are applicable only to those stream reaches that were included in the modeling.

Unit hydrograph theory (see Section 4.6.2 and 4.6.3) provides a generally applicable procedure for developing flood hydrographs using a basin-specific unit hydrograph and an appropriate rainfall hyetograph. Many computer models use unit hydrograph theory. With careful development of a basin-specific unit hydrograph, this versatile method can be adapted to a wide range of conditions.

Computer modeling is appropriate when limitation of the simpler methods are exceeded, complex situations are being studied, or more detailed information is required.

4.2 HYETOGRAPHS

The rainfall data in Section 4.2.1 represent average intensity or volume over a particular duration. When a flood hydrograph is to be developed, however, a time variable distribution or hyetograph is required. Two methods are described in Sections 4.2.2 and 4.2.3 for constructing the design storm hyetograph: the SCS Type II storm distribution (24-hour duration) and the balanced storm approach (any duration).

4.2.1 DESIGN STORM DATA

Hydrologic design storm data should be collected by the following procedure:

1. Select an appropriate procedure for hydrologic calculations using information in Section 4.1.

2. Determine the type of precipitation data that are needed. Generally, either intensity-duration-frequency (IDF) curves or hyetographs for historic or design storm conditions are used.

IDF curves and data for Huntsville are available in the NOAA Atlas 14. Use the latest version available.

Huntsville area climatological data are collected by the Nation Weather Service Office (WSO). The WSO at the Huntsville/Madison County Airport, Carl T. jones Field, is located 11 miles southwest of the center of Huntsville. All official weather reports and data for Huntsville currently come from Jones Field.

The National Climatic Data Center evaluates and summaries collected climatological data on a daily, monthly, and annual basis. Publications summarizing the climatological data collected are available at a small fee to the general public from the U.S. Department of Commerce, National Climatic Data Center, Federal Building, Asheville, North Carolina 28801, or from WSO at Jones Field.

4.2.2 SCS TYPE II STORM

The SCS has developed a set of 24-hour storm distributions representing average conditions for different regions of the United States. The Type II storm is appropriate for Huntsville and its cumulative mass curve is presented in

Figure 4-2. The curve is applicable to the 24-hour rainfall depth for any recurrence interval.

The SCS dimensionless hyetograph approach takes the following steps:

1. Select a design storm time step and return period.

2. Obtain the 24-hour precipitation depth, P 24, from Figure 4-1 for the return period selected in Step 1.

3. Use the time step selected above and tabulation Px/P24 rations for each time step of the 24-hour storm, using Figure 4-2.

4. Multiply the PX/P24 ratios from Step 3 by the 24 hour precipitation depth from Step 2. Results of the calculations provide a cumulative rainfall hyetograph. Incremental hyetograph data can be prepared by finding the difference between each cumulative depth interval.

4.2.3 BALANCED STORM

The balanced storm approach provides a precipitation hyetograph that has the same return period for each time interval within the total storm duration. This approach places the largest increment of rainfall for the selected time step at the midpoint of the toil storm duration. For example, the midpoint for a 24-hour storm is 12 hours. Smaller depth increments are arranged symmetrically about the largest value, with the second largest value placed before and the third largest value placed after the largest. This "before and after" process is illustrated in Figure 4-3 and continues until the entire hyetograph is developed.

The data from Section 4.2.1 can be used to develop a site-specific balanced storm for Huntsville using the procedure described below. The balanced storm is nearly identical to the 24-hour SCS Type II storm, except that the SCS storm distribution is slightly more intense for durations between 15 minutes and 1 hour of midpoint.

1. Select the design storm time step and return period. The time step should correspond with the unit hydrograph duration if unit hydrograph procedures will be used (see Section 4.6.3).

2. Use the time step and return period from Step 1 to obtain precipitation depth data. Depth-duration frequency data can be obtained from Figure 4-1 and the increments of precipitation determined from the following steps:

    a. Find the intensity at each multiple of the time step (time intervals) to the duration of the storm.

    b. Determine the rainfall volume for each time interval (volume = intensity x time interval).

    c. Determine the increment of precipitation depth for each time step by subtracting the volume of the previous time interval from the volume of the present time interval.

3. Select the largest increment of precipitation depth from Step 2 and place it at the midpoint of the total storm duration. For a 24-hour storm, the midpoint is 12 hours,

4. Arrange smaller depth increments symmetrically about the largest depth. The second largest value is placed before the largest and the third largest value is placed after the largest.

5. Continue placing sequential depth increments before and after the largest value until the entire storm duration has been considered.

An example of the procedure for developing a balanced storm is presented in Section 4.6.

4.3 RAINFALL EXCESS

Rainfall excess is the depth of precipitation that runs off an area during or immediately following a rainstorm, or the water depth remaining when the following abstraction are subtracted from the total precipitation:

1. Evaporation
2. Infiltration
3. Transpiration
4. Interception
5. Depression storage

Because the complexity of the actual process precludes a detailed determination of each abstraction, several methods are available to approximate the combined effects based on watershed characteristics.

Either the runoff coefficient or the SCS curve number can be used to estimate rainfall excess. Each approach is expressed mathematically as shown below:

Runoff Coefficient

$$R_T = C_T \quad P_T \qquad (4\text{-}1)$$

SCS Curve Number

$$R_T = \frac{(P_T - 0.2s)^2}{P_T + 0.8S} \qquad (4\text{-}2)$$

$$S = \frac{1000}{CN} - 10 \qquad (4\text{-}3)$$

where:

R_T = Rainfall excess for return period T, in inches

C_T = Runoff coefficient for return period T, Dimensionless (see Section 4.3.1)

P_T = Precipitation depth for return period T, in inches

S = Maximum soil storage, in inches

CN = Watershed curve number (see Section 4.3.2)

Procedures for determining the runoff coefficient and SCS curve number are discussed below. Variables that should be considered for either procedure include soil type, land use, antecedent moisture conditions, and precipitation volume.

Runoff coefficients or SCS curve numbers may be adjusted slightly if calibration data demonstrate a different value is justified. However, in the absence of adequate field data, the general procedures described in this section should be used.

4.3.1 RUNOFF COEFFICIENTS

The runoff coefficients for various land uses, soil types, and watershed slopes in Table 4-2 apply when a designed storm with a return period of 10 years or less is considered. Runoff coefficients can be taken directly from the table for homogeneous land use. For mixed land uses, a weighted C value should be calculated as follows:

$$\overline{C} = \frac{\sum_{i=1}^{n} C_i A_i}{A_T} \qquad (4\text{-}4)$$

where: $\overline{C}$ = Weighed composite runoff coefficient

n = Total number of areas with uniform runoff coefficients

C_i = Runoff coefficient for subarea i

A_i = Land area contained in subarea i with uniform land use conditions, in acres or square miles

A_T = Total area of watershed, in acres or square miles

For return periods of more than 10 years, the coefficients from Table 4-2 should be multiplied by the frequency factors from Table 4-3. The following relationship is used to combine the data presented in Tables 4-2 and 4-3:

$$C_T = C_{10} X_T \tag{4-5}$$

where:

C_T = Runoff coefficient for return period T, dimensionless

C_{10} – Runoff coefficient for a design storm return period of 10 years or less (see Table 4-2)

X_T = Design storm frequency factor for the return period T (see Table 4-3)

The value of C_T should never be increased above 1.0.

4.3.2 SCS CURVE NUMBERS

The following procedure determines the SCS curve number based on soil survey information published by the SCS:

1. Identify soil types using the SCS soil survey report for Madison County, Alabama (USDA, SCS, 1958).

2. Assign a hydrologic group to each soil type. The SCS has classified more than 4,000 soil series into four hydrologic soil groups, denoted by the letters A, B, C, and D. Soils in the A group have the lowest runoff potential; soils in the D group have the highest. Two letters can be assigned to the certain wet soils that may be drained or undrained; the first letter applies to the drained condition and the second to the undrained. The following criteria are used for assigning dual groups:

    a. Soils are rated D in their natural condition.

    b. Drainage is feasible and practical.

    c. Drainage improves the soil hydrologic group by at least two classes (e.g., D to B).

    The hydrologic soil group classification considers only the soil properties that influence the minimum rate of infiltration obtained for a bare soil after prolonged wetting.

3. Identify drainage areas with uniforms soil type and land use conditions.

4. Use tables to select curve number values for each uniform drainage area identified in Step 3. A curve number value for Antecedent Moisture Conditions (AMC) II can be selected using the Table 4-4 and 4-5. Table 4-4 provides curve numbers for selected urban and suburban land uses; Table 4-5 gives information on rural land uses.

Several special factors should be considered when curve numbers are developed for an urban area, including the degree to which heavy equipment may compact the soil and the degree of surface and subsurface soil mixing caused by grading. In addition, the amount of barren pervious area (with little sod established) should be evaluated. Any one of these factors could move a soil normally placed in hydrologic group A or B to group B or C, respectively.

5. Calculate a composite curve number for the watershed using the equation:

$$\overline{CN} = \frac{\sum_{i=1}^{n} CN_i A_i}{A_T} \qquad (4\text{-}6)$$

where:

$\overline{CN}$ = Composite curve number for the watershed

N = Total number of areas with uniform soil type and land use Conditions

CN_i – Curve number for subarea i with a given soil type and land Use conditions (from Tables 4-4 and 4-5)

A_i = Land area for subarea i with uniform soil type and land use conditions, in acres or square miles

A_T = Total area of watershed, in acres or square miles

4.3.3 EXAMPLE PROBLEMS

Example 4-1. Rainfall Excess Using the Rational Method Runoff Coefficient

A 50-acre wooded watershed with an average slope of about 4 percent and good ground cover on both sandy and clay soils is to be developed as follows:

1. Undisturbed woodland on sandy soil (hydrologic soil group A) = 10 acres
2. Undisturbed woodland on clay soil (hydrologic soil group D) = 10 acres
3. Multi-family residential site on sandy soil group D) = 10 acres
4. Industrial site on sandy soil (hydrologic soil group D) = 10 acres

Calculate the rainfall excess for proposed conditions from a 25-year, 24 hour storm using the Rational Method runoff coefficient.

1. From Figure 4-1, the 25-year, 24-hour rainfall depth is 6.21 inches.

2. The composite weighted runoff coefficient is computed from Equation 4-4 (repeated below)

$$\overline{C} = \frac{\sum_{i=1}^{n} C_i A_i}{A_T}$$

as follows:

   a. From Table 4-2, for rolling (2-7 percent) woodland areas on sandy soil and assuming mid-range values, $C_1 = 0.17$

   b. From Table 4-2, for rolling (2-7 percent) woodland areas on clay soil and assuming mid-range values, $C_2 = 0.22$

   c. From Table 4-2, for rolling (2-7 percent) multi-family residential areas on sandy soil and assuming mid-range values, $C_3 = 0.70$

   d. From Table 4-2, for rolling (2-7 percent) industrial areas on sandy soil and assuming mid-range values, $C_4 = 0.85$

Sub-Area (i)	Area in Acres (A_i)	Runoff Coefficient (C_i)	Runoff Coefficient x Area (C_iA_i)
1	10	0.17	1.7
2	10	0.22	2.2
3	20	0.70	14.0
4	10	0.85	8.5
Total	50		26.4

$\overline{C} = \frac{26.4}{50}$, $\overline{C} = 0.53$

3. From Equation 4-5 and Table 4-3, for a 25-year return period, the runoff coefficient is

$C_{25} = \overline{C}\, X_{25}$

$C_{25} = 0.53\ (1.1)$, $C_{25} = 0.58$

4. The rainfall excess is computed with Equation 4-1:

$R_{25} = 0.58\ (6.21)$

R_{25} = 3.6 inches

Example 4-2. Rainfall Excess Using the SCS Curve Number

Using the watershed and proposed development from Example 4-1, calculate the rainfall excess for proposed conditions from a 25-year, 24-hour storm by the SCS curve number method.

1. From Figure 4-1, the 25-year, 24-hour rainfall depth is 6.21 inches.

2. The composite weighted curve number is computed from Equation 4-6 (repeated below)

$$\overline{CN} = \frac{\sum_{i=1}^{n} CN_i A_i}{A_T}$$

as follows:

a. From Table 4-5, for woodlands with good ground cover on hydrologic soil group A, CN_1 = 30

b. From Table 4-5, for woodlands with good ground cover on hydrologic soil group D, CN_2 = 77

c. From Table 4-4, for residential area with 1/8-acre average lot size on hydrologic soil group B, CN_3 = 85

d. From Table 4-4, for industrial areas on hydrologic soil group D, CN_4 = 93

Sub-Area (i)	Area in Acres (A_i)	Curve Number (C_i)	Curve Coefficient x Area (CN_iA_i)
1	10	30	300
2	10	77	770
3	20	85	1,700
4	10	93	930
Total	50		3,700

$$\overline{CN} = \frac{3{,}700}{50}, \quad \overline{CN} = 74$$

3. From Equation 4-3, the maximum soil storage in inches is

$$S = \frac{1{,}000}{} - 10, \quad S = 3.51 \text{ inches}$$

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3. The rainfall excess is computed using Equation 4-2:

$$R_{25} = \frac{[6.21 - 0.2\ (3.51)\ ]^2}{1.21\ + 0.8\ (3.51)}$$

$$R_{25} = \frac{(5.51)^2}{9.02}$$

$$R_{25} = 3.4 \text{ inches}$$

4.4 TIME OF CONCENTRATION

The time of concentration, t_C, is the time required for hydraulic wave to travel across the watershed. It is often approximated as the time required for runoff to travel from the hydraulically most remote part of the watershed to the point of reference.

To calculate the time of concentration of a watershed, at least three runoff components should be considered:

1. Overland
2. Shallow channel (typically rill or gutter)
3. Main channel

The Velocity Method is a segmental approach that can be used to account for each of these components. The average velocity for each flow segment being evaluated is considered and a travel time calculated using the following equation

$$t_i = \frac{L_i}{(60)\ V_i} \qquad (4\text{-}7)$$

where:

t_i = Travel time for flow segment I, in minutes

L_i = Length of the flow path for segment i, in feet

v_i = Average velocity for segment I, in feet/second

The time of concentration is then calculated, expressed as:

$$Tc = t_1 + t_2 + t_3 + \ldots t_i \qquad (4\text{-}8)$$

where:

t_c = Time of concentration, in minutes

t_1 = Overland flow travel time, in minutes

t_2 = Shallow channel (typically rill or gutter flow) travel time, in minutes

t_3 = Main channel travel time, in minutes

t_i = Travel time for the $i^{\underline{th}}$ segment, in minutes

Procedures for estimating the average flow velocity are discussed in subsequent sections.

4.4.1 OVERLAND FLOW

The length of the non-concentrated overland flow segment generally should be limited to 300 feet (Engman, 1983). The kinematic wave equation developed by Ragan (1971) is recommended for calculation the travel time for such overland flow conditions. Figure 4-4 presents a nomograph that can be used to solve this equation, which is expressed as:

$$t_1 = 0.93 \left(\frac{L^{0.6} \; n^{0.6}}{I^{0.4} \; S^{0.3}} \right) \tag{4-9}$$

where:

t_1 = Overland flow travel time, in minutes

L = Overland flow length, in feet

n = Manning’s roughness coefficient for overland flow (see Table 4-7)

I = Rainfall intensity, in inches/hour

S = Average slope of overland flow path, in feet/foot

Manning’s n values reported in Table 4-6 were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations.

Equation 4-9 generally entails a trial and error process using the following steps:

1. Assume a trail value of rainfall intensity, I.

2. Find the overland travel time, t_1, using Figure 4-4.

3. Use t_1 from Step 2 in Equation 4-8 to find the actual rainfall intensity for a storm duration of t_c Section 4.2.1).

4. Compare the trail and actual rainfall intensities. If they are not similar, select a new trail rainfall intensity and repeat the process until both trail and actual rainfall intensities agree.

SCS TR-55 uses a non-iterative approximation to the overland flow travel time for flow paths of less than 300 feet. This approximation is expressed as:

$$t_1 = 0.42 \left(\frac{(nL)^{0.8}}{P_2^{0.5} S^{0.4}} \right) \qquad (4\text{-}10)$$

where:

t_1 = Overland flow travel time, in minutes

P_2 = 2-year, 24-hour rainfall, in inches

And the remaining terms are as defined in Equation 4-9

Equation 4-10 is based on a single rainfall intensity from a 2-year, 24-hour rainfall event. For many cases, this approximate method will yield acceptable results; however, for very flat or very steep watersheds, overland flow travel time should be checked using the iterative method for Equation 4-9.

The overland flow time has been predicted by the Federal Aviation Administration after analysis of airport areas to be as follows:

$$t = \frac{1.8\,(1.1\text{-}C)\,(L_o)^{.5}}{S^{1/3}}$$

t = overland flow time, minutes
c = rational runoff coefficient
S = watershed slope (avg.), percent
L_o = longest distance of overland flow to the collection point, feet

For irregularly shaped drainage areas, it will be necessary to evaluate several alternative overland flow distances.

4.4.2 SHALLOW CHANNEL FLOW

Average velocities for shallow channel flow in rills and gutters can be obtained directly from Figure 4-5, if the slope of the flow segment in percent is known. If the flow path length and average velocity are known, the travel time is estimated using Equation 4-7. Other types of shallow Manning's Equation (see Chapter 5). Additional procedures for evaluating gutter flow velocity are presented in Chapter 7. More than one segment of shallow channel flow can be considered to represent changing conditions.

4.4.3 MAIN CHANNEL FLOW

Average velocities for main channel flow should be evaluated using bankfull condition and the open channel procedures presented in Chapter 5. More than one main channel flow segment can be used where necessary.

4.4.4 EXAMPLE PROBLEM

Example 4-3. Time of Concentration Computation

The hydrologic flow path of the watershed described in Example 4-1 is about 2,000 feet in length with a total elevation change of ab out 40 feet. This flow path may be divided into the following three segments:

Segment No.	Type of Flow	Segment Length (ft)	Elevation Change (ft)	Slope (%)
1	Overland (woodland)	250	25	10
2	Shallow channel	750	13	1.7
3	Main channel n = 0.025 Width = 10 feet Depth = 2 feet Approximately rectangular channel	1,000	2	0.20

Compute the watershed time of concentration for a 100-year storm.

1. Compute the overland flow travel time, t_1, using the SCS TR-55 method from Equation 4-10 (repeated below).

$$t_1 = 0.42 \frac{(nL)^{0.8}}{P_2^{0.5} S^{0.4}}$$

From Figure 4-1, the 2-year, 24-hour rainfall depth, P_2, is 3.78 inches.

From Table 4-7, for woodlands, n = 0.45.

$$t_1 = 0.42 \frac{(0.45 \text{ X } 250)^{0.8}}{(3.78)^{0.5} (0.10)^{0.4}}$$

t_1 = 24 minutes

2. Compute the shallow channel flow travel time.

   From Figure 4-5, for a slope of 1.7 percent, the flow velocity is 2.6 feet/second.

   By Equation 4-7,

$$t_2 = \frac{750}{(60)\,(2.6)}$$

$t_2 = 4.8$ minutes

3. Compute the main channel flow travel time.

   The flow velocity is given by Manning's Equation (Equation 5-5),

   $$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

   For a rectangular channel with a 10-foot bottom width and depth of 2 feet,

   $R = (2 \text{ X } 10)/\ [2(2)+10]$

   $R = 20/14 = 1.43$ feet

   $$V = \frac{1.49}{0.025} (1.43)^{0.67} (0.002)^{0.5}$$

   $V = 3.4$ feet/second

   By Equation 4-7,

   $$t_3 = \frac{1{,}000}{(60)\,(3.4)}$$

   $t_3 = 4.9$ minutes

4. Compute the watershed time of concentration.

   From Equation 4-8,

   $T_c = 24 + 4.8 + 4.9$
   $T_c = 34$ minutes

5. Check the time of concentration using the kinematic wave equation (Equation 4-9, repeated below).

   $$t_1 = 0.93 \frac{L^{0.6} n^{0.6}}{I^{0.4} S^{0.3}}$$

   From Step 1, n = 0.45. From Figure 4-1, for t_1 = 25 minutes from Step 1, the 10-year return frequency rainfall intensity is 4.05 inches per hour.
   Assuming the rainfall intensity to be 4.05 inches per hour, Equation 4-9 gives

$$t_1 = 0.93 \frac{(250)^{0.6}(0.45)^{0.6}}{(4.05)^{0.4}(0.10)^{0.3}}$$

t_1 = 18 minutes

From Step 2, t_2 = 4.8 minutes. From Step 3, t_3 = 4.9 minutes. From Equation 4-8,

t_c = 18 + 4.8 + 4.9

t_c = 28 minutes

From Figure 4-1, for t_c = 28 minutes, the 10-year return frequency rainfall intensity is 3.75 inches per hour.

Trail rainfall intensity and computed rainfall intensity do not agree.

6. Repeat computation assuming rainfall intensity is 3.75 inches per hour.

$$t_1 = 0.93 \frac{(250)^{0.6}\quad(0.45)^{0.6}}{(3.75)^{0.4}(0.10)^{0.3}}$$

T_1 = 18.6 minutes

T_c = 28 minutes

From Figure 4-1, for t_c = 28 minutes, rainfall intensity is about 3.75 inches/hour.

Trail rainfall intensity and computed rainfall intensity agree.

7. Use t_c = 28 minutes.

Note: For this steep slope example, the SCS TR-55 method overestimates the time of concentration by about 20 percent. This demonstrates the need to check Results from TR-55 for extreme cases.

4.5 PEAK RUNOFF RATES

4.5.1 GAGED SITES

Streamflow and flood frequency data for gaged watersheds are available from the U.S. Geological Survey (USGS) and TVA for selected streams, as presented in Table 4-8. In the event that streamflow measurements have not been analyzed to develop appropriate flood frequency curves, guidelines presented by the U.S, Water Resources Council (1986) should be followed.

The major drainage basins in Huntsville are Aldridge Creek, Spring Branch and Indian Creek. Smaller drainage basins are tributary to Dry Creek, McDonald Creek, Bradford Creek, and Betts Spring branch. The Aldridge Creek Valley watershed, which drains an area of 21.7 square miles, begins in Huntsville near U.S. Highway 431 and flows south to the Tennessee River. The valley is gently sloping, with broad floodplains and a number of small tributaries. Development in the valley is primarily residential and has occurred to some extent in the floodplain.

The Huntsville Spring Branch Valley drains 85.7 square miles and is a broad, undulating-to-rolling belt traversing high developed municipal and community property. The number tributaries flowing into Huntsville Spring Branch include Pinhook Creek, Fagan Creek, Dallas Branch and Broglan Branch. Pinhook Creek has been highly developed for a number of years.

The southern portion of the City is affected by the Tennessee River. The system of reservoirs operated by the TVA on the Tennessee River and its tributaries now comprises 33 major projects. Out of all the 21 multi-purpose project, 11 are on tributaries and 5 are on the main river upstream from Huntsville. Fourteen of these upstream projects reserve storage space for the control of flood flows. Their combined effect is to greatly alleviate flood damage threats at many locations in the Tennessee River basin.

4.5.2 RATIONAL METHOD

In this manual, the Rational Method is expressed in the equation:

$$Q_T = C_T \, I_{tc} \, A \qquad (4\text{-}11)$$

where:

Q_T = Peak runoff rate for return period T, in cfs

CT = Runoff coefficient for return period T, expressed as the dimensionless ratio of rainfall excess to total rainfall

I_{tc} = Average rainfall intensity, in inches/hour, during a period of time equal to t_c for the return period T

t_c = Time of concentration (see Section 4.4), in minutes

A = Watershed drainage area, in acres, tributary to the design point

The following procedure is recommended for using the Rational Method:

1. Collect watershed data.

2. Calculate time of concentration using information in Section 4.4.

3. Use the IDF curves in Figure 4-1 to determine the average rainfall intensity for the return period T and the time of concentration, t_c, from Step 2.

4. Obtain a runoff coefficient for the return period T, using the information in Section 4.2.

5. Compute the peak runoff rate for the return period T, using Equation 4-11.

4.5.3 TVA REGRESSION EQUATIONS

Flood frequency regression equations were developed by the TVA (1986) as part of an urban flood study for small, ungagged, urban watersheds in Huntsville with drainage areas between 60 and 3,200 acres and with imperviousness between 3 and 50 percent. These equations are applicable only where the impact from manmade structures and channelization is insignificant.

The TVA regression equations and the TVA graphical flow frequency estimates presented in Section 4.5.4 were compared with results from USGS regression equations (Olin and Bingham, 1977). Similar results were obtained from watersheds with an imperviousness of about 20 percent, but the TVA relationships yield better estimates for other imperviousness ranges. As a result, the TVA relationships are more representative of urban conditions in Huntsville and should be used instead of the USGS equations.

The TVA regression equations for the 2-, 10-, 50-, 100-, and 500-year return frequency flood events are shown in Table 4-8. Graphical solutions to these equations are presented in Figures 4-6 through 4-10. Tributary drainage area and imperviousness are the only watershed characteristics required as input for the equations. The tributary area and imperviousness are the only watershed characteristics required as input for the equations. The tributary available sources, including the USGS 1” = 2,000’ topographic maps and the City’s 1” = 200’ maps. Imperviousness, in percent, should be determined from actual or proposed impervious surfaces such as rooftops and paved areas. Unless specific development plans are available to provide better estimates, imperviousness associated with future development may be estimated using the typical values presented in Table 4-9.

4.5.4 TVA GRAPHS

Graphical flow frequency estimates were developed by the TVA (1986) for the selected stream reaches listed in Table 4-10 and apply only to the stream reach for which each was developed. The estimates, which are based on future development conditions, are presented in Figures 4-11 through 4-33 as a function of stream mile. Other hydrologic methods are required to develop flood hydrographs or to account for routing through stormwater facilities.

4.5.5 SCS TR-55 GRAPHICAL METHOD

The SCS has developed a graphical peak discharge method, presented in SCS TR-55 (1986), for estimating the peak runoff rate watersheds with a single homogeneous land use.

The method is based on the results of computer analysis per formed using TR-20 (USDA, SCS, 1983) and is subject to the following limitations:

1. Estimates of peak design flows only.
2. Design storm = SCS Type II 24-hour distribution.
3. Time of concentration, t_c, of 0.1 hour $\leq t_c \leq$ 10 hours.
4. Presence of a single homogenous subbasin. The procedure was developed using a DA of 1 square mile.
5. Curve number, CN, of 40 $\leq$ CN $\leq$ 98.
6. Ratio of initial abstraction to precipitation I_a/P, of $0.1 \leq I_a/P \leq 0.5$.
7. Unit hydrograph shape factor of 484.
8. Only one main stream channel in the watershed or, if more than one exists, nearly equal times of concentration for the branches.
9. Use of the 1986 version of TR-55 in place of the 1975 procedures.
10. No consideration of hydrologic channel routing.

The graphical peak discharge method described in Chapter 4 of SCS TR-55 is based on the following equation:

$$Q_t = q_u A_m R_T F_P \qquad (4\text{-}12)$$

where:

Q_t = Peak runoff rate for return period T, in cfs

Qu = Unit peak discharge (from figure 4-34). In cubic feet per second per square mile per inch (csm/inch)

A_m= Drainage area, in square miles

R_T = Rainfall excess for return period T, in inches (see Equation 4-2)

F_p = Pond and swamp adjustment factor (see Step 5 below)

Computation using the graphical peak discharge method proceeds as follows:

1. The 24-hour rainfall depth is determined from Figure 4-1 for the selected return frequency.

2. The runoff curve number, CN, is estimated using the procedures of Section 4.3.2 and rainfall excess is estimated using Equation 4-2.

3. The CN value is used to determine the initial subtraction, I_a, from Table 4-11 and the ration I_a/P is then computed.

4. The watershed time of concentration is computed using the procedures in Section 4.4 and is used with ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 4-34. If the ratio I_a/P lies outside the range shown in Figure 4-34, 0.1 to 0.5, another peak discharge method should be used.

5. The pond and swamp adjustment factor, Fp, is estimated from below (TR-55m, USDA, SCS, 1986):

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

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

6. The peak runoff rate is computed using Equation 4-12. The same procedure should be used to estimate pre-and post-development times of concentration when computing pre- and post-development peak discharge.

Since the 1986 version of TR-55 includes extensive revisions to the 1975 version, the earlier version is no longer appropriate for use. The 1986 version can be obtained from the Nation Technical information Service in Springfield, Virginia 22161. The catalog number TR-55, “Urban Hydrology for Small Watersheds,” is PB87-101580. Microcomputer diskettes with TR-55 procedures are also available under catalog number PR87-101598.

4.5.6 OTHER TECHNIQUES

Techniques other than those listed here may be used for computation of design flow rates, subject to the approval of the Engineering Division.

4.5.7 EXAMPLE PROBLEMS

Example 4-4. Rational Method Peak Runoff Rate

Use the Rational Method to compute the peak runoff rate from the watershed in Example 4-1 for a 25-year, 24-hour storm event.

1. Total area of watershed = 50 acres.

2. From Example 4-1, the 25-year runoff coefficient, C_{25}, is 0.58.

3. From Example 4-3, the watershed time of concentration is

    a. By TR-55 (Equation 4-10), t_c = 34 minutes

    b. By kinematic wave equation (Equation 4-9), t_c = 28 minutes

4. From Figure 4-1, the rainfall intensity for a 25-year storm is

    a. For tc = 34 minutes, I = 4.0 inches/hour

    b. For tc = 28 minutes, I = 4.4 inches/hour

5. The peak runoff rate is computed from Equation 4-11 as follows:
    a. Using the SCS TR-55 method for t_c:

        Q_{25} = (0.58) (4.0) (50)

        Q_{25} = 116 cfs

    b. Using the kinematic wave equation for t_c:

        Q_{25} = (0.58) (4.4) (50)

        Q_{25} = 128 cfs

Example 4-5. TVA Regression Equation Peak Runoff Rate

The 50-acre watershed in Example 4-1 has an estimated imperviousness of about 30 percent resulting from urbanization. Determine the 10-, 50-, and 100-year peak runoff rates using the TVA regression equations.

1. Determine if the watershed characteristics are within the limits of applicability of regression equations.

    a. Area = 50 acres or 0.078 square miles

    b. Imperviousness = 30 percent

    The area is slightly less than the lower limit of applicability (60 acres) and the imperviousness is within the range of applicability (3 to 50 percent). Although the area is less than 60 acres, use the regression equations for comparison.

2. From Table 4-9, the peak runoff rates are as follows:

    a. $Q_{10} = 365\ (0.078)^{0.69}\ (30)^{0.23}$

       $Q_{10} = 137$ cfs

    b. $Q_{50} = 680\ (0.078)^{0.69}\ (30)^{0.15}$

       $Q_{50} = 195$ cfs

    c. $Q_{100} = 855\ (0.075)^{0.69}\ (30)^{0.13}$

       $Q_{100} = 229$ cfs

<u>Example 4-6. SCS TR-55 Graphical Method Peak Runoff Rate</u>

Use the SCS graphical peak discharge method to compute the peak runoff rate from a 25-yer, 24-hour storm event from the watershed in Example 4-1. Use the watershed time of concentrated computed in Example 4-3.

1. From Figure 4-1, the 25-year, 24-hour rainfall depth is 6.21 inches.

2. From Example 4-2, the composite curve number, $\overline{CN}$, is 74.

3. From Example 4-2, Step 3, the soil storage, S, is 3.51 inches.

4. Using Equation 4-2, the rainfall excess is

    $$R_{25} = \frac{[6.21 - 0.2\ (3.51)\ ]^2}{6.21\ + 0.8\ (3.51)}$$

5. From Table 4-12, the initial abstraction, I_a, is 0.703 inches.

6. The I_a/P ratio is

$I_a/P = 0.703/6.21$

$I_a/P = 0.11$

7. From Figure 4-34, for the time of concentration, t_c, from Example 4-3 of 34 minutes (0.57 hours) and with an I_a/P ration of 0.11, the unit peak discharge, q_u, is 500 csm/inch of runoff.

8. The pond and swamp adjustment factor F_p, is 1.0 since no pond or swamp area exists.

9. The peak runoff rate is computed using Equation 4-12, as follows:

   Q25 = (500) (50/640) (3.4) (1.0)

   Q25 = 133 cfs

4.6 FLOOD HYDROGRAPHS

Following a discussion of hydrograph terminology, flood hydrograph procedures for unit hydrograph theory and the SCS TR-55 (1986) tabular method are discussed below.

4.6.1 HYDROGRAPH TERMINOLOGY

A flood hydrograph is a continuous plot of the surface runoff flow rate versus time. Although historical streamflow data are preferred, they are not generally collected for the typical watershed; thus, synthetic methods for developing flood hydrographs are often required. If historical data are not available for a given basin, then data from a similar basin may be appropriate.

A typical hydrograph resulting from an isolated period of rainfall has the following major components, as illustrated in Figure 4-35:

1. Rising limb
2. Crest segment
3. Falling limb or recession curve

The shape of the rising limb is influenced primarily by the characteristics of the storm that produced the surface runoff. The point of inflection on the recession curve of the hydrograph is commonly assumed to mark when surface inflow to the channel system ceases (see Figure 4-36). Thereafter, the recession curve represents the withdrawal of water from storage within the watershed. As a result, the recession curve is largely independent of the storm and is

Influenced instead by watershed characteristics, such as channel slope and storage availability.

The hydrograph terminology used throughout this manual is presented in Figure 4-35, along with appropriate SCS hydrograph equations. A rainfall excess hyetograph, which in this case is a single block of rainfall excess over duration, D, is shown in upper part of Figure 4-35. The runoff hydrograph is presented directly below the rainfall excess hyetograph.

The area enclosed by the hyetograph and by the runoff hydrograph represents the same volume, Q, of direct runoff. The maximum flow rate of the hydrograph is the peak flow, q_p, while the time from the start of the hydrograph to q_p is the time to peak, tp. The total time duration of the hydrograph is known as the time base, tb. The watershed lagtime, t, is defined as the time from the center of mass of the rainfall excess to the runoff hydrograph peak. The following equation summarizes the relationship between the time parameters for the rising limb of a direct runoff hydrograph:

$$t_p = \frac{D}{2} + t_L \qquad (4\text{-}13)$$

where:

t_p = Time to peak or time of rise of the runoff hydrograph

D = Duration of runoff-producing rainfall

t_L = Watershed lagtime

The recession time for a hydrograph is the difference between the time base and the time to peak and can be expressed as:

$$t_r = t_b - t_p \qquad (4\text{-}14)$$

where:

t_r = Hydrograph recession time
t_b = Hydrograph time base
t_p = Time to peak or time of rise of the runoff hydrograph

4.6.2 UNIT HYDROGRAPH THEORY

The concept of a unit hydrograph, published by L.K. Sherman in 1932, provides a widely accepted basis for converting rainfall excess from a watershed to a runoff hydrograph. Although the tools and data available for developing unit hydrographs have become more extensive since Sherman first proposed unit hydrograph theory, the concept has not changed.

A unit hydrograph is defined as a runoff hydrograph that is produced by 1 inch of rainfall excess distributed uniformly over a watershed and occurring at a uniform rate during a specified period of time. Sherman originally used “unit” to denote the specified duration of rainfall excess for a particular unit hydrograph. The word “unit” is often misinterpreted as 1 inch or a “unit depth” of effective rainfall excess rather than as a “unit of time” for rainfall excess as originally intended.

The following assumptions constitute the basis of unit hydrograph theory:

1. The rainfall excess is uniformly distributed within its unit duration or specified period of time.

2. The rainfall excess is uniformly distributed in space over a particular watershed.

3. The time base for a direct runoff hydrograph due to a rainfall excess of unit duration is constant.

4. The ordinates of the direct runoff hydrographs, hen a common unit duration is considered, are directly proportional to the total volume of direct runoff represented by each hydrograph (principle of linearity or superposition).

5. For a given drainage basin, the hydrograph of runoff caused by a unit duration and volume of rainfall excess is invariable (principle of time invariance).

These assumptions cannot be precisely applied to natural precipitation and drainage basin characteristics. However, experience has shown that the unit hydrograph method gives results that are sufficiently accurate for most drainage problems.

The two fundamental assumptions that must always be considered when applying unit hydrograph theory are the principle of linearity (Assumption 4) and the principle of time invariance (Assumptions 5). Theoretically, each increment of rainfall excess can be routed through the subject watershed in accordance with the principle of linearity. In practice, this means that the product of a rainfall excess volume and the sequence of unit hydrograph ordinates (i.e., runoff rates in cubic feet per second per inch of rainfall excess) produces an estimate of the runoff hydrograph for that volume of rainfall excess. In addition, the principle of linearity allows individual runoff hydrographs developed from a sequence of individual rainfall excess volumes (i.e., a design storm of rainfall excess increments arranged in units of time equal to the unit duration) to be superimposed and added when estimating a total runoff hydrograph. The characteristics of the drainage basin be fixed or specified for a particular unit hydrograph. Land development and channel improvements are typical activities that violate the principal of time invariance.

4.6.3 UNIT HYDROGRAPH PROCEDURE

A flood hydrograph can be developed through the following steps using the unit hydrograph procedure:

1. Develop a unit hydrograph for the subject watershed using an appropriate procedure (see discussion below).

2. Develop a design storm hyetograph using the time interval for which the unit hydrograph was developed and an appropriate procedure (as presented in Section 4.2).

3. Develop a rainfall excess hyetograph using an appropriate procedure as presented in Section 4.3.

4. Route the rainfall excess hyetograph through the subject watershed by multiplying the ordinates of the unit hydrograph by the respective rainfall excess increments. Each increment of rainfall excess will produce a routed incremental hydrograph. Each routed incremental hydrograph is delayed by the design storm time interval.

5. Develop the composite synthetic runoff hydrograph by summing the ordinates of each routed incremental hydrograph from Step 4 at each time interval of the hydrograph.

6. Check to ensure that the volume of the synthetic runoff hydrograph is equal to the volume of rainfall excess, using the equation:

$$V = \frac{12\Delta t \ \Sigma q_i}{A\,(43{,}560)} \qquad (4\text{-}15)$$

where:

V = Volume under the hydrograph, in inches

Δt = Time increment of the runoff hydrograph ordinates, in cfs, for each time increment i

Σq_i = Sum of the runoff hydrograph ordinates, in cfs, for each time increment i

A = Watershed drainage area, in acres

4.6.4 SYNTHETIC UNIT HYDROGRAPHS

Unit hydrographs should be developed using observed rainfall and streamflow records when they are available. Procedures for deriving unit hydrograph parameters from observed data are well-documented in publications by Linsley, Kohler, and Paulhus (1982), Viessman et al. (1977), Chow (1964), and the USDOT, FHWA (HEC-19, 1984). When observed data are not available for deriving unit hydrograph parameters, as is often the case, synthetic procedures are required. The SCS dimensionless unit hydrograph approach is presented below.

Two types of dimensionless unit hydrographs have been developed by the SCS; the first has a curvilinear shape as shown in Figure 4-37 and the second is a triangular approximation to that curvilinear shape (see Figure 4-36). In both cases, once the time to peak and peak flow for a particular unit hydrograph have been defined, the entire shape of the unit hydrograph can be estimated using the dimensionless unit hydrograph ratios and mass data, as shown in Table 4-12.

The procedure for using the SCS curvilinear dimensionless unit hydrograph is as follows:

1. Estimate the time of concentration, t_c, using an appropriate method (see Section 4.4).

2. Calculate the incremental duration of runoff producing rainfall, ΔD, using the equation:

$$\Delta D = 0.133\ t_c \qquad (4\text{-}16)$$

where:

ΔD = incremental duration of runoff producing rainfall, in minutes

t_c = Time of concentration, in minutes

3. Calculate the time to peak, t_p, using the equation:

$$t_p = \frac{\Delta D}{2} + 0.6\ t_c \qquad (4\text{-}17)$$

where:

t_p = Time to peak, in minutes

ΔD = Incremental duration of runoff producing rainfall, in minutes

t_c = Time of concentration, in minutes

4. Calculate the peak flow rate, q_p, as follows:

$$q_p = 60\ \frac{B\ A}{t_p} \qquad (4\text{-}18)$$

where:

q_p = Peak flow rate, cfs

B = Hydrograph shape factor, ranging from 300 for swampy areas to 600 in steep terrain. SCS standard B value = 484.

A = Drainage area, in square miles

t_p = Time to peak, in minutes

5. List the hydrograph time, t, in increments of D and calculate t/t_p.

6. Using Table 4-12 or Figure 4-37, find the q/q ratio for the appropriate t/t_p ratios calculated in Step 5.

7. Calculate the appropriate unit hydrograph ordinates by multiplying the q/q_p rations by q_p.

8. Determine the volume under the unit hydrograph to ensure that it is equal to 1 inch. The SCS triangular dimensionless unit hydrograph procedure is identical to the curvilinear procedure presented above. However, to draw the required unit hydrograph, only t/t_p ratios of 0,1, and 2.67 are needed. When applying the triangular dimensionless unit hydrograph, the time of concentration, t_c is computed using Equation 4-16, the time to peak, t_p, is computed using Equation 4-17, and the time base t_b, is computed as follows:

$$t_b = 2.67\ t_p \qquad (4\text{-}19)$$

where:

t_p = Time to peak, in minutes

t_b = Time base, in minutes

If a short-duration unit hydrograph is used to develop a long-duration unit hydrograph is used to develop a long-duration synthetic hydrograph, the actual shape of the unit hydrograph is not nearly as important as the time peak and peak flow rate for that unit hydrograph. Therefore, a triangular unit hydrograph would probably produce approximately the same synthetic runoff hydrograph as a curvilinear unit hydrograph.

4.6.5 SCS TR-55 TABULAR METHOD

The tabular method developed by SCS can be used to estimate flood hydrographs from watersheds that can be divided into relatively homogeneous land uses. The method is based on the results of computer analyses performed using TR-20 (USA, SCS, 1983) and can be used to approximate the effects of hydrologic channel routing. Details for applying the procedure are contained in SCS TR-55 (1986). Limitations of the tabular method are summarized as follows:

1. Design storm = SCS Type II, 24-hur distribution.

2. Time of concentration, t_c, of 0.1 hour $\leq t_c \leq$ 2 hours.

3. Das of individual subareas do not differ by a factor of 5 or more. The procedure was developed for DA of 1 square mile.

4. Curve number, CN, of $40 \leq CN \leq 98$.

5. Ratio of initial abstraction to precipitation, I_a/P, of $0.1 \leq I_a/P \leq 0.5$.

6. Unit hydrograph shape factor = 484.

7. The reach travel time, t_T, is 0 to 3 hours.

Because the 1986 version of TR-55 includes extensive revisions to the 1975 version, the earlier version is no longer appropriate. 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. Micro-Computer diskettes with TR-55 procedures are also available under catalog number PB87-101598.

4.6.6 OTHER METHODS

Other methods of developing flood hydrographs may be used, subject to approval by the Director of City Engineering.

4.6.7 EXAMPLE PROBLEMS

Example 4-7. SCS Dimensionless Unit Hydrograph

Develop a synthetic unit hydrograph for the watershed in Example 4-1 using the SCS curvilinear approach.

1. From Example 4-3, the watershed time of concentration, t_c, is 34 minutes.

2. From Equation 4-16, the incremental duration of runoff producing rainfall,

   $\Delta D = 0.1333\ (34)$

   $\Delta D = 4.5$ minutes or 0.08 hours

3. From Equation 4-17, the time to peak, t_p, is

   $t_p = \frac{4.5}{2} + 0.6\ (34)$

   $t_p = 23$ minutes

4. From Equation 4-18, the unit hydrograph peak flow rate, q_p, is

   $q_p = 60\ \frac{484\ (50/640)}{23}$

   $q_p = 99$ cfs

5. From figure 4-37 or Table 4-12, determine the q/q_p ratio for appropriate t/t_p ratios and calculate the unit hydrograph ordinates by multiply the q/q_p ration by q_p as follows:

Time, T (hours)	t/t_p (t/0.4 hours)	q/q_p (q/99 cfs)	q (cfs)
0.00	0.00	0.000	0
0.08	0.20	0.100	10
0.16	0.40	0.310	31
0.24	0.60	0.660	65
0.32	0.80	0.930	92
0.40	1.00	1.000	99
0.48	1.20	0.930	92
0.56	1.40	0.780	77
0.64	1.60	0.560	55
0.72	1.80	0.390	39
0.80	2.00	0.280	28
0.88	2.20	0.207	20
0.96	2.40	0.147	15
1.04	2.60	0.107	11
1.12	2.80	0.077	8
1.20	3.00	0.055	5
1.28	3.20	0.040	4
1.36	3.40	0.029	3
1.44	3.60	0.021	2
1.52	3.80	0.015	1
1.60	4.00	0.011	1
1.68	4.20	0.008	1
1.76	4.40	0.005	0
1.84	4.60	0.002	0
1.92	4.80	0.001	0
2.00	5.00	0.000	0
			600

6. Check that the unit hydrograph volume equals 1 inch using Equation 4-15:

$$V = \frac{12\,(0.08)\,(3{,}600)\ \ (660)}{(50)\,(43{,}560)}$$

$$V = 1.05 \approx 1 \text{ (close enough)}$$

Example 4-8. Flood Hydrograph Using Hydrograph Theory

Develop a synthetic runoff hydrograph for a 25-year, 2-hour design storm for the watershed described in Example 4-1 using the unit hydrograph developed in Example 4-7.

1. Develop a balanced storm hyetograph and cumulative mass curve using the IDF curve for a 25-year storm from Figure 4-1, as shown in Table 4-13.

2. Develop a rainfall excess hyetograph using the SCS curve number approach. From Example 4-2, CN is 74 and S is 3.51 inches. From Figure 4-1, the 25-year, 2-hour rainfall depth is 3.78 inches.

   Apply Equation 4-2 to the cumulative depth as follows to obtain the rainfall excess hyetograph:

Time, t (hours)	Cumulative Depth (inches)	Cumulative Rainfall Excess (inches)	Rainfall Excess Hyetograph (inches)
0.00	0.00	0.00	0.00
0.08	0.01	0.00	0.00
0.16	0.04	0.00	0.00
0.24	0.12	0.00	0.00
0.32	0.20	0.00	0.00
0.40	0.30	0.00	0.00
0.48	0.41	0.00	0.00
0.56	0.51	0.00	0.00
0.64	0.63	0.00	0.00
0.72	0.80	0.00	0.00
0.80	0.98	0.02	0.02
0.88	1.24	0.07	0.05
0.96	1.68	0.21	0.14
1.04	2.33	0.51	0.30
1.12	2.66	0.70	0.19
1.20	2.90	0.85	0.15
1.28	3.06	0.95	0.10
1.28	3.06	0.95	0.10
1.36	3.20	1.04	0.09
1.44	3.31	1.11	0.07
1.52	3.42	1.18	0.07
1.60	3.62	1.32	0.07
1.68	3.62	1.32	0.07
1.76	3.70	1.38	0.06
1.84	3.75	1.42	0.04
1.92	3.77	1.43	0.01
2.00	3.78	1.44	0.01

3. Route the rainfall excess hyetograph through the watershed using the unit hydrograph developed in Example 4-7. Each increment of rainfall excess from the design storm is multiplied by the unit hydrograph ordinates. This routed incremental hydrograph begins at the time interval during which the rainfall excess occurred. The rainfall

excess hydrograph is obtained by summing the ordinates of each routed incremental hydrograph, as shown in Table 4-14.

4. Check that hydrograph volume is equal to the rainfall excess.

$$V = \frac{12(0.08)\ (3{,}600)\ (702)}{50\ (43{,}560)} = 1.48 \text{ inches (close enough)}$$

4.7 HYDROLOGIC CHANNEL ROUTING

The Muskingum Method of hydrologic channel routing is recommended when computer-based procedures are not used. A tabular method presented by the SCS in TR-55 (1986) is appropriate for preliminary desktop calculations.

4.7.1 MUSKINGUM METHOD

The Muskingum Method is applied with the following steps:

1. Select a representative flow rate for evaluating the parameters K and X. Use 75 percent of the inflow hydrograph peak. If this flow exceeds the channel capacity, use the channel capacity as representative.

2. Find the velocity of a small kinematic wave in the channel using the equation:

$$v = \frac{1}{B}\left(\frac{Q\,(Y + \Delta Y) - Q\,(Y)}{\Delta Y}\right) \qquad (4\text{-}20)$$

where:

v = Velocity of small kinematic wave, in feet/second

$Q\,(Y)$ = A representative flow rate for channel routing at representative Depth Y, in cfs

ΔY = A small increase in the representative depth of flow in the channel

$Q(Y + \Delta Y)$ = Flow rate at the new depth $Y + \Delta Y$, in cfs

B = Top width of water surface, in feet

3. Estimate the minimum channel length allowable for the routing, using the following equation, and make sure that the ΔL_{min}:

$$\Delta L_{min} = \frac{Q}{BS_o v} \qquad (4\text{-}21)$$

where:

ΔL_{min} = Minimum channel length for routing calculations, in feet

Q = Flow rate, in cfs

B = Top width of water surface, in feet

S_O = Slope of channel bottom, in feet/ foot

v = Velocity of a small kinematic wave, in feet/second

4. Estimate a value of K using the following equation (make sure that K is less than the time of rise for the inflow hydrograph):

$$K = \frac{\Delta L}{v} \qquad (4\text{-}22)$$

where:

K = Muskingum channel routing time constant for a particular channel segment

ΔL = Channel routing segment length, in feet

v = Velocity of a small kinematic wave, in feet/second

5. Estimate the value of X using the equation:

$$X = 0.5\left(1 - \frac{Q}{BS_o v \Delta L}\right) \qquad (4\text{-}23)$$

where:

X = Dimensionless factor that determines the relative weights of inflow and outflow on the channel storage volume

Q = Flow rate, in cfs

B = Top width of water surface, in feet

S_o = Slope of channel bottom, in feet/foot

v = Velocity of a small kinematic wave, in feet/second

ΔL = Channel routing segment length, in feet

6. Select a reasonable channel routing time period, Δt, using the criteria expressed by the following inequality:

$$\frac{K}{3} \leq \Delta t \leq K \qquad (4\text{-}24)$$

7. Determine coefficients C_0, C_1, and C_2 using the following equations (make sure that $C_0 + C_1 + C_2 = 1.0$):

$$C_0 = -\frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (4\text{-}25)$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (4\text{-}26)$$

$$C_2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (4\text{-}27)$$

where:

K = Muskingum channel routing time constant for a particular segment

X = Dimensionless factor that determines the relative weights of inflow and outflow on the channel storage volume

Δt = Routing time period, in hours

8. Determine an initial outflow, O_1, then calculate an ending outflow, O_2, using the equation:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \qquad (4\text{-}28)$$

where:

O_2 = Outflow rate at the end of routing time period Δt, in cfs

I_2 = Inflow rate at the end of routing time period Δt, in cfs

I_1 = Inflow rate at the beginning of routing time period Δt, in cfs

O_1 – Outflow rate at the beginning of routing time period Δt, in cfs

The routing is then performed by repetitively solving Equation 4-28, assigning the current value of O_2 to O_1, and determining a new value of O_2. This sequence of calculations continue until the entire inflow hydrograph is routed through the channel.

4.7.2 SCS TR-55 TABULAR METHOD

The SCS tabular method can be used to approximate the effects of hydrologic channel routing on flood hydrographs. Details for applying the procedure are contained in SCS TR-55 (1986). Limitation are summarized in Section 4.6.5. The 1986 version of TR-55 supersedes the 1975 version and should be used in place of the older publication.

Table 4-1
GUIDELINES FOR SELECTING HYDROLOGIC PROCEDURES

Hydrologic Method	Section of Manual	Peak Flow	Hydrograph
1. Rational Method[a]	4.5.2	Yes	No
2. SCS TR-55 Graphical	4.5.5	Yes	No
3. SCS TR-55 Tabular	4.6.5	Yes	Yes
4. TVA Regression Equations	4.5.3	Yes	No
5. TVA Graphs	4.5.4	Yes	No
6. Unit Hydrograph	4.6.3	Yes	Yes

Limits of Application

	Design Storm	Time of Concentration, t_c	Drainage Area, DA	Imperviousness	I_a/P
1. Rational Method[a]	t_c	5 min $\leq t_c \leq$ 30 min	$\leq$ 50 acres	0-100%	N/A
2. SCS TR-55 Graphical	24-hr Type II	0.1 hr $\leq t_c \leq$ 10 hr	b	40 $\leq$ CN $\leq$ 98	.1-.5
3. SCS TR-55 Tabular	24-hr Type II	0.1 hr $\leq t_c \leq$ 2 hr	c	40 $\leq$ CN $\leq$ 98	.1-.5
4. TVA Regression Equations	N/A	N/A	0.1 sq. mi $\leq$ DA $\leq$ 5 sq. mi.	3-50%	N/A
5. TVA Graphs	N/A	N/A	d	d	N/A
6. Unit Hydrograph	Any	> 0	> 0	0-100%	N/A

[a]Use of the Rational Method beyond the limits shown requires approval by the Director of City Engineering, and results should be compared using other methods.
[b]A single homogeneous sub basin is required. The procedure was developed using a DA of 1 square mile.
[c]Drainage areas of individual subareas cannot differ by a factor of 5 or more. The procedure was developed from resists pf TR-20 computer analysis with a DA of 1 square mile.
[d]Applicable only to the stream reached modeled as identified on graphs.
N/A = Not applicable.

GENERAL NOTE:
At the discretion of the Director of City Engineering, design discharge values available from other projects or studies shall be used in the design or analysis of selected stormwater management facilities.

Table 4-2

RUNOFF COEFFICIENTS[a] FOR A DESIGN STROM RETURN PERIOD OF 10 YEARS OR LESS

Slope	Land Use	Sandy Soils Min.	Sandy Soils Max.	Clays Soils Min.	Clays Soils Max.
Flat (0-2%)	Woodlands	0.10	0.15	0.15	0.20
	Pasture, grass, and farmland[b]	0.15	0.20	0.20	0.25
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavement[c]	0.75	0.95	0.95	0.95
	SFR: 1-2 – acre lots and larger	0.30	0.35	0.35	0.45
	Smaller lots	0.35	0.45	0.40	0.50
	Duplexes	0.35	0.45	0.40	0.50
	MFR: Apartments, townhouses, and condominiums	0.45	0.60	0.50	0.70
	Commercial and industrial	0.50	0.95	0.50	0.95
Rolling (2-7%)	Woodlands	0.15	0.20	0.20	0.25
	Pasture, grass, and farmland[b]	0.20	0.25	0.25	0.30
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavement[c]	0.80	0.95	0.90	0.95
	SFR: 1/2 acre lots and larger	0.35	0.50	0.40	0.55
	Smaller lots	0.40	0.55	0.45	0.60
	Duplexes	0.40	0.55	0.45	0.60
	MFR: Apartments, townhouses, and condominiums	0.50	0.70	0.60	0.80
	Commercial and industrial	0.50	0.95	0.60	0.95
Steep (7%+)	Woodlands	0.20	0.25	0.25	0.30
	Pasture, grass, and farmland[b]	0.25	0.35	0.30	0.40
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavement[c]	0.85	0.95	0.90	0.95
	SFR: ½ acre lots and larger	0.40	0.55	0.50	0.65
	Smaller lots	0.45	0.60	0.55	0.70
	Duplexes	0.45	0.60	0.55	0.70
	MFR: Apartments, townhouses, And condominiums	0.60	0.75	0.65	0.85
	Commercial and industrial	0.60	0.95	0.65	0.95

Reference: Dekalb County, Georgia (1976).

Note: SFR = Single-family residential
MFR = Multi-family residential

[a]weighed coefficient based on percentage of impervious surfaces, and green areas must be selected for each site.

[b]Coefficients assume good ground cover and conservation treatment.

[c]Depends on depth and degree of permeability of underlying strata.

Table 4-3
DESIGN STORM FREQUENCY FACTORS
FOR PERVIOUS AREA RUNOFF COEFFICIENTS

Return Period (years)	Design Storm Frequency Factor, X_T
2 to 10	1.0
25	1.1
50	1.2
100	1.25

Reference: Wright-McLaughlin Engineers (1969).

Table 4-4

RUNOFF CURVE NUMBERS FOR URBAN AREAS[a]

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	Average Percent Impervious Area[b]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawn parks gulf courses, cemeteries, etc.)[c]					
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		98	98	98	98
Paved parking lots, roofs, driveways, etc. (excluding right-of-way					
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	95	95
Industrial	72	81	88	91	93
Residential districts by average lot size					
1/8 acre or less (townhouses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
½ acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [d]		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 4-5)					

Reference: USDA, SCS, TR-55 (1986).

[a]Average runoff condition, Antecedent Moisture Condition II, and $I_a = 0.2S$.

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

[c]CNS shoun are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

[d]Composite CNs to us for the design of temporary measures during grading and construction should be computed based on the degree of development (impervious area percentage and the CNs for the newly graded pervious areas.

Table 4-5
RUNOFF CURVE NUMBERS FOR RURAL AREAS[a]

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type	Hydrologic Condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing[b]	Poor	65	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay	--	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element[c]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	32	58	72	79
Woods—grass combination (orchard or tree farm)[e]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[f]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[d]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots	--	59	74	82	86

Reference: USDA, SCS, NEH-4 (1972).

[a]Average runoff condition, Antecedent Moisture Condition II, and $I_a = 0.2S$.

[b]Poor: <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

[c]Poor: <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

[d]Actual curve number is less than 30; use CN = 30 for runoff computations.

[e]CNs shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pastures.

[f]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.

Table 4-6
OVERLAND FLOW MANING'S n VALUES

	Recommended Value	Range of Values
Concrete	.011	.01 - .013
Asphalt	.012	.01 - .015
Bare sand[a]	.010	.010 - .016
Graveled surface[a]	.012	.012-.030
Bare clay-loam (eroded)[a]	.012	.012 -.033
Fallow (no residue)[b]	.05	.006 - .16
Chisel plow (<1/4 ton/acre residue)	.07	.006 - .17
Chisel plow (1/4 -1 tons/acre residue)	.18	.07 - .34
Chisel plow (1-3 tons/acre residue)	.30	.19 - .47
Chisel plow (>3 tons/acre residual)	.40	.34 - .46
Disk/harrow (<1/4 ton/acre residue)	.08	.008 - .41
Disk/harrow (1/4 -1 ton/acre residue)	.16	.10 - .25
Disk/harrow (1-3 tons/acre residue)	.25	.14 - .53
Disk/harrow (>3 tons/acre residue)	.30	-- --
No till (<1/4 -1 ton/acre residue)	.04	.03 - .07
No till (1-3 tons/acre residue)	.07	.01 - .13
No till (1-3 tons/acre residue)	.30	.16 - .47
Plow (fall)	.06	.02 -.10
Coulter	.10	.05 - .13
Range (natural)	.13	.01 - .32
Range (clipped)	.08	.02 - .24
Grass (bluegrass sod)	.45	.39 -.63
Short grass praire[a]	.15	.10 -.20
Dense grass[c]	.24	.17 - .30
Bermudagrass[c]	.41	.30 -.48
Woods	.45	-- --

Notes: These values were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations. See Chapter 5 for open channel flow procedures.

All values are from Engman (1983), unless noted otherwise.

[A]woolhiser (1975).

[b]Fallow has been idle for one year and is fairly smooth.

[c]Palmer (1946). Weeping lovegrass, bluegrass, buffalograss, blue gramma grass, native grass mix (OK), alfalfa, lespedeza

Table 4-7
STREAM GAGES IN VICINITY OF HUNTSVLLE

Gaging Station	Number	Drainage Area (square miles)	Period of Record	Agency
Walker Branch near Pleva	03574796	0.44	1971-1975[a]	USGS
Morris Branch near Toney	03574840	1.43	1971-1975	USGS
Straight Ditch at Huntsville	03574852	0.17	1971-1975[a]	USGS
Glover Cove Creek near Owens Cross Road	03575340	3.52	1971-1975[a]	USGS
Aldridge Creek at Huntsville	03575686	1.15	1971-1975[a]	USGS
Aldridge Creek near Lily Flagg	03575696	13.9	1971-1975[a]	USGS
Aldridge Creek near Farley			1960-1964[b]	USGS
(Mt. Gap Road)	03575700	14.1	1986-present	
Pinhook Creek at Clinton	03575890	22.5	1986-present	USGS
Indian Creek near Madison	03575830	49.0	1959-1966[b]	USGS
			1966-1971[c]	USGS
			1971-1975[a]	USGS
Five Points Ditch at Huntsville (Tributary to Pinhook Creek at River Mile 15.15)	03575880	0.62	1971-1975[a]	USGS
Pinhook Creek at Huntsville	03575890	22.5	1966-1968[b]	TVA
			1971-1973[a]	USGS
Pinehaven Ditch at Huntsville	03575910	0.16		
Broglan Branch at Holmes Avenue at Huntsville	03575930	8.87	1971-1975[a]	USGS
Broglan Branch at Clinton	03575933	9.51	1986-present	USGS
Huntsville Spring Branch at Johnson Road at Huntsville	03575950	41.8	1967-1986[b]	TVA
			1971-1975[a]	USGS
			1986-present	
McDonald Creek at Pailon Road	03575980	--	1986-present	USGS
Dallas Branch at Coleman Street	0357587728	2.79	1986-present	USGS

USGS = U.S. Geological Survey

TVA = Tennessee Valley Authority

[a]Flood hydrograph partial-record station

[b]Continuous-record station

[c]Crest-stage partial-record station

Table 4-8
TVA REGRESSION EQUATIONS FOR SMALL, UNGAGED, URBAN WATERSHEDS IN HUNTSVILLE, ALABAMA[a]

Equation	Standard Error[b] of Estimate (%)
$Q_2 = 156\ A^{0.69}\ I^{0.32}$	44
$Q_{10} = 365\ A^{0.69}\ I^{0.23}$	44
$Q_{50} = 680\ A^{0.69}\ I^{0.13}$	--
$Q_{100} = 855\ A^{0.69}\ I^{0.13}$	44
$Q_{500} = 1{,}400\ A^{0.69}\ I^{0.09}$	--

where:

Q_T = Estimated discharge, in cfs, for the indicated recurrence interval, T

A = Tributary drainage area, in square miles

I = Watershed imperviousness, in percent

Reference: TVA (1986).

[a]Tributaty drainage area ranged from 0.1 to 5 square miles and imperviousness values ranged from 3 to 50 percent.

[b]The 50-year regression equation was developed by interpolation and the 500-year regression equation was developed by extrapolation; as a result, the standard error was not computed for these relationships.

Table 4-9
TYPICAL IMPERVISOUNESS FOR
DEVELOPMENT DENSITY IN HUNTSTVILLE, ALABAMA[a]

Development Destiny	Typical Imperviousness (%)
Single-Family Residential	37
Multi-Family Residential	49
Light Industrial and Institutional	32
Medium Industrial (Research Park)	50
Heavy Industrial and Commercial	88

Reference: TVA (1986).
[a]Use actual development plan to estimate imperviousness when available.

Table 4-10
STREAM REACHES WITH AVAILABLE
MODELED FLOW FREQUENCY DATA

Stream	Reach (miles)	Figure
Pinhook Creek – Existing Channel Conditions	14.63-19.07	4-11
Pinhook Creek—Proposed Channel conditions	14.63-19.07	4-12
East Fork Pinhook Creek	0-1.68	4-13
West Fork Pinhook Creek	0-2.54	4-14
Blue Spring Creek	0-1.90	4-15
Normal Branch	0-3.28	4-16
Dallas Branch	0-2.74	4-17
Dallas Branch Bypass	0-1.37	4-18
Fagan Creek	0-3.16	4-19
Dry Creek	0-2.60	4-20
Broglan Branch – Existing Channel Conditions	0-4.48	4-21
Broglan Branch – Proposed Channel Conditions	0-4.48	4-22
McDonald Creek	3.67 – 9.07	4-23
Tributary to McDonald Creek at Mile 6.14	0-1.00	4-24
Sherwood Branch	0-3.14	4-25
Tributary to Sherwood Branch at Mile 1.42	0.05	4-26
Huntsville Spring Branch – Existing Channel Conditions	9.75-14.63	4-27
Huntsville Spring Branch – Proposed Channel Conditions	9.75-14.63	4-28
Indian Creek	12.80 – 17.52	4-29
Aldridge Creek		
Aldridge Creek Tributaries		
Unnamed Creek Tributary at Mile 3.33	0-0.26	4-31
Mountain Gap at Mile 4.62	0-0.92	4-31
Gourdneck at Mile 3.44	0-0.64	4-31
Camelot at Mile 4.63	0-0.62	4-31
Sunset Cove at Mile 6.09	0-0.75	4-32
Esslinger Cove at Mile 7.33	0-1.01	4-32
Weatherly Cove at Mile 5.94	0-0.70	4-32
Drake Cove at Mile 8.72	0-1.36	4-32
Baily Cove at Mile 9.23	0-1.01	4-33
Toney Hallow at Mile 10.35	0-0.61	4-33
Martin Hallow at Mile 9.87	0-1.11	4-33
Greenwycke Village at Mile 10.83	0-0.86	4-33

Reference: TVA (1986).
Note: Stream Mileages are shown on the drainageway maps

Table 4-11
I_a VALUES FRO RUNOFF CURVE NUMBERS

Curve Number	I_a (inches)	Curve Number	I_a (inches)
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	0.986	97	0.062
68	0.941	98	0.041
69	0.8999		

Reference: USDA, SCS, TR-55 (1986).

Table 4-12
SCS DIMESIONLESS UNIT HYDROGRAPH
RATION AND MASS DATA

Time Ratios (t/t_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Qa/Q)
0	.000	.000
.1	.030	.001
.2	.100	.006
.3	.190	.012
.4	.310	.035
.5	.470	.065
.6	.660	.107
.7	.820	.163
.8	.930	.228
.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.21	.993
3.8	.015	.995
4.0	.011	.997
4.5	.055	.999
5.0	.000	1.000

Reference: USDA, SCS, NEH-4 (1972).

Table 4-13
BALANCED STORM HEYTOGRAPH AND CUMULATIVE
MASS CURVE COMPUTATION, EXAMPLE 4-8

Time, T (hours)	Intensity, i (inches/hour)	Rainfall, Depth (inches)	Icrem. Depth (inches)	Balanced Depth (inches)	Cumulative Depth (inches)	Rainfall Excess (inches)	Rainfall Hyetograph (inches)
0.00	9.50	0.00	0.00	0.00	0.00	0.00	0.00
0.08	8.15	0.65	0.65	0.01	0.01	0.00	0.00
0.16	6.80	1.09	0.44	0.03	0.04	0.00	0.00
0.24	5.92	1.42	0.33	0.08	0.12	0.00	0.00
0.32	5.25	1.68	0.26	0.09	0.20	0.00	0.00
0.40	4.80	1.92	0.24	0.10	0.30	0.00	0.00
0.48	4.38	2.10	0.18	0.11	0.41	0.00	0.00
0.56	4.04	2.26	0.16	0.10	0.51	0.00	0.00
0.64	3.80	2.43	0.17	0.12	0.63	0.00	0.00
0.72	3.57	2.57	0.14	0.17	0.80	0.00	0.00
0.80	3.36	2.69	0.12	0.18	0.98	0.02	0.02
0.88	3.18	2.80	0.11	0.26	1.24	0.07	0..05
0.96	3.02	2.90	0.10	0.44	1.68	0.21	0.14
1.04	2.89	3.01	0.11	0.65	2.33	0.51	0.30
1.12	2.78	3.11	0.11	0.33	2.66	0.70	0.19
1.20	2.68	3.22	0.10	0.24	2.90	0.85	0.15
1.28	2.59	3.32	0.10	0.16	3.06	0.95	0.10
1.36	2.51	3.41	0.10	0.14	3.20	1.04	0.09
1.44	2.43	3.50	0.09	0.11	3.31	1.11	0.07
1.52	2.36	3.59	0.09	0.11	3.42	1.18	0.07
1.60	2.29	3.66	0.08	0.10	3.52	1.25	0.07
1.68	2.21	3.71	0.05	0.10	3.62	1.32	0.07
1.76	2.13	3.74	0.03	0.09	3.70	7.38	0.06
1.84	2.05	3.76	0.02	0.05	3.75	1.42	0.04
1.92	1.96	3.76	0.01	0.02	3.77	1.43	0.01
2.00	1.89	3.78	0.01	0.01	3.78	1.44	0.01

Table 4-14
HYDROGRAPH COMPUTATION, EXAMPLE 4-8

Unit Hydrograph (cfs/inch)	0	10	31	65	92	99	92	77	55	39	28	20	15	11	8	5	4	3	2	1	1	1	0	0	0	0
Runoff time[a] (minutes)	0	5	10	14	19	24	29	34	38	43	48	53	58	62	67	72	77	82	86	91	96	101	106	110	115	120
Incremental Rainfall Excess (inches)																										
0.02	0	0.2	0.6	1.2	1.6	1.8	1.6	1.4	1.0	0.7	0.5	0.4	0.3	0.2	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.05		0.0	0.5	1.6	3.3	4.7	5.0	4.7	3.9	2.8	2.0	1.4	1.0	0.8	0.6	0.4	0.3	0.2	0.2	0.1	0.1	0.1	0.1	0.0	0.0	0.0
0.14			0.0	1.4	4.3	9.1	12.9	13.9	12.9	10.8	7.7	5.5	3.9	2.8	2.1	1.4	1.1	0.7	0.6	0.4	0.3	0.1	0.1	0.1	0.0	0.0
0.30				0.0	3.0	9.4	19.7	27.9	30.0	27.9	23.3	16.7	11.7	8.5	6.1	4.5	3.3	2.4	1.5	1.2	0.9	0.6	0.3	0.3	0.3	0.0
0.19					0.0	1.9	5.8	12.1	17.2	18.5	17.2	14.4	10.3	7.3	5.2	3.7	2.8	2.1	1.5	0.9	0.7	0.6	0.4	0.2	0.2	0.2
0.15						0.0	1.5	4.5	9.4	13.4	14.4	13.4	11.2	8.0	5.7	4.1	2.9	2.2	1.6	1.2	0.7	0.6	0.4	0.3	0.1	0.1
0.10							0.0	1.0	3.1	6.6	9.3	10.0	9.3	7.8	5.6	3.9	2.8	2.0	1.5	1.1	0.8	0.5	0.4	0.3	0.2	0.1
0.09								0.0	0.9	2.8	5.9	8.3	8.9	8.3	6.9	5.0	3.5	2.5	1.8	1.4	1.0	0.7	0.5	0.4	0.3	0.2
0.07									0.0	0.7	2.3	4.8	6.8	7.3	6.8	5.7	4.0	2.9	2.1	1.5	1.1	0.8	0.6	0.4	0.3	0.2
0.07										0.0	0.7	2.2	4.7	6.6	7.1	6.6	5.5	4.0	2.8	2.0	1.4	1.1	0.8	0.6	0.4	0.3
0.07											0.0	0.7	2.2	4.6	6.5	7.0	6.5	5.4	3.9	2.7	2.0	1.4	1.1	0.8	0.6	0.4
0.07												0.0	0.7	2.1	4.5	6.3	6.8	6.3	5.3	3.8	2.7	1.9	1.4	1.0	.08	0.5
0.06													0.0	0.6	1.9	4.0	5.7	6.1	5.7	4.8	3.4	2.4	1.7	1.2	0.9	0.7
0.04														0.0	0.4	1.1	2.3	3.3	3.5	3.3	2.7	2.0	1.4	1.0	0.7	0.5
0.01															0.0	0.1	0.4	0.9	1.3	1.4	1.3	1.1	0.8	1.6	0.4	0.3
0.01																0.0	0.1	0.2	0.5	0.7	0.7	0.7	0.6	0.4	0.3	0.2
Total Runoff (cfs)	0	0	1	4	12	27	47	65	78	84	83	78	71	65	59	54	48	41	33	26	19	14	10	7	5	4
																										Total = 935

[a]Time zero is when runoff first begins

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

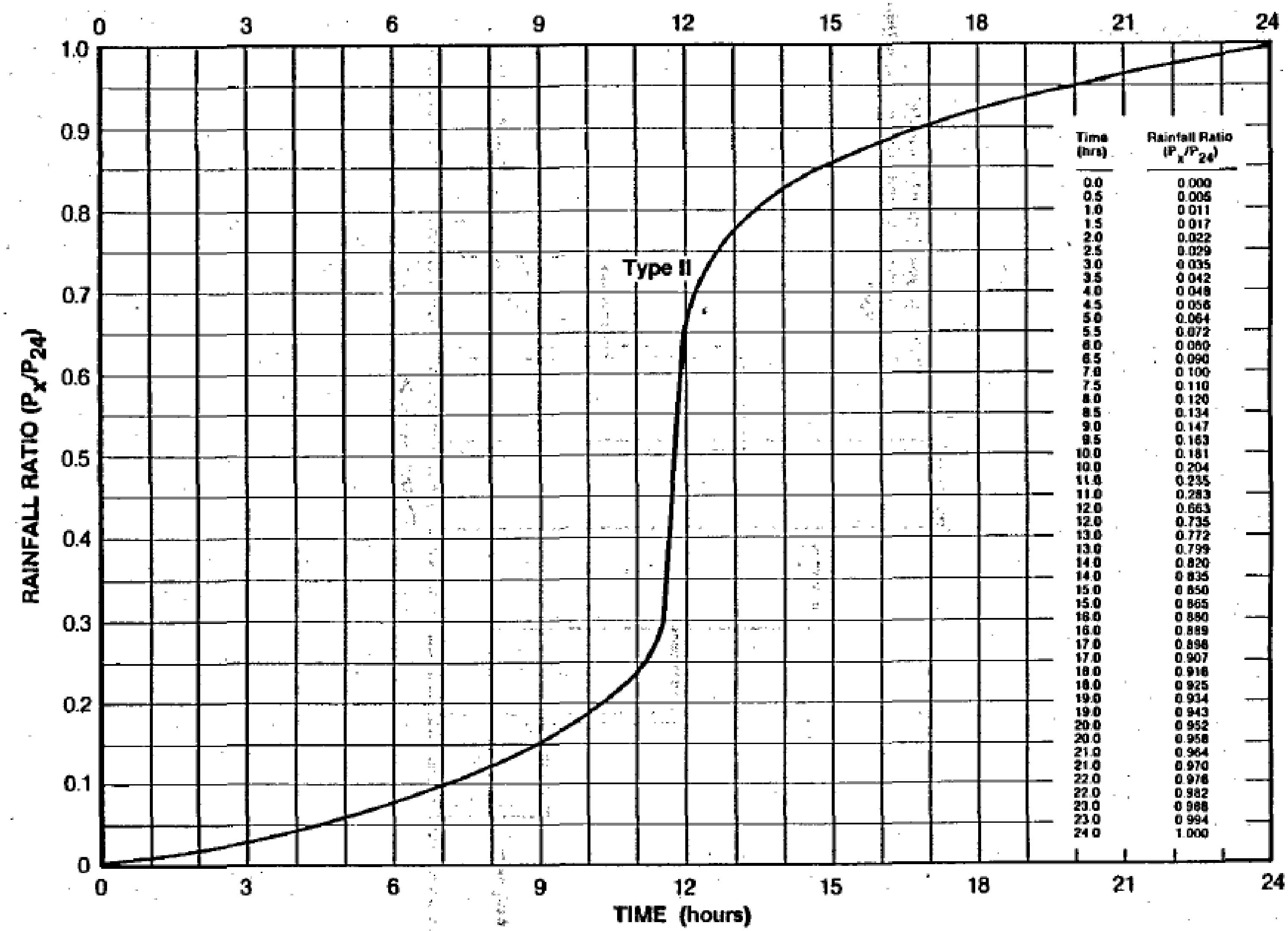
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.400 (0.325-0.499)	0.468 (0.379-0.584)	0.582 (0.470-0.729)	0.682 (0.546-0.858)	0.826 (0.638-1.08)	0.942 (0.708-1.25)	1.06 (0.767-1.45)	1.19 (0.818-1.67)	1.37 (0.897-1.98)	1.51 (0.957-2.21)
10-min	0.586 (0.475-0.731)	0.685 (0.554-0.855)	0.853 (0.688-1.07)	0.999 (0.800-1.26)	1.21 (0.935-1.58)	1.38 (1.04-1.83)	1.56 (1.12-2.12)	1.74 (1.20-2.45)	2.00 (1.31-2.89)	2.20 (1.40-3.23)
15-min	0.715 (0.580-0.891)	0.835 (0.676-1.04)	1.04 (0.839-1.30)	1.22 (0.976-1.53)	1.48 (1.14-1.93)	1.68 (1.26-2.24)	1.90 (1.37-2.59)	2.13 (1.46-2.98)	2.44 (1.60-3.53)	2.69 (1.71-3.94)
30-min	1.03 (0.836-1.29)	1.20 (0.971-1.50)	1.49 (1.20-1.86)	1.74 (1.39-2.19)	2.11 (1.63-2.77)	2.41 (1.81-3.20)	2.72 (1.97-3.72)	3.05 (2.10-4.29)	3.52 (2.31-5.08)	3.88 (2.47-5.69)
60-min	1.39 (1.13-1.74)	1.60 (1.30-2.00)	1.97 (1.59-2.46)	2.28 (1.83-2.87)	2.74 (2.12-3.59)	3.11 (2.34-4.14)	3.50 (2.53-4.78)	3.91 (2.69-5.49)	4.48 (2.94-6.48)	4.93 (3.14-7.22)
2-hr	1.75 (1.43-2.16)	2.01 (1.64-2.48)	2.44 (1.99-3.03)	2.82 (2.28-3.52)	3.37 (2.63-4.37)	3.82 (2.90-5.02)	4.28 (3.12-5.77)	4.77 (3.31-6.62)	5.45 (3.62-7.78)	5.98 (3.85-8.66)
3-hr	2.01 (1.65-2.47)	2.29 (1.88-2.81)	2.77 (2.26-3.41)	3.19 (2.59-3.95)	3.78 (2.97-4.87)	4.26 (3.25-5.56)	4.76 (3.49-6.38)	5.29 (3.70-7.28)	6.01 (4.02-8.52)	6.57 (4.26-9.46)
6-hr	2.46 (2.04-2.99)	2.81 (2.32-3.42)	3.39 (2.80-4.14)	3.90 (3.19-4.78)	4.61 (3.64-5.86)	5.18 (3.99-6.68)	5.77 (4.27-7.64)	6.38 (4.51-8.69)	7.22 (4.88-10.1)	7.87 (5.17-11.2)
12-hr	2.93 (2.45-3.53)	3.37 (2.81-4.06)	4.11 (3.41-4.97)	4.74 (3.91-5.75)	5.63 (4.48-7.08)	6.33 (4.92-8.08)	7.06 (5.28-9.23)	7.81 (5.58-10.5)	8.83 (6.04-12.2)	9.62 (6.40-13.5)
24-hr	3.45 (2.90-4.11)	3.98 (3.35-4.75)	4.87 (4.08-5.83)	5.63 (4.68-6.76)	6.69 (5.37-8.31)	7.52 (5.90-9.49)	8.37 (6.32-10.8)	9.25 (6.68-12.3)	10.4 (7.23-14.3)	11.4 (7.65-15.8)
2-day	4.04 (3.43-4.77)	4.64 (3.94-5.49)	5.64 (4.76-6.68)	6.47 (5.43-7.70)	7.63 (6.19-9.37)	8.54 (6.76-10.6)	9.46 (7.22-12.1)	10.4 (7.59-13.7)	11.7 (8.17-15.8)	12.6 (8.61-17.3)
3-day	4.44 (3.79-5.21)	5.06 (4.31-5.94)	6.08 (5.17-7.16)	6.94 (5.86-8.21)	8.13 (6.63-9.92)	9.06 (7.21-11.2)	10.0 (7.68-12.7)	11.0 (8.06-14.3)	12.3 (8.65-16.4)	13.2 (9.10-18.1)
4-day	4.78 (4.10-5.59)	5.42 (4.63-6.33)	6.46 (5.50-7.57)	7.33 (6.21-8.63)	8.54 (6.99-10.4)	9.49 (7.59-11.7)	10.4 (8.06-13.2)	11.4 (8.44-14.8)	12.7 (9.03-17.0)	13.7 (9.48-18.6)
7-day	5.70 (4.91-6.60)	6.37 (5.49-7.39)	7.48 (6.42-8.70)	8.40 (7.17-9.81)	9.67 (7.97-11.6)	10.6 (8.58-13.0)	11.6 (9.05-14.5)	12.6 (9.41-16.2)	13.9 (10.00-18.4)	14.9 (10.4-20.1)
10-day	6.51 (5.64-7.51)	7.23 (6.25-8.34)	8.39 (7.23-9.70)	9.35 (8.01-10.9)	10.7 (8.83-12.7)	11.7 (9.45-14.1)	12.7 (9.91-15.7)	13.7 (10.3-17.4)	15.0 (10.8-19.7)	16.0 (11.2-21.4)
20-day	8.88 (7.76-10.1)	9.70 (8.47-11.1)	11.0 (9.58-12.6)	12.1 (10.4-13.9)	13.5 (11.3-15.9)	14.6 (11.9-17.4)	15.6 (12.3-19.1)	16.6 (12.6-20.9)	18.0 (13.1-23.2)	18.9 (13.5-25.0)
30-day	11.0 (9.63-12.4)	11.9 (10.4-13.5)	13.4 (11.7-15.2)	14.6 (12.7-16.7)	16.1 (13.5-18.8)	17.3 (14.2-20.5)	18.4 (14.6-22.3)	19.4 (14.8-24.2)	20.7 (15.2-26.5)	21.6 (15.5-28.3)
45-day	13.7 (12.1-15.5)	14.9 (13.1-16.8)	16.7 (14.7-18.9)	18.1 (15.8-20.5)	19.8 (16.7-22.9)	21.1 (17.4-24.8)	22.2 (17.8-26.7)	23.3 (17.9-28.7)	24.5 (18.1-31.1)	25.3 (18.3-32.8)
60-day	16.2 (14.3-18.2)	17.6 (15.6-19.7)	19.7 (17.4-22.2)	21.3 (18.7-24.1)	23.3 (19.7-26.7)	24.6 (20.4-28.7)	25.8 (20.7-30.8)	26.8 (20.8-32.9)	27.9 (20.9-35.2)	28.6 (21.0-37.0)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

FIGURE 4-1
Intensity-Duration-Frequency Curves and Depth-Duration-Frequency Data for Huntsville



Time (hrs)	Rainfall Ratio (P_x/P_{24})
0.0	0.000
0.5	0.005
1.0	0.011
1.5	0.017
2.0	0.022
2.5	0.029
3.0	0.035
3.5	0.042
4.0	0.048
4.5	0.056
5.0	0.064
5.5	0.072
6.0	0.080
6.5	0.090
7.0	0.100
7.5	0.110
8.0	0.120
8.5	0.134
9.0	0.147
9.5	0.163
10.0	0.181
10.0	0.204
11.0	0.235
11.0	0.283
12.0	0.663
12.0	0.735
13.0	0.772
13.0	0.799
14.0	0.820
14.0	0.835
15.0	0.850
15.0	0.865
16.0	0.880
16.0	0.889
17.0	0.898
17.0	0.907
18.0	0.916
18.0	0.925
19.0	0.934
19.0	0.943
20.0	0.952
20.0	0.958
21.0	0.964
21.0	0.970
22.0	0.976
22.0	0.982
23.0	0.988
23.0	0.994
24.0	1.000

Reference: USDA, SCS, NEH-4 (1972).

FIGURE 4-2
SCS Type II Cumulative Rainfall Distribution

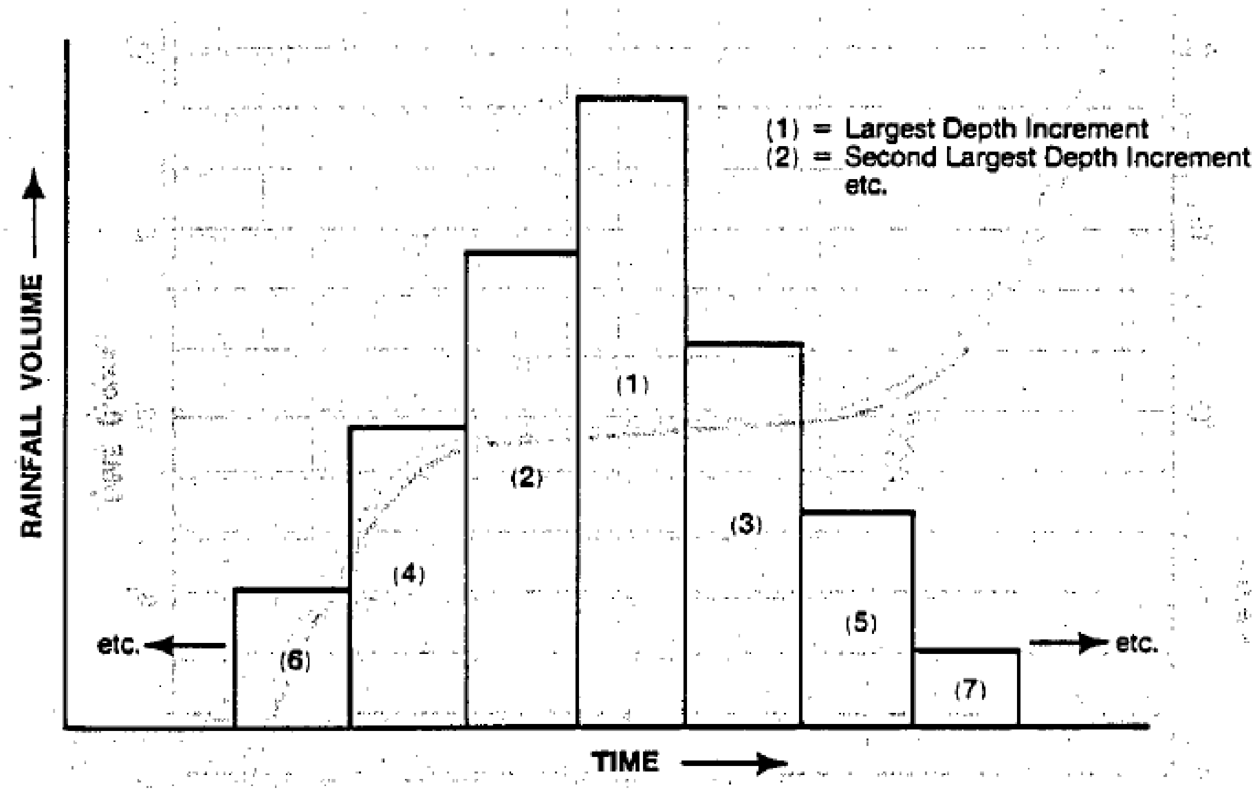
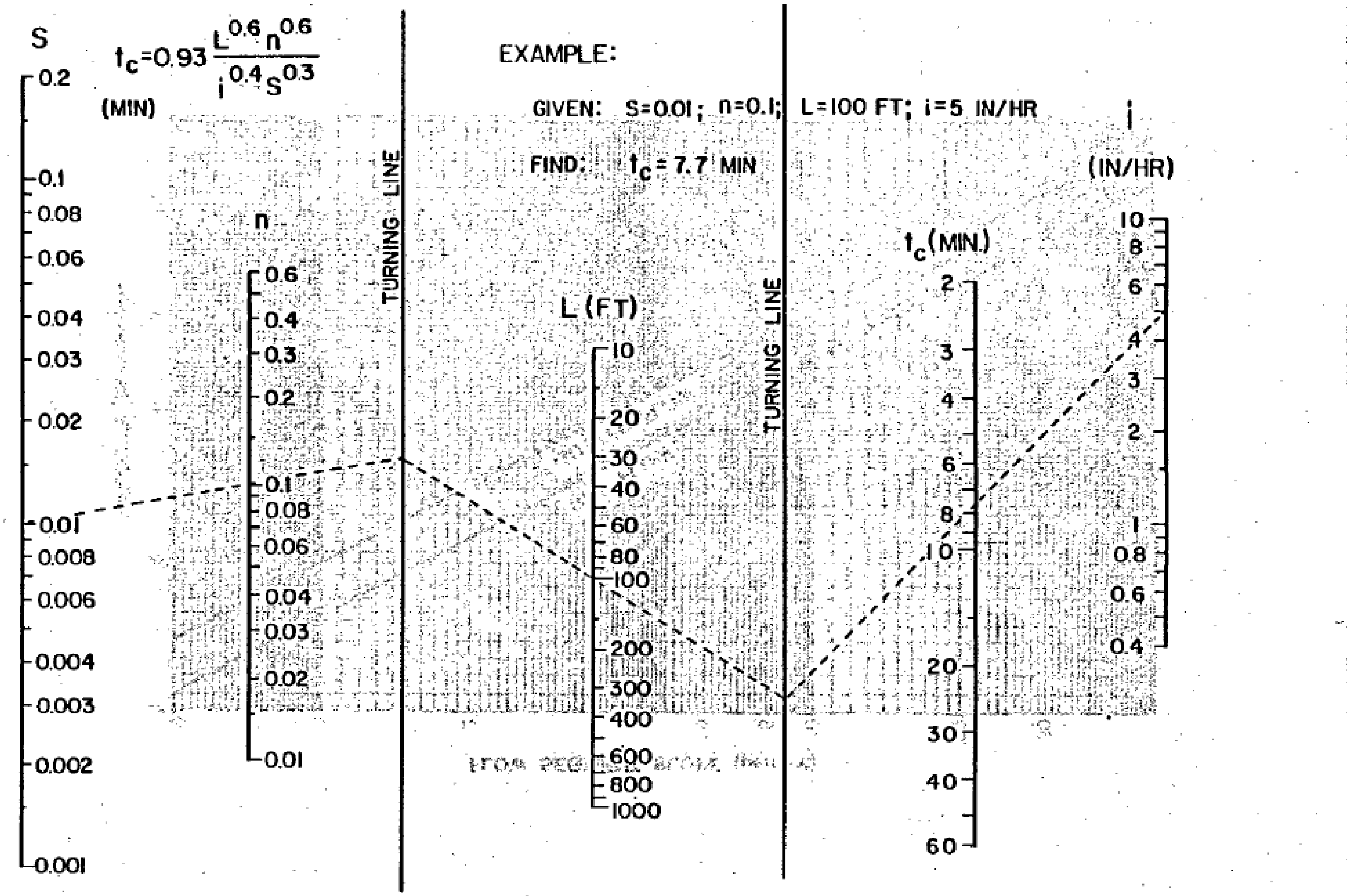


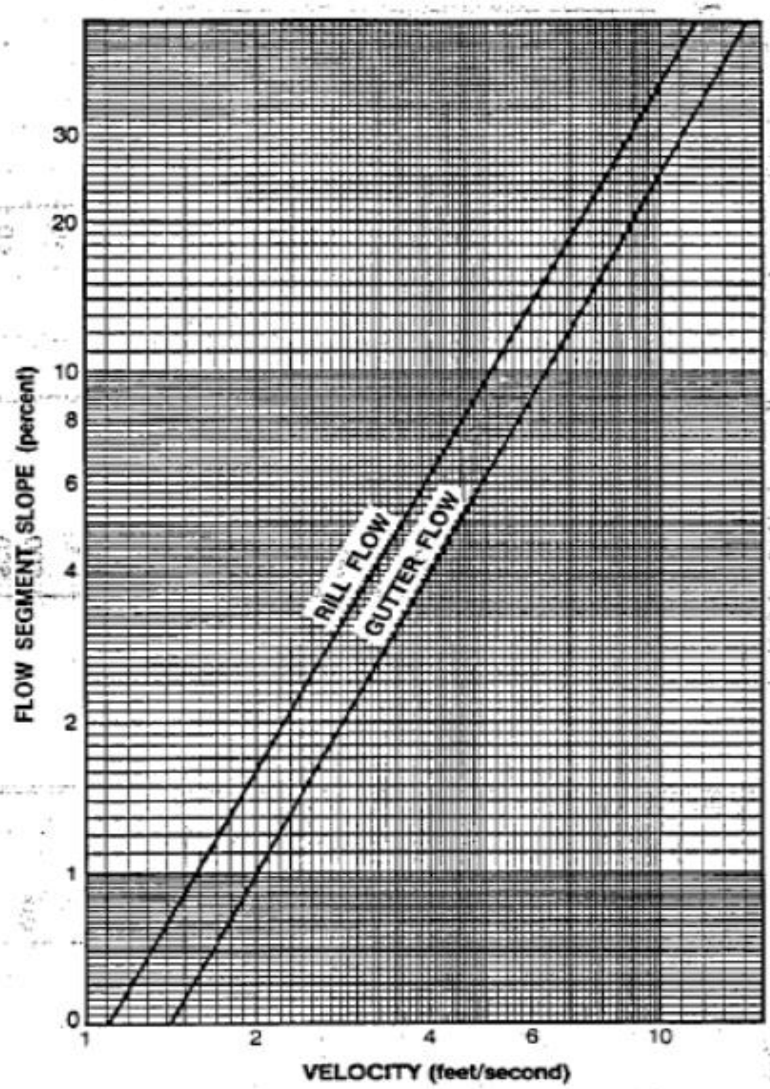
FIGURE 4-3
Balanced Storm Approach for Developing a Design Storm Hyetograph



Reference: USDOT, FHWA, HEC-12 (1984).

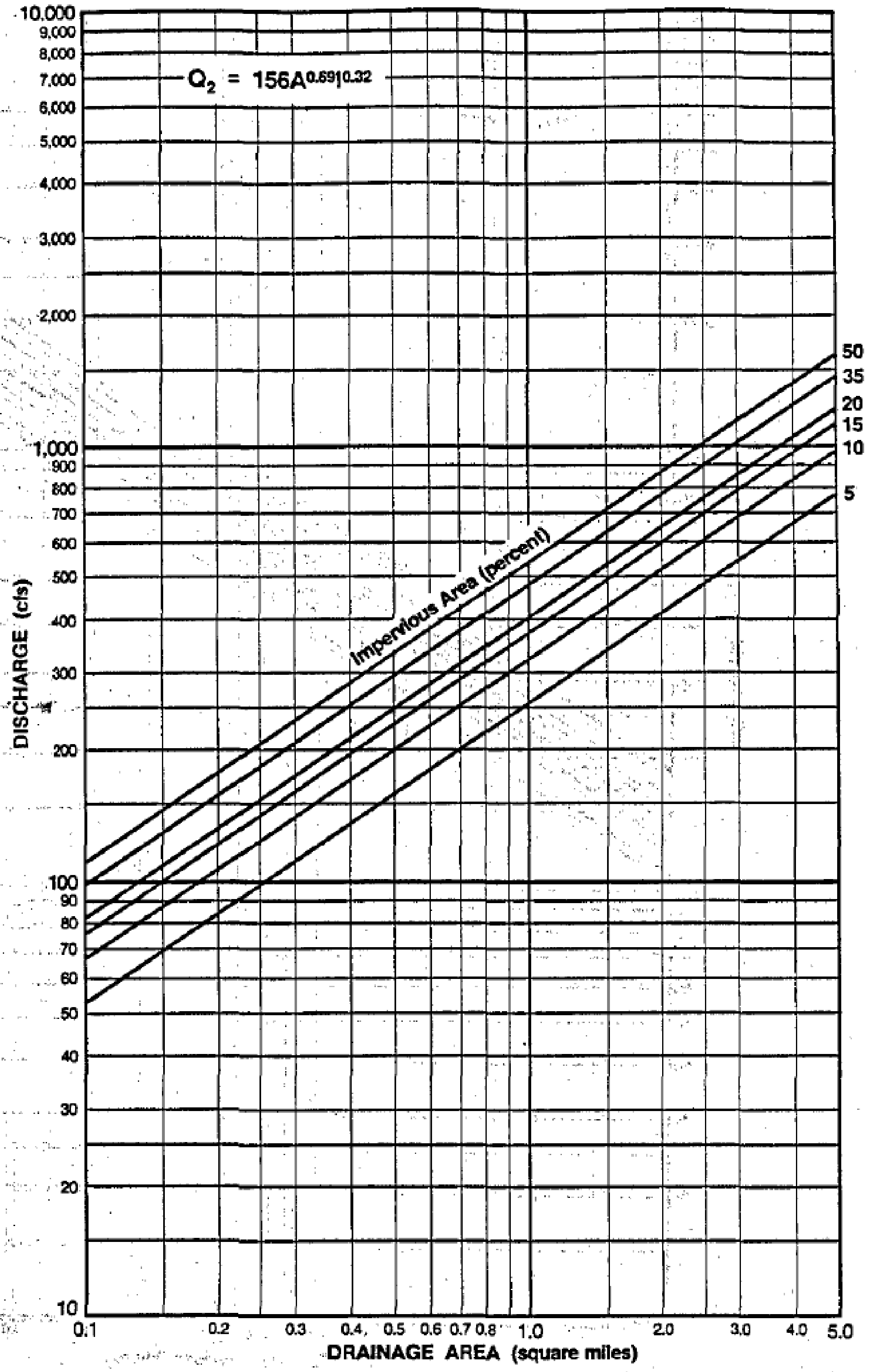
FIGURE 4-4
Kinematic Wave Nomograph for Estimating Overland Flow Travel Time

Reference: USDA, SCS, TR-55 (1986).



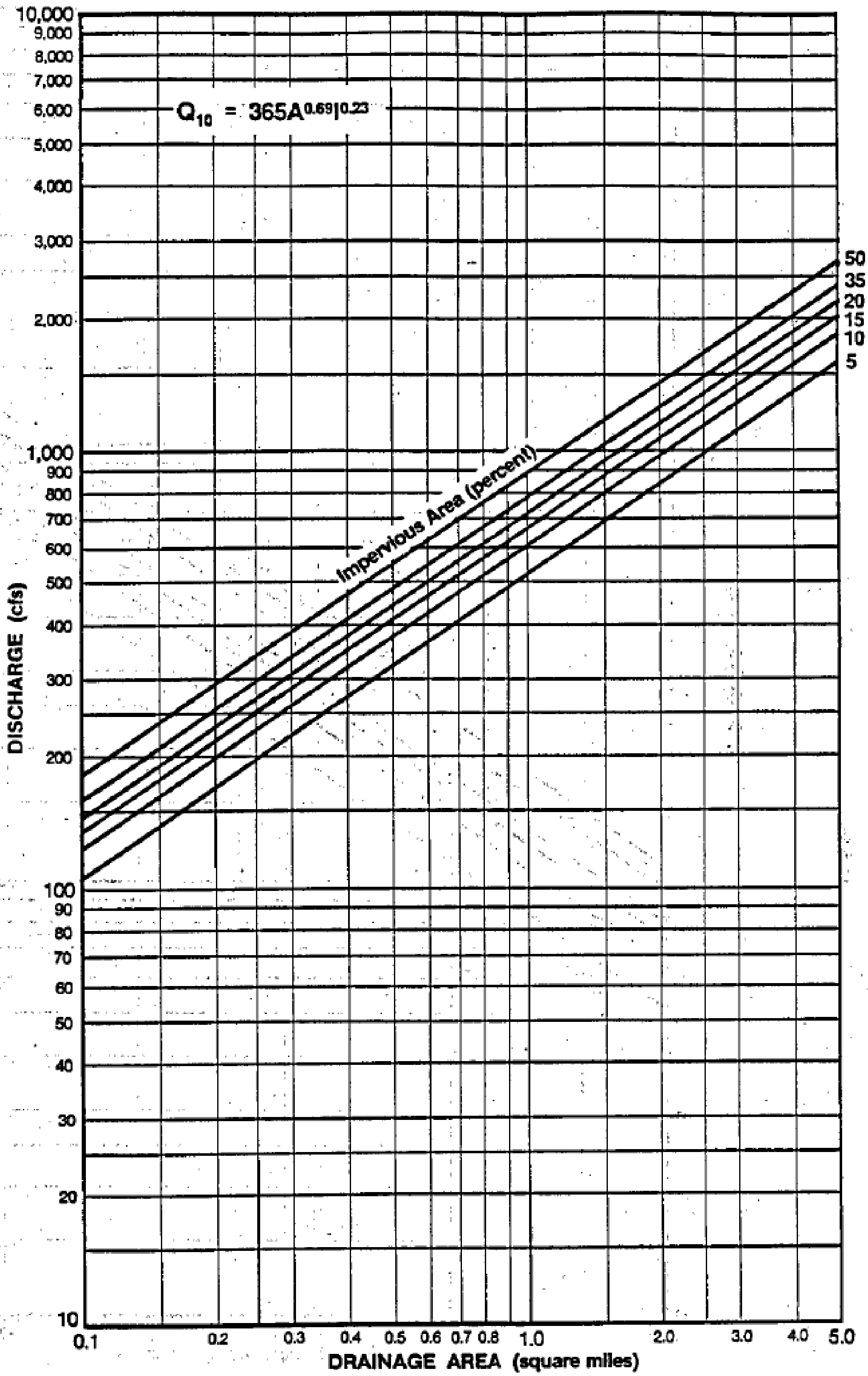
Reference: USDA, SCS, TR-55 (1986).

FIGURE 4-5
Average Velocities for Estimating Travel Time for Shallow Channel Flow



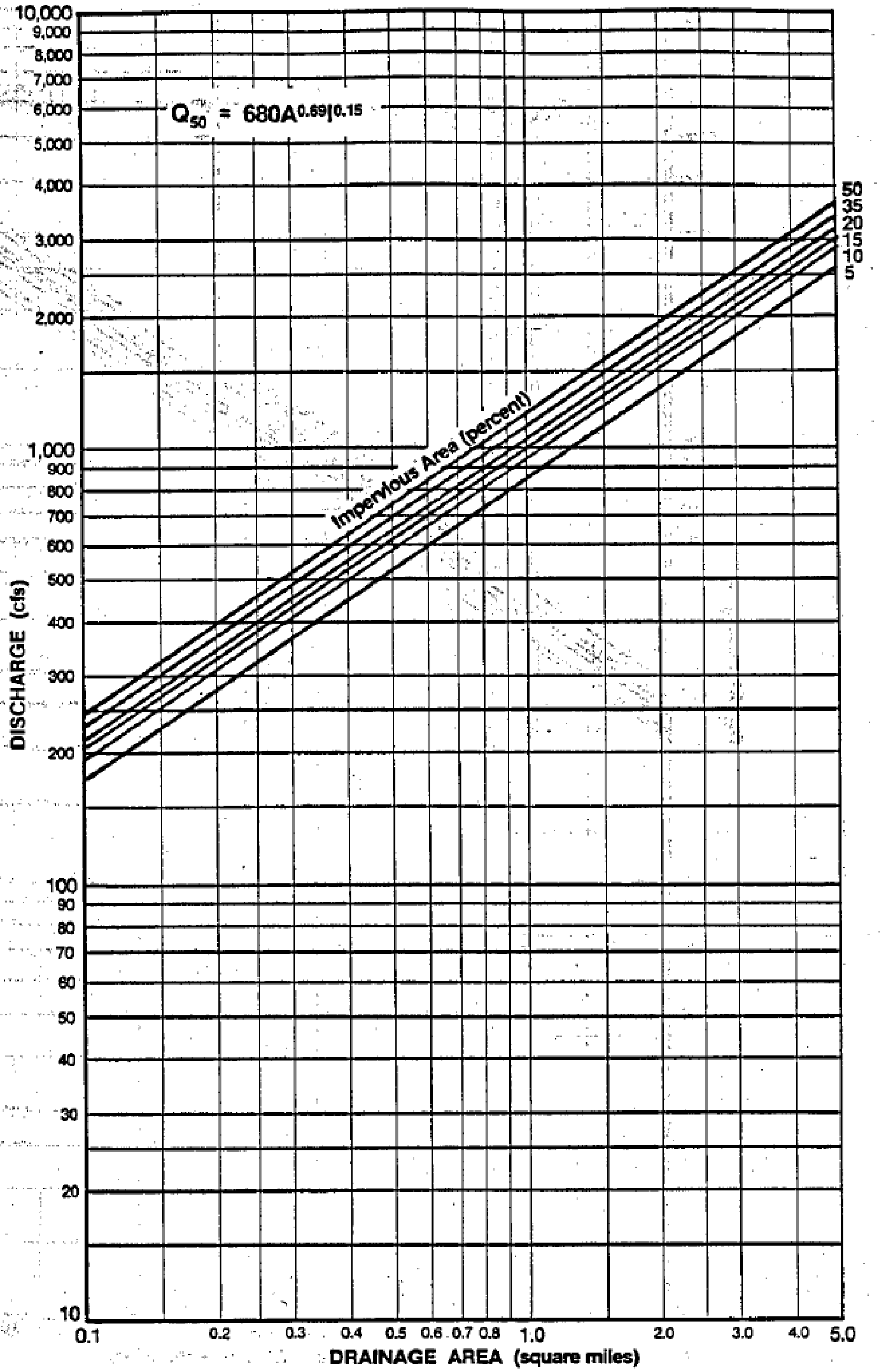
Reference: TVA (1986).

FIGURE 4-6
Graphical Solution to the 2-Year Return Frequency Peak Discharge Regression Equation



Reference: TVA (1986).

FIGURE 4-7
Graphical Solution to the 10-Year Return Frequency Peak Discharge Regression Equation



Reference: TVA (1986).

FIGURE 4-8
Graphical Solution to the 50-Year Return Frequency Peak Discharge Regression Equation

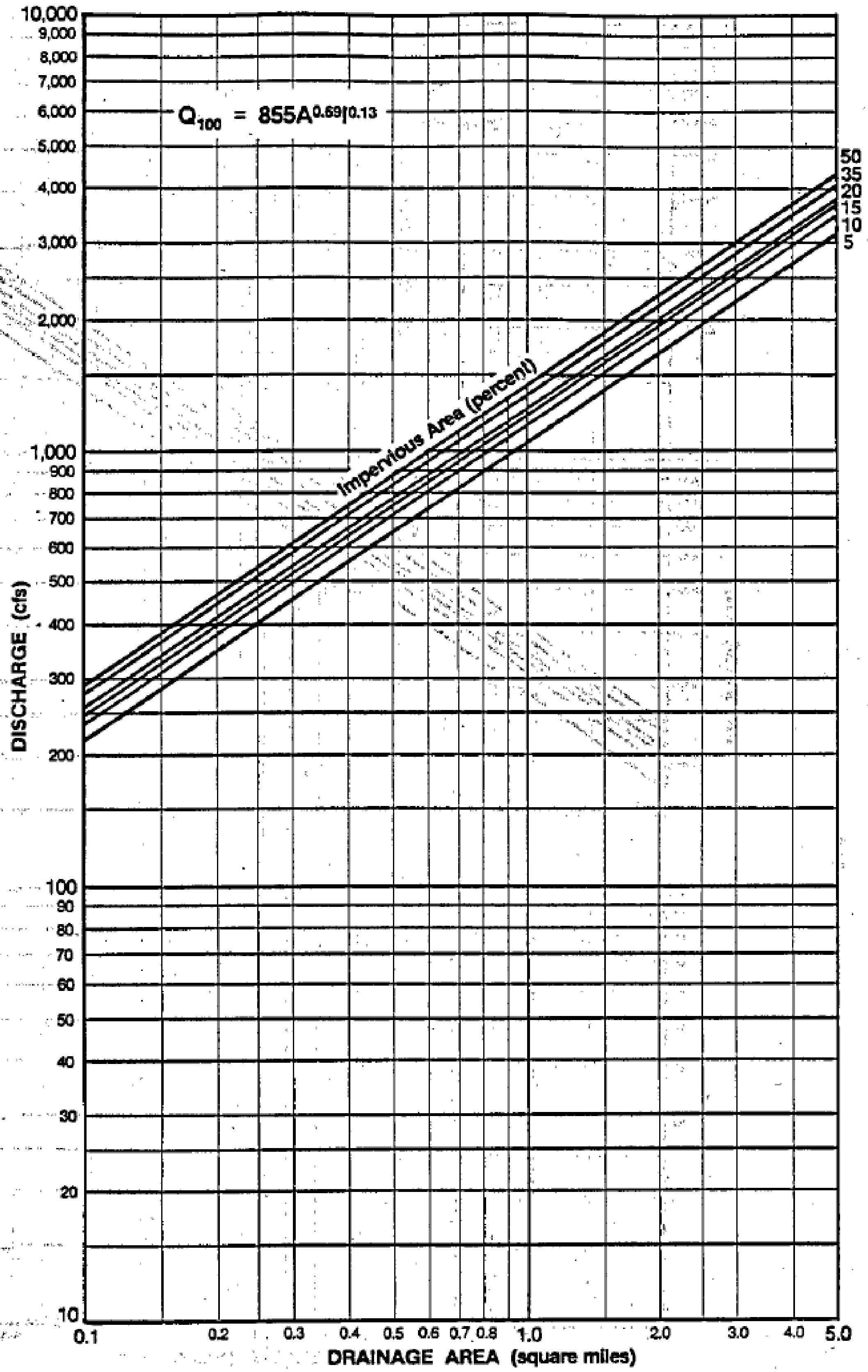


FIGURE 4-9
Graphical Solution to the 100-Year Return Frequency Peak Discharge Regression Equation

Reference: TVA (1986).

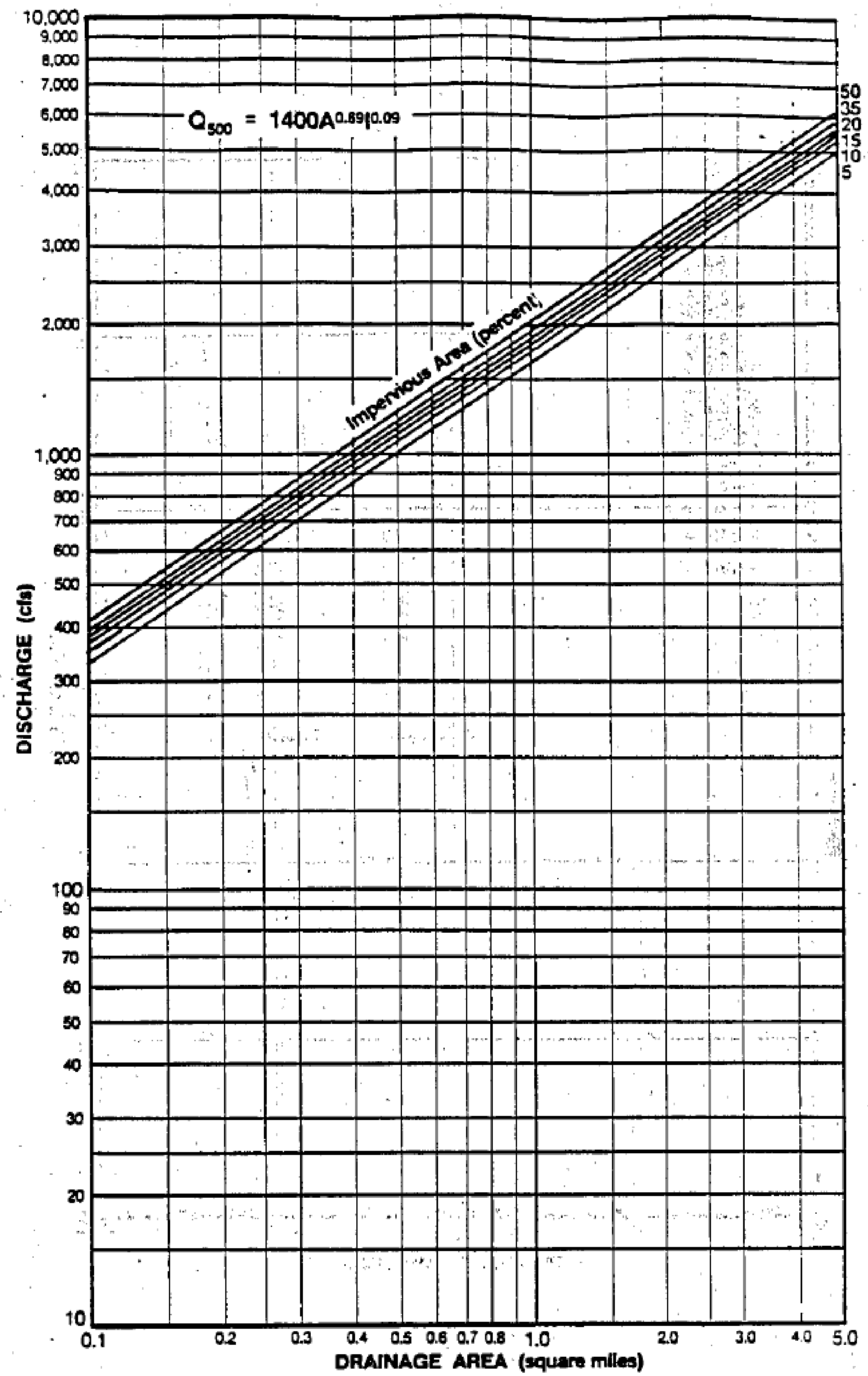
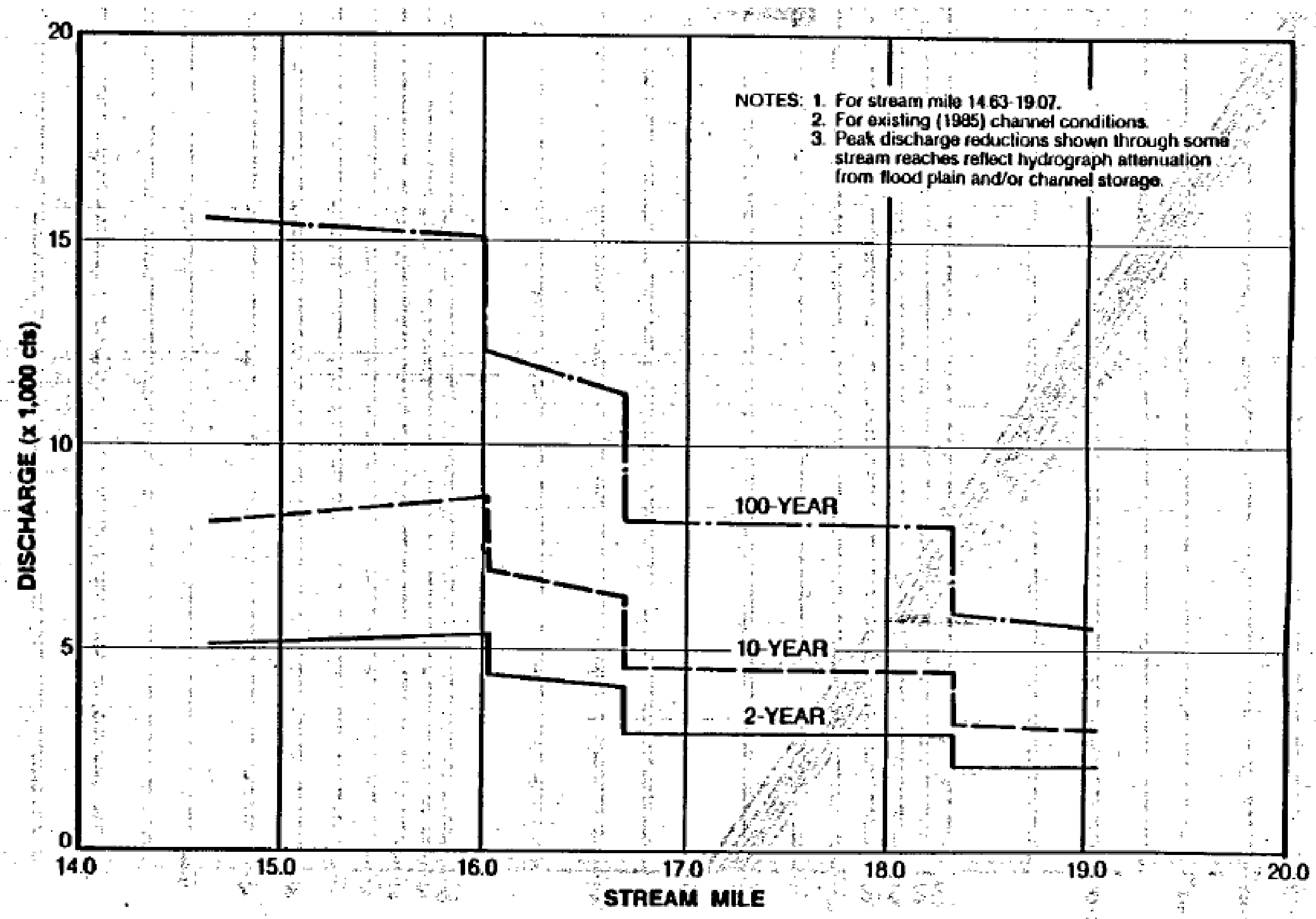


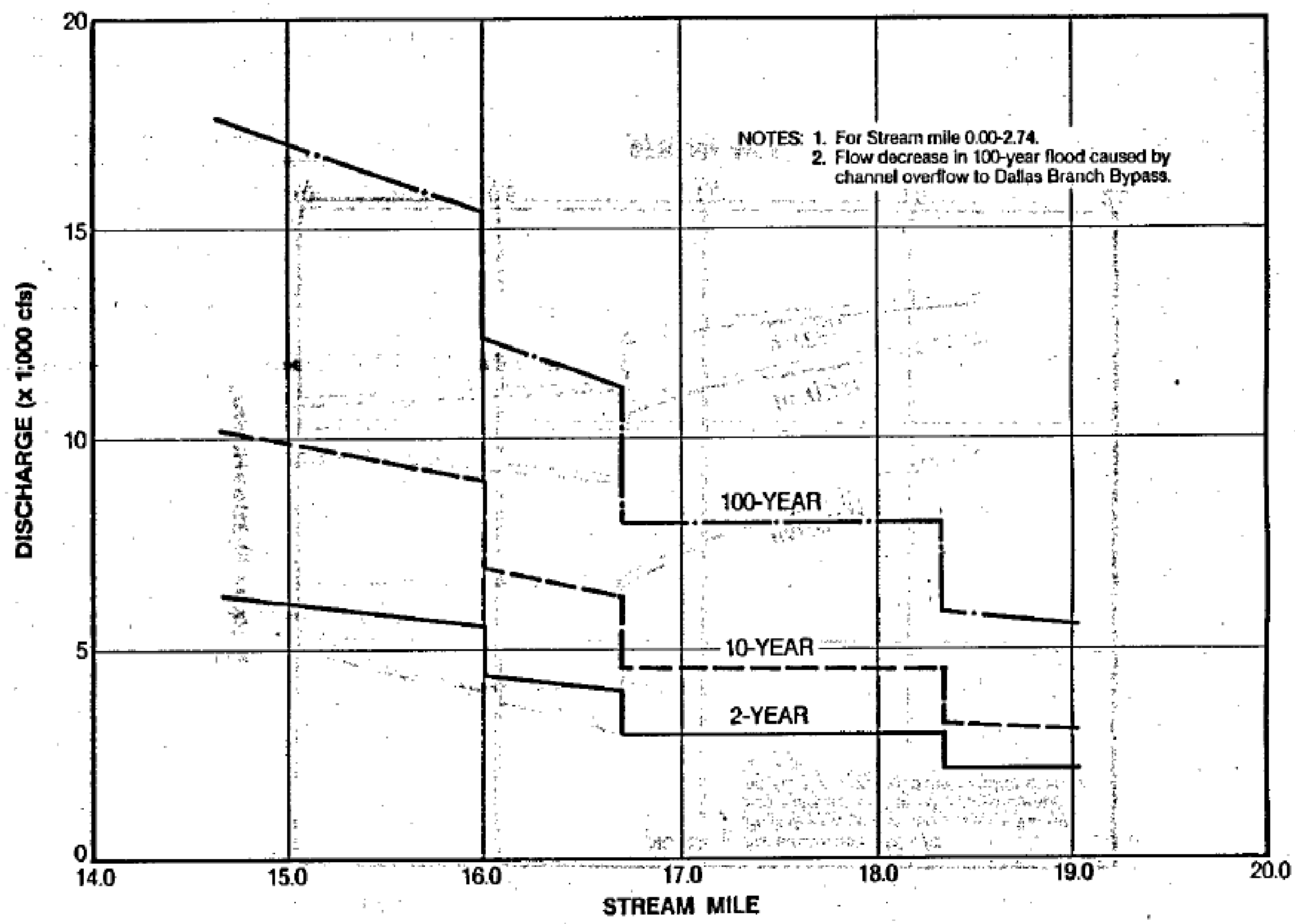
FIGURE 4-10
Graphical Solution to the 500-Year Return Frequency Peak Discharge Regression Equation

Reference: TVA (1986).



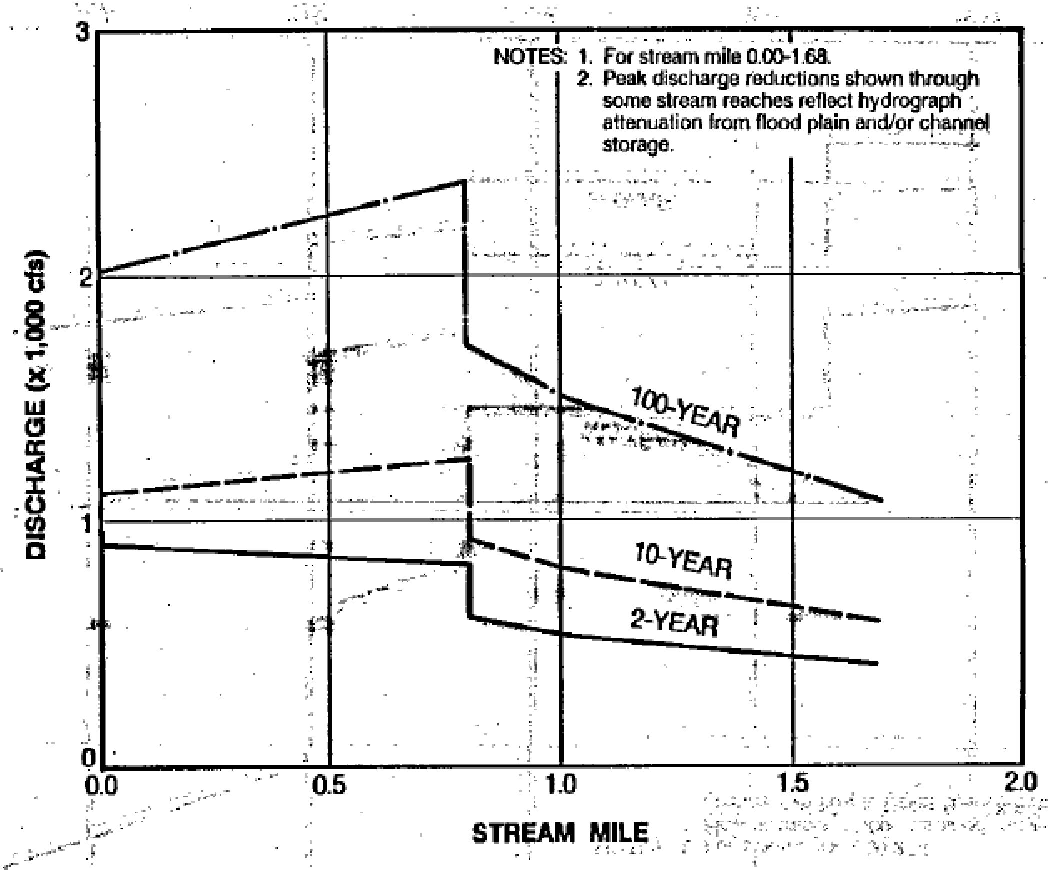
Reference TVA (1986)

FIGURE 4-11
Flow Frequency Estimates for Pinhook Creek, Existing Channel



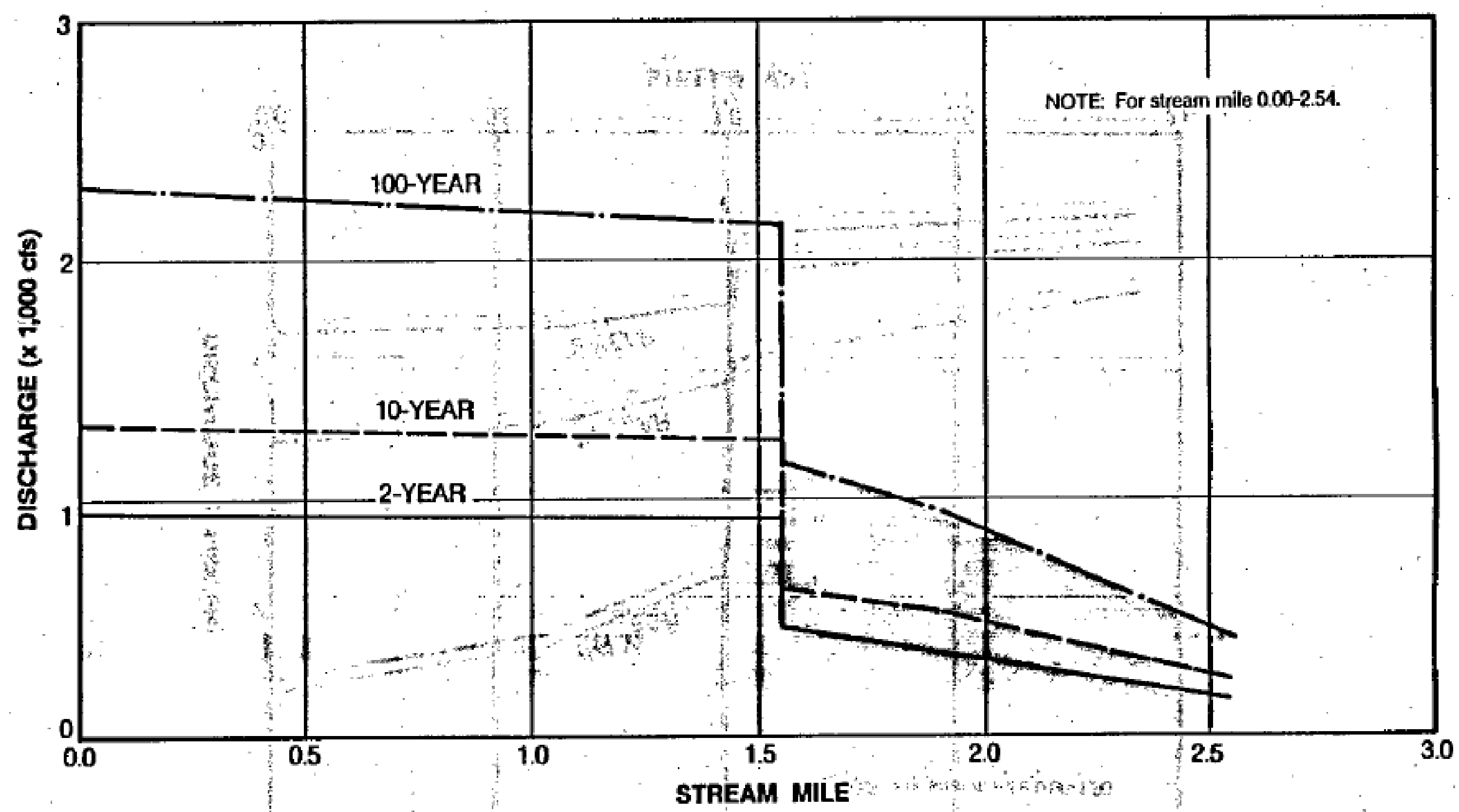
Reference: TVA (1986).

FIGURE 4-12
Flow Frequency Estimates for Pinhook Creek, Proposed Channel



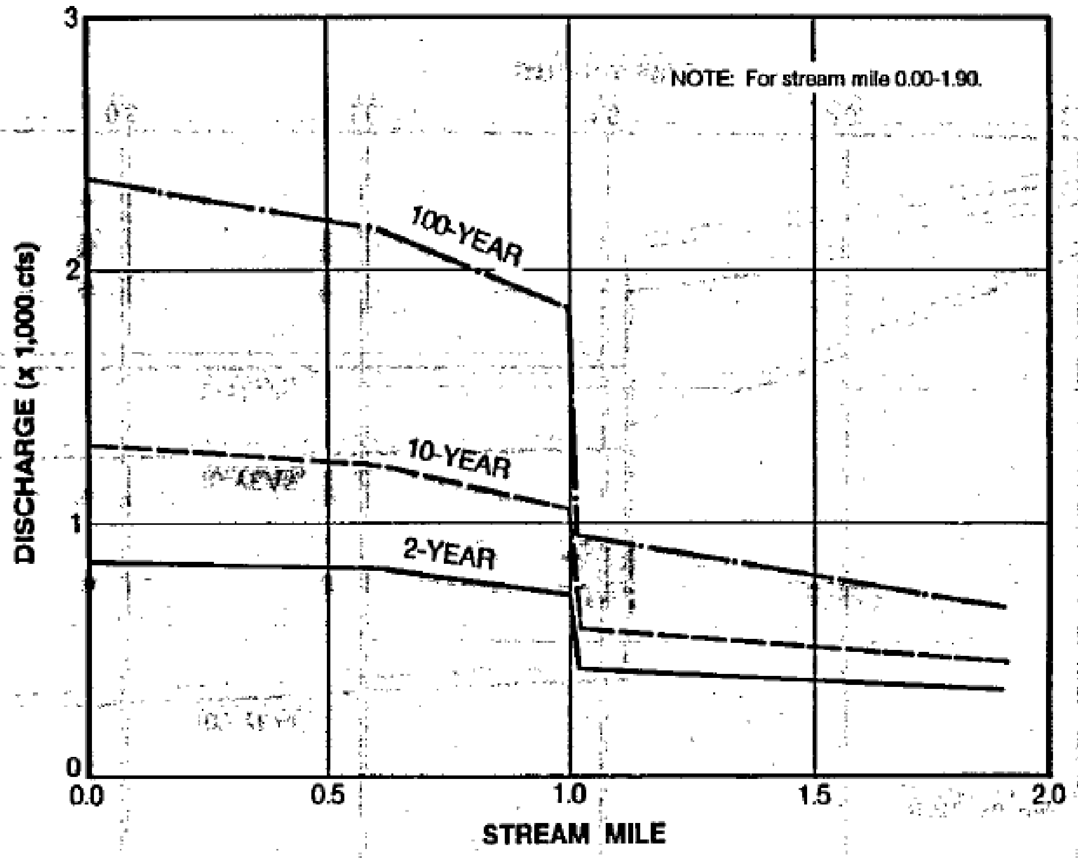
Reference: TVA (1986).

FIGURE 4-13
Flow Frequency Estimates for East Fork Pinhook Creek



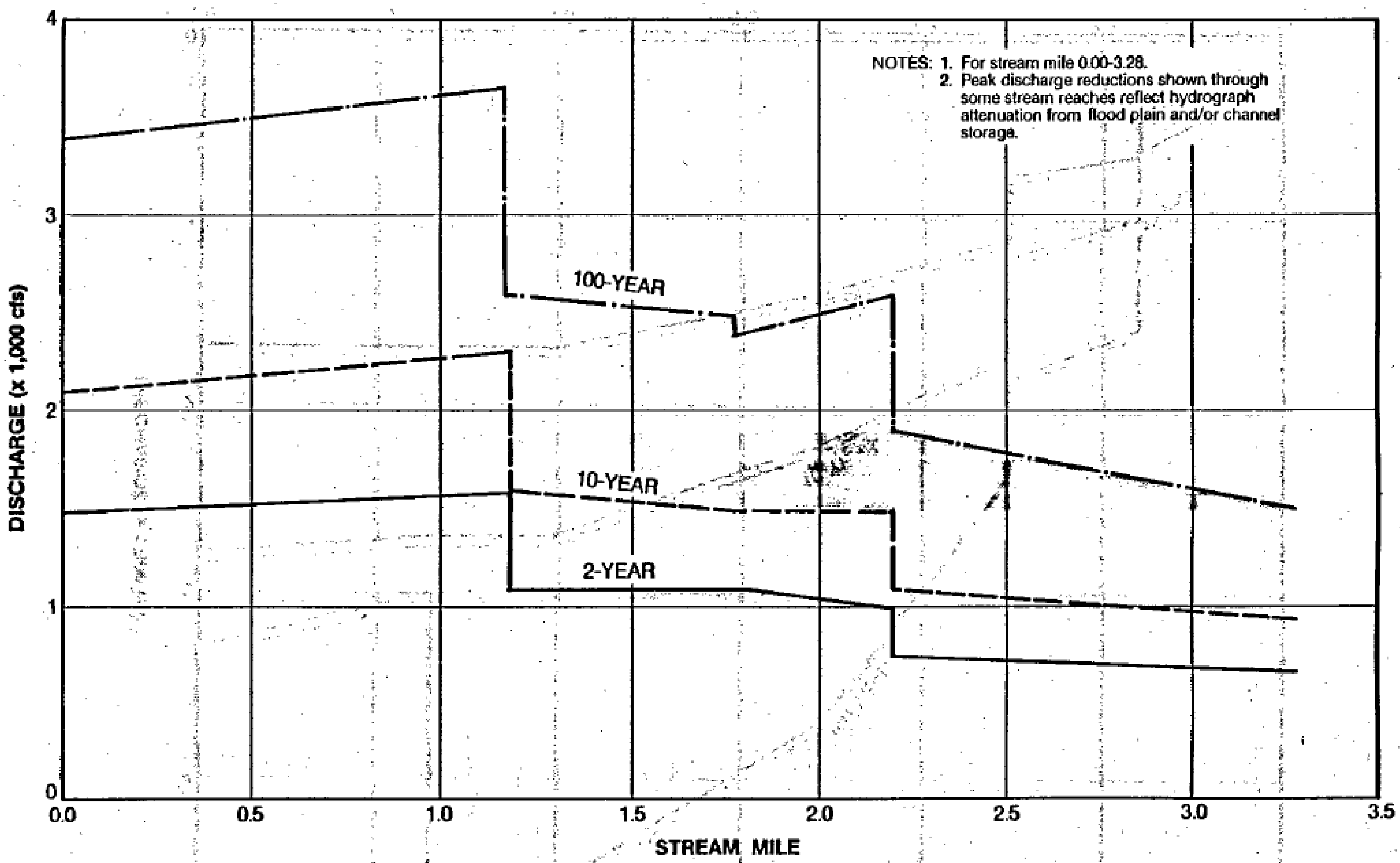
Reference: TVA (1986).

FIGURE 4-14
Flow Frequency Estimates for West Fork Pinhook Creek



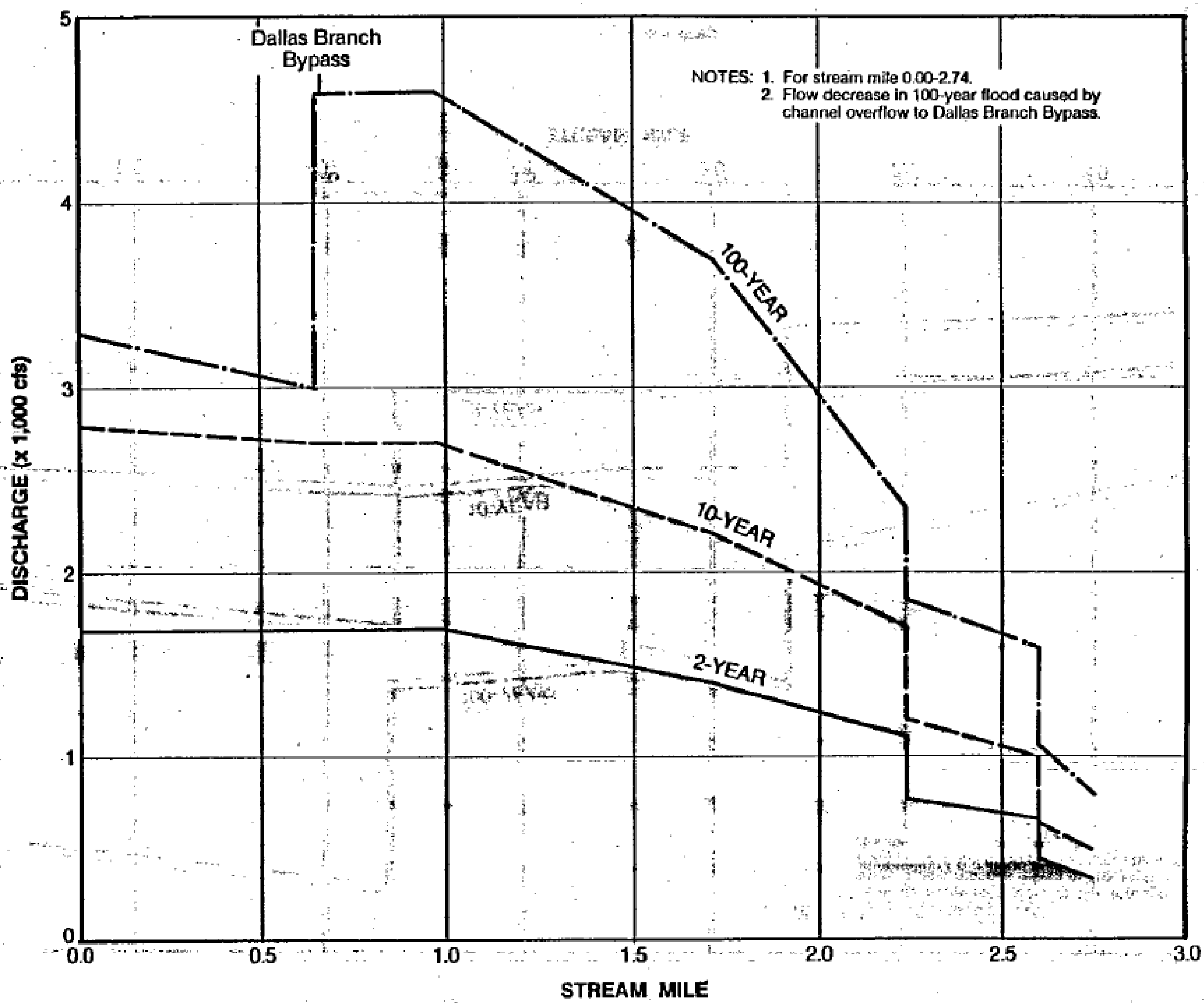
Reference: TVA (1986).

FIGURE 4-15
Flow Frequency Estimates for Blue Spring Creek



Reference: TVA (1986).

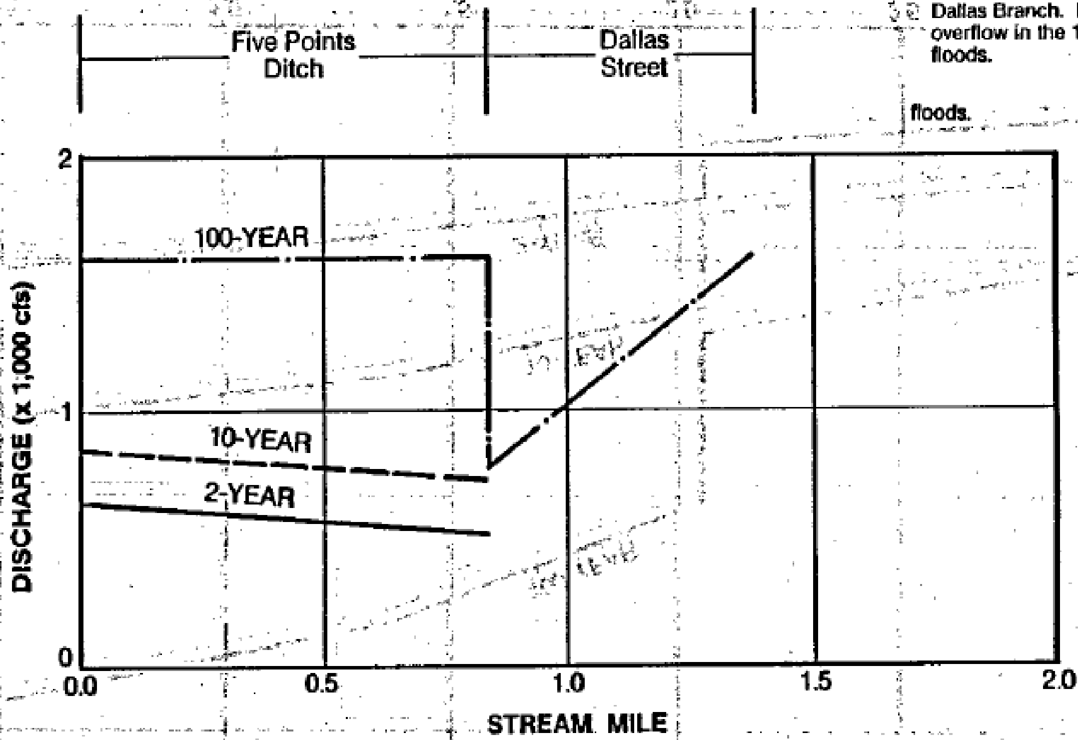
FIGURE 4-16
Flow Frequency Estimates for Normal Branch



Reference: TVA (1986).

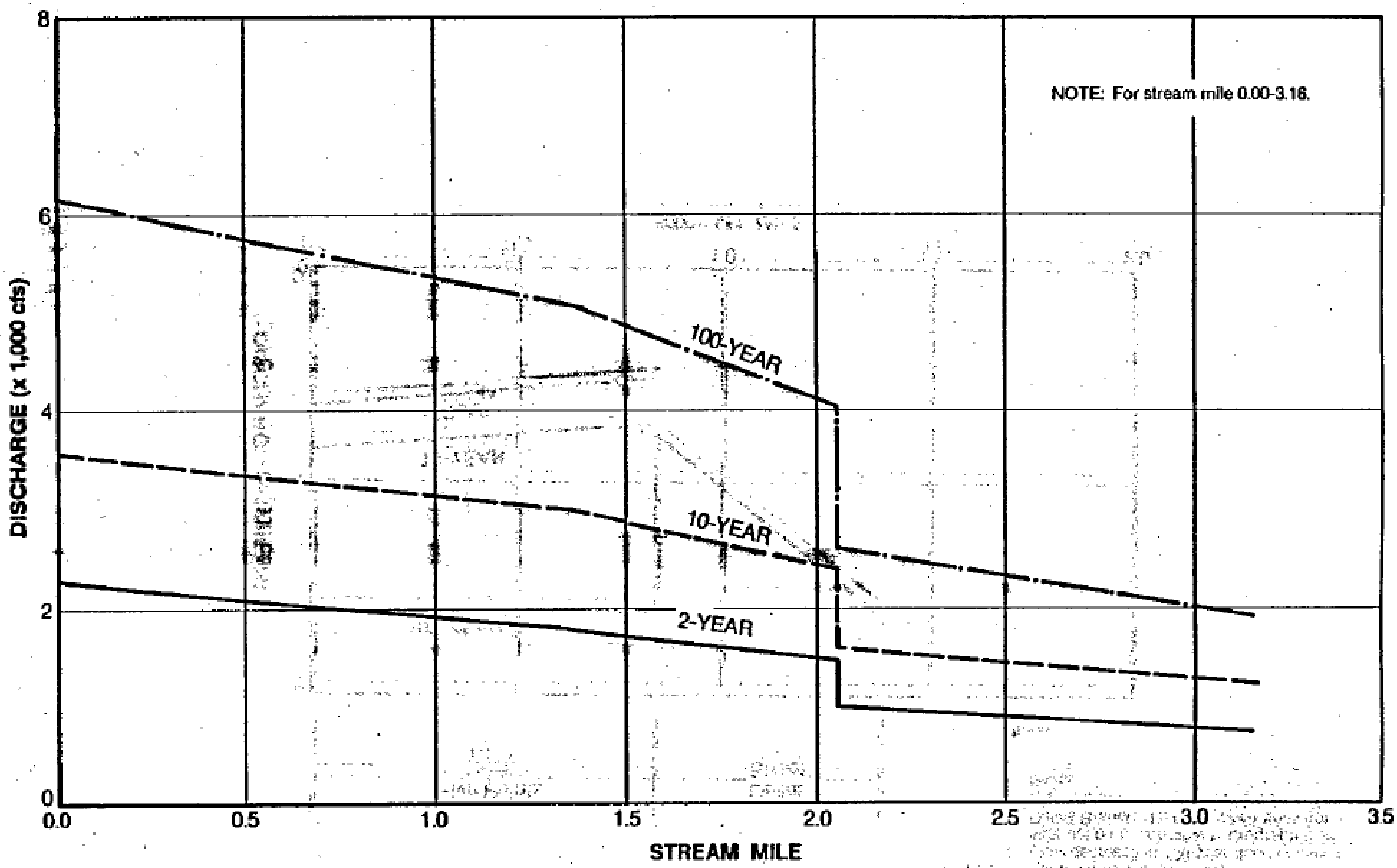
FIGURE 4-17
Flow Frequency Estimates for Dallas Branch

NOTES: 1. For stream mile 0.00-1.37.
2. Flow decrease in 100-year flood caused by attenuation of overflow hydrograph from Dallas Branch. Dallas Branch does not overflow in the 10-year and 2-year floods.



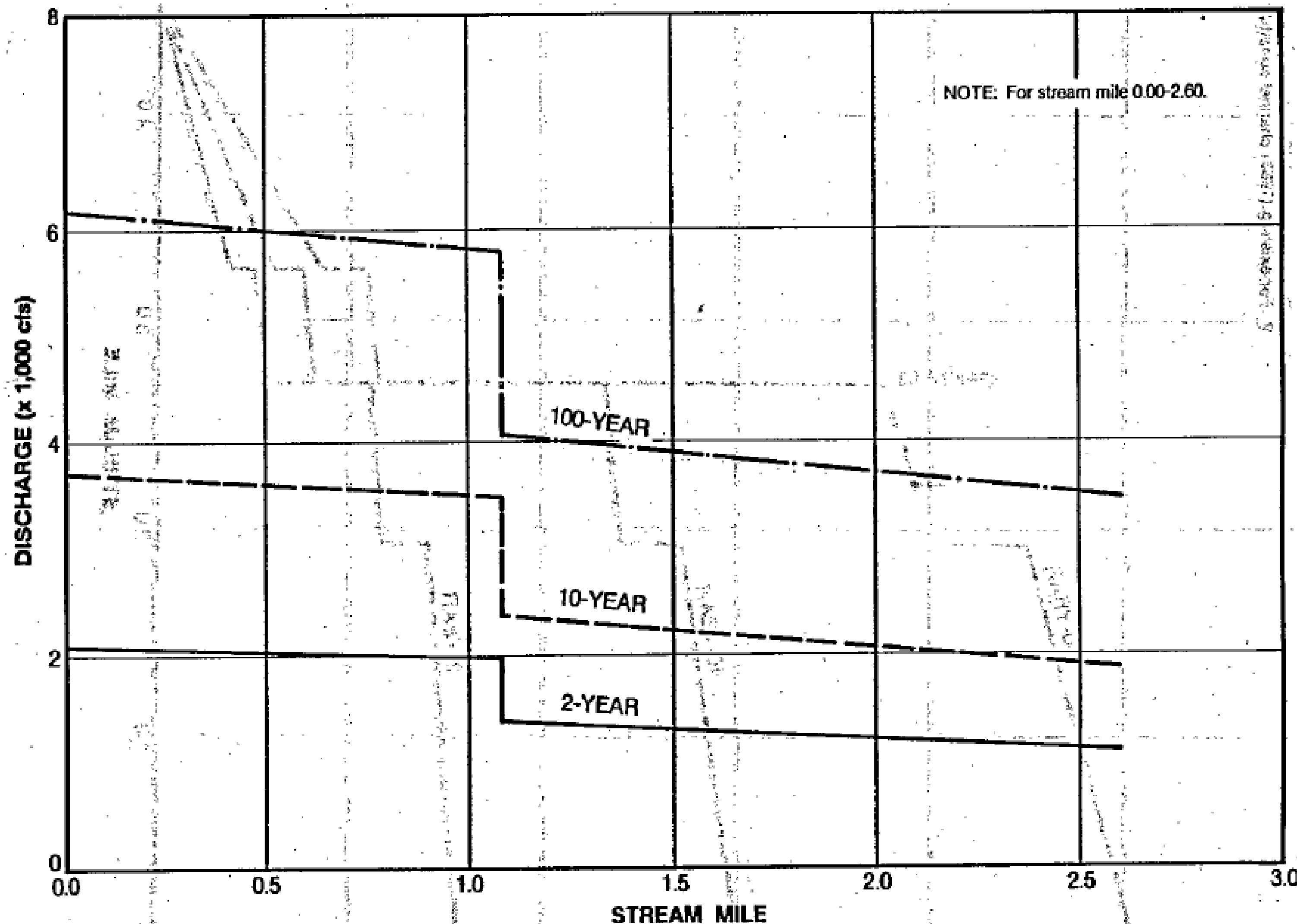
Reference: TVA (1986).

FIGURE 4-18
Flow Frequency Estimates for Dallas Branch Bypass



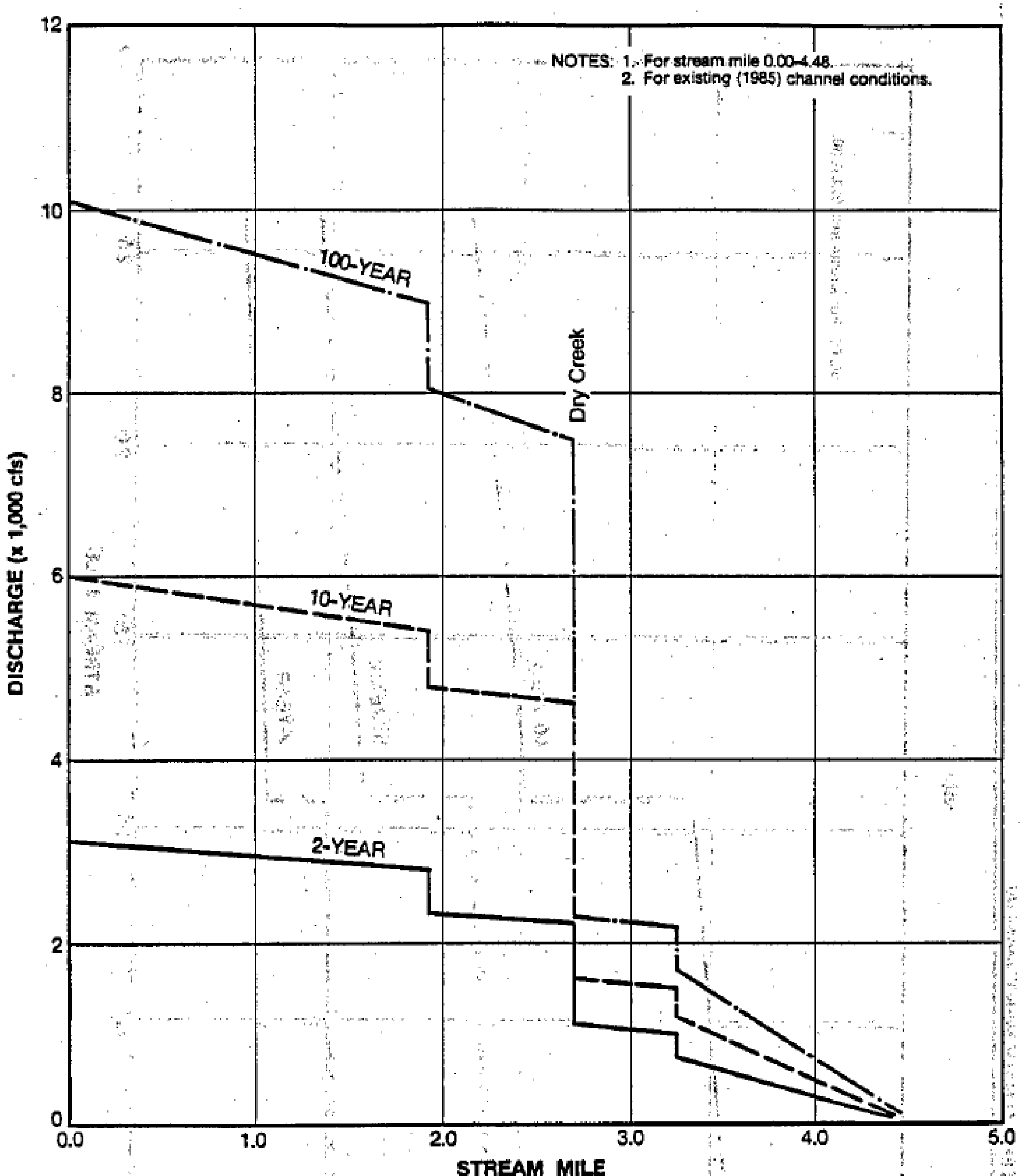
Reference: TVA (1986).

FIGURE 4-19
Flow Frequency Estimates for Fagan Creek



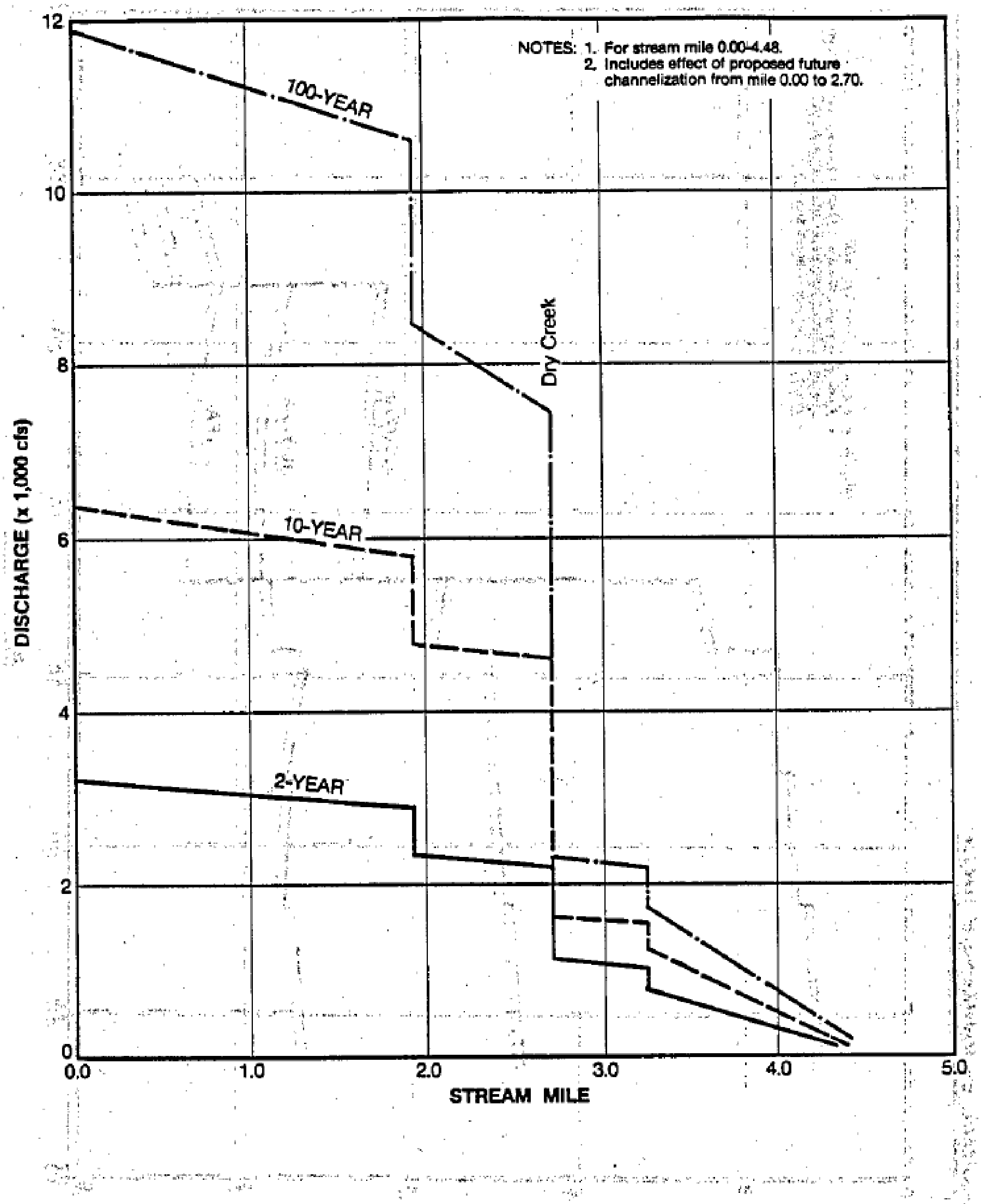
Reference: TVA (1986).

FIGURE 4-20
Flow Frequency Estimates for Dry Creek



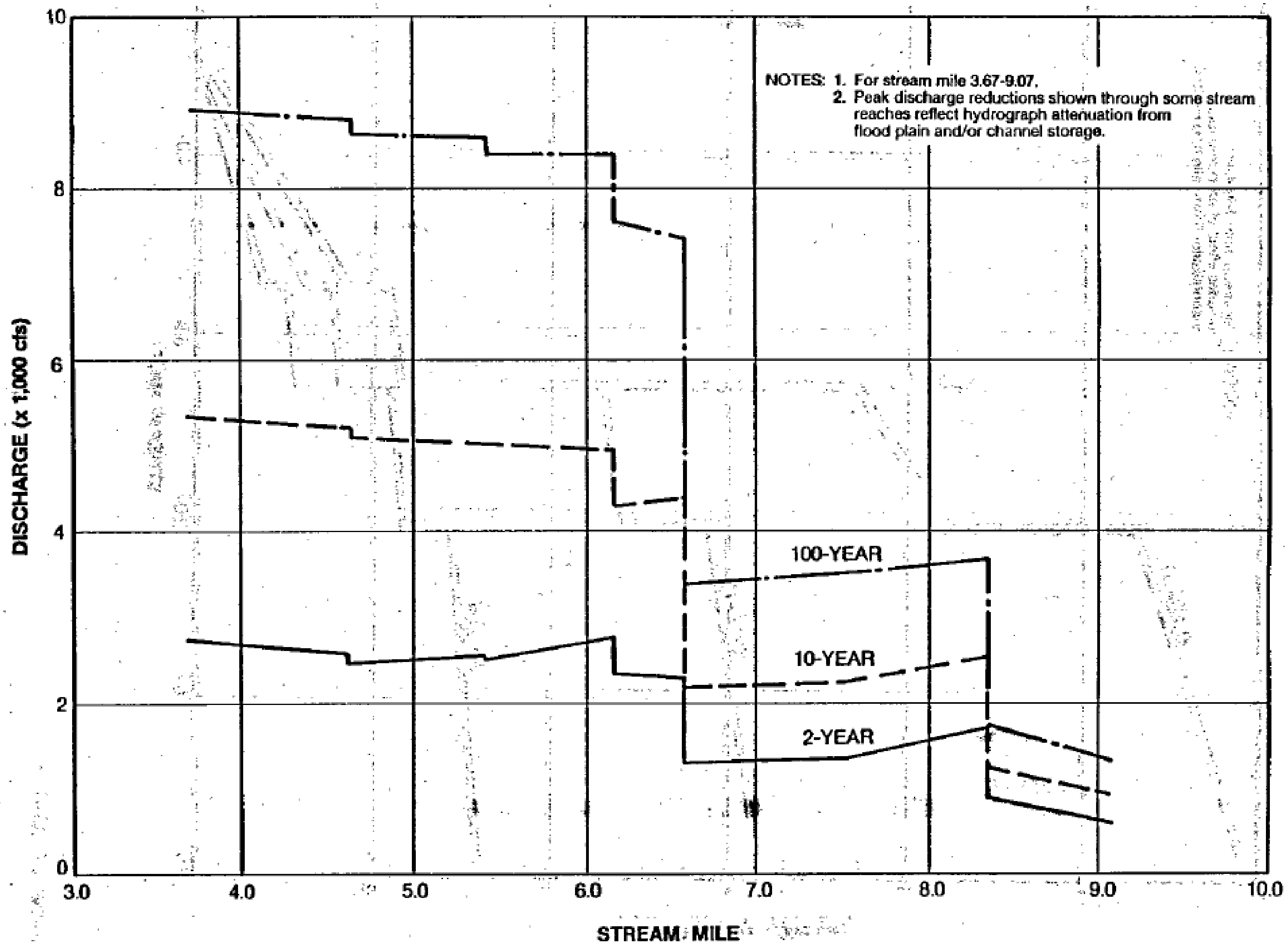
Reference: TVA (1986).

FIGURE 4-21
Flow Frequency Estimates for Broglan Branch, Existing Channel



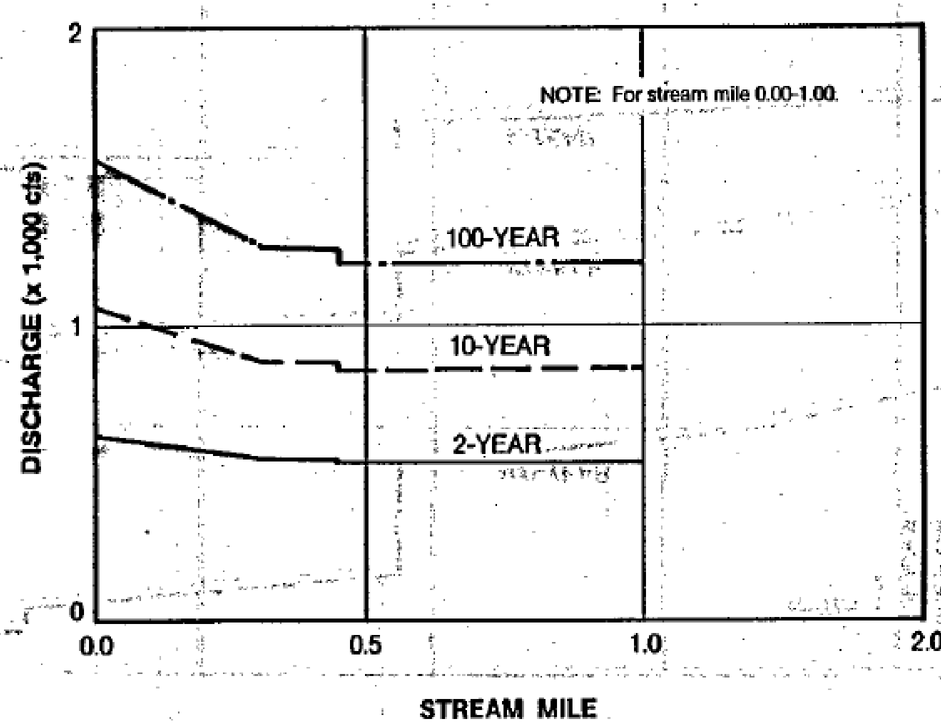
Reference: TVA (1986).

FIGURE 4-22
Flow Frequency Estimates for Broglan Branch, Proposed Channel



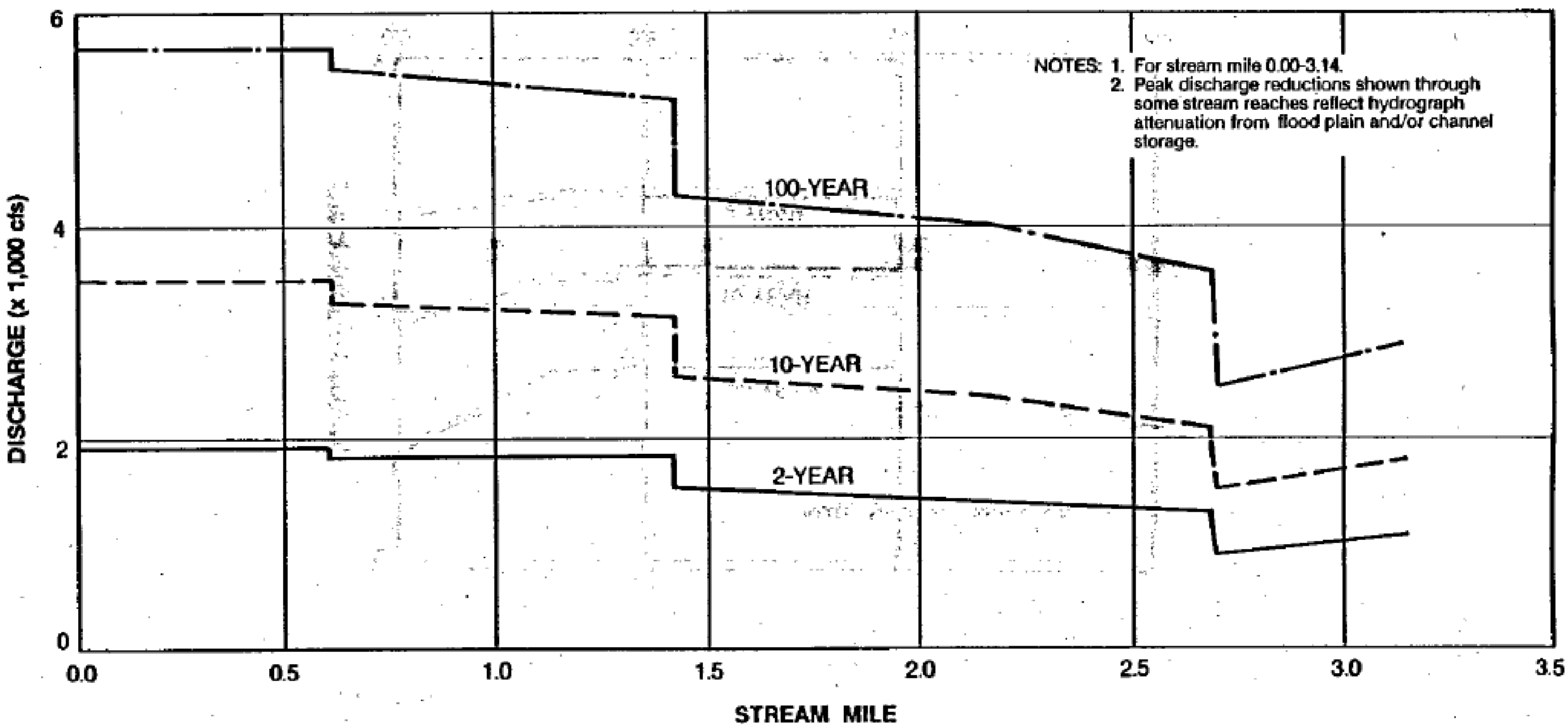
Reference: TVA (1986).

FIGURE 4-23
Flow Frequency Estimates for McDonald Creek



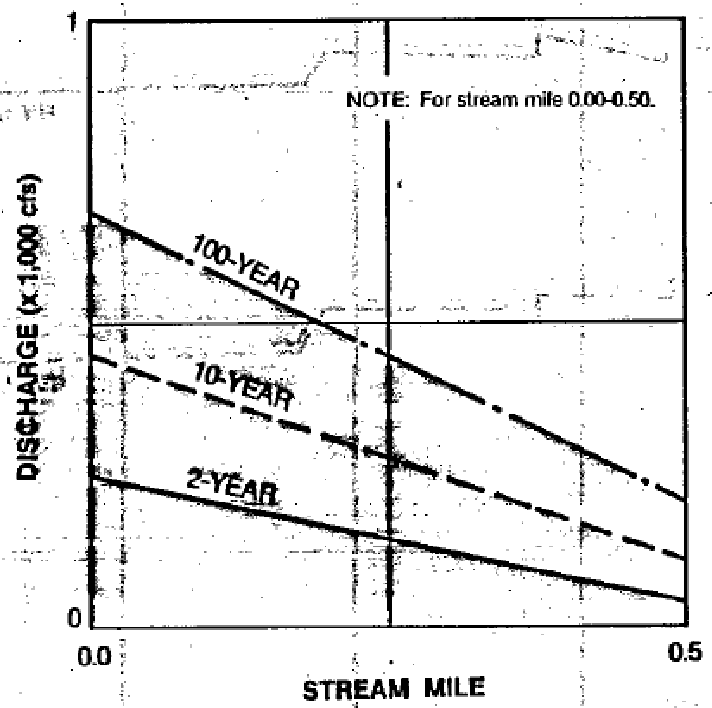
Reference: TVA (1986).

FIGURE 4-24
Flow Frequency Estimates for Tributary to McDonald Creek at Mile 6.14



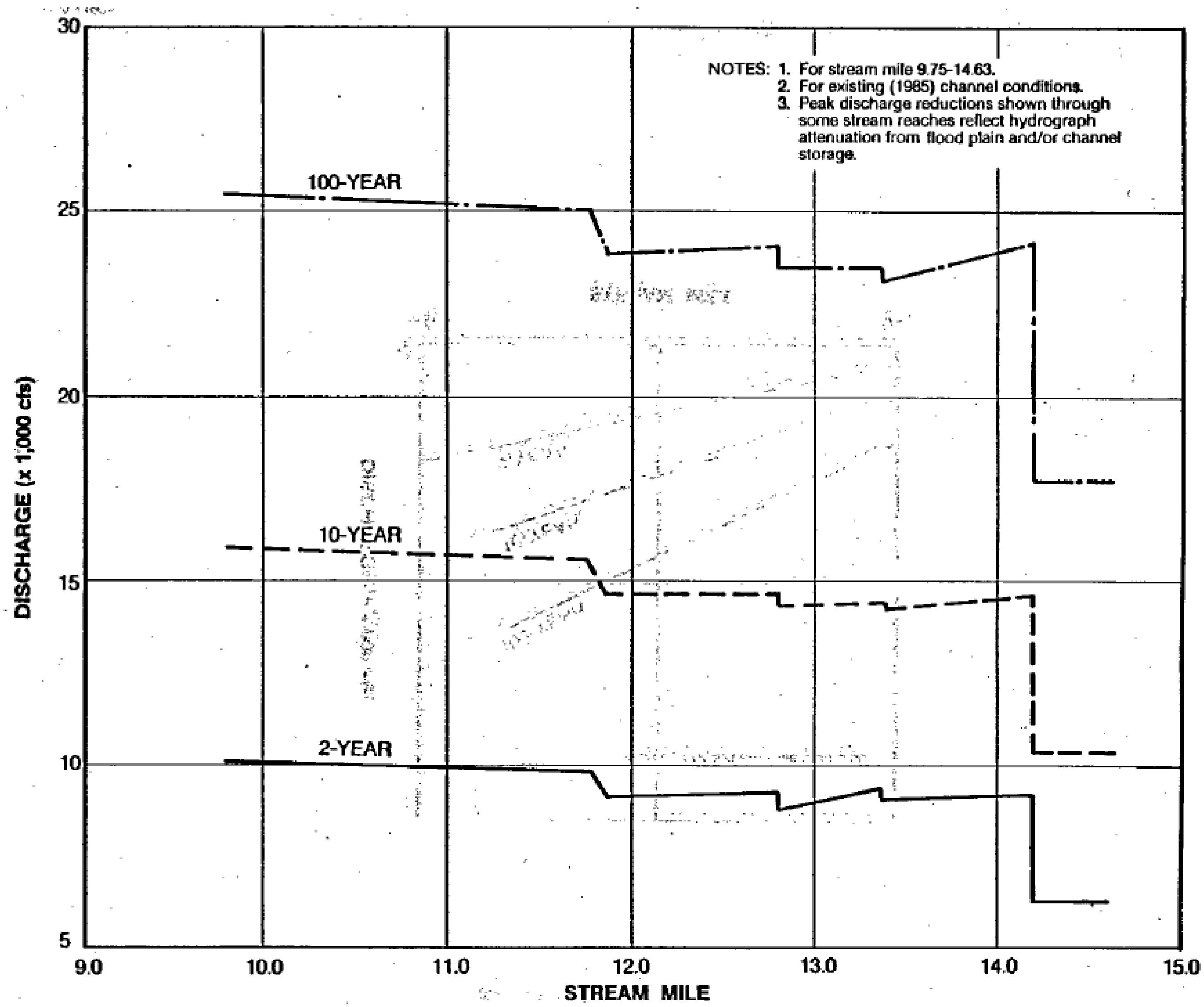
Reference: TVA (1986).

FIGURE 4-25
Flow Frequency Estimates for Sherwood Branch



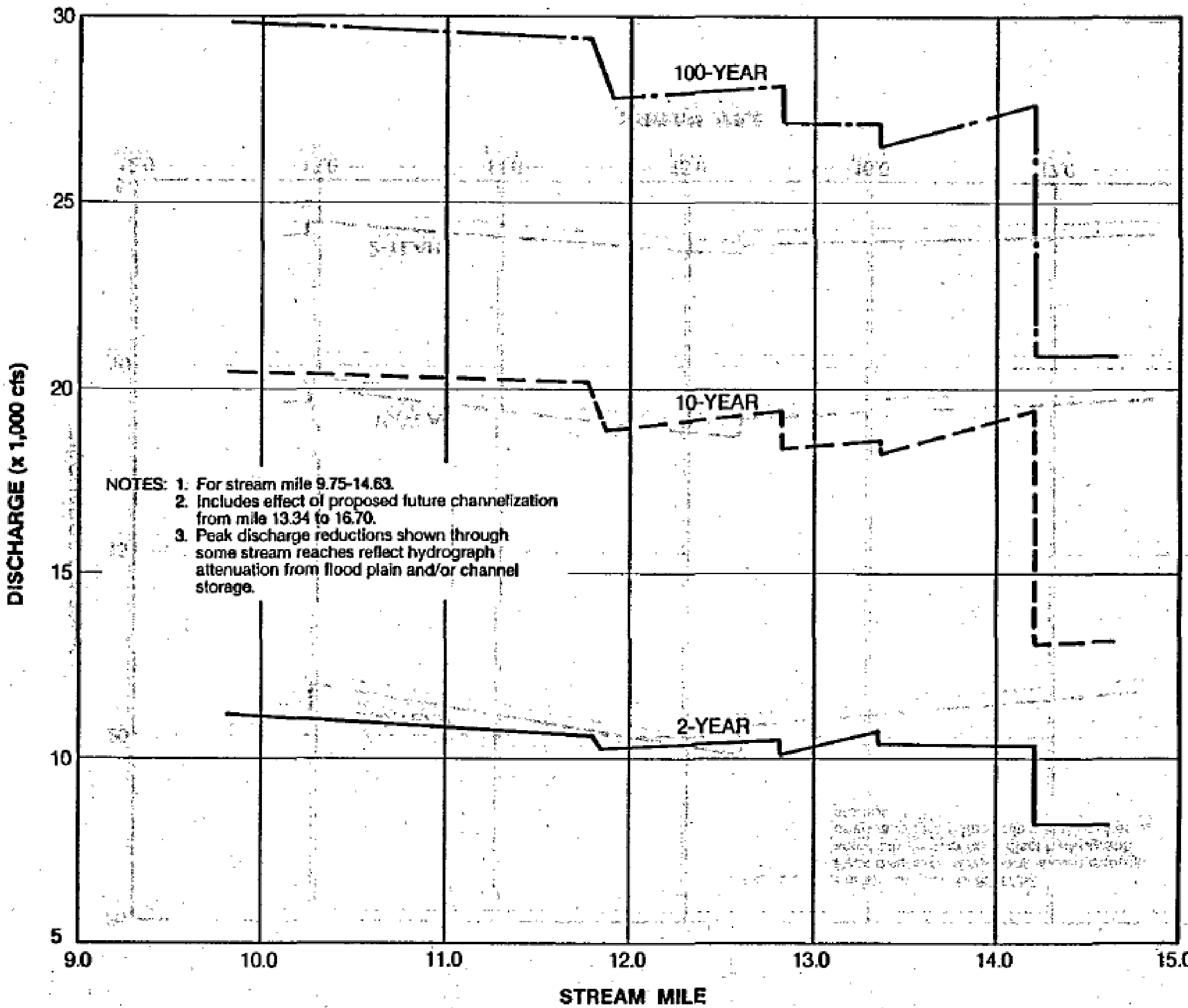
Reference: TVA (1986).

FIGURE 4-26
Flow Frequency Estimates for Tributary to Sherwood Branch at Mile 1.42



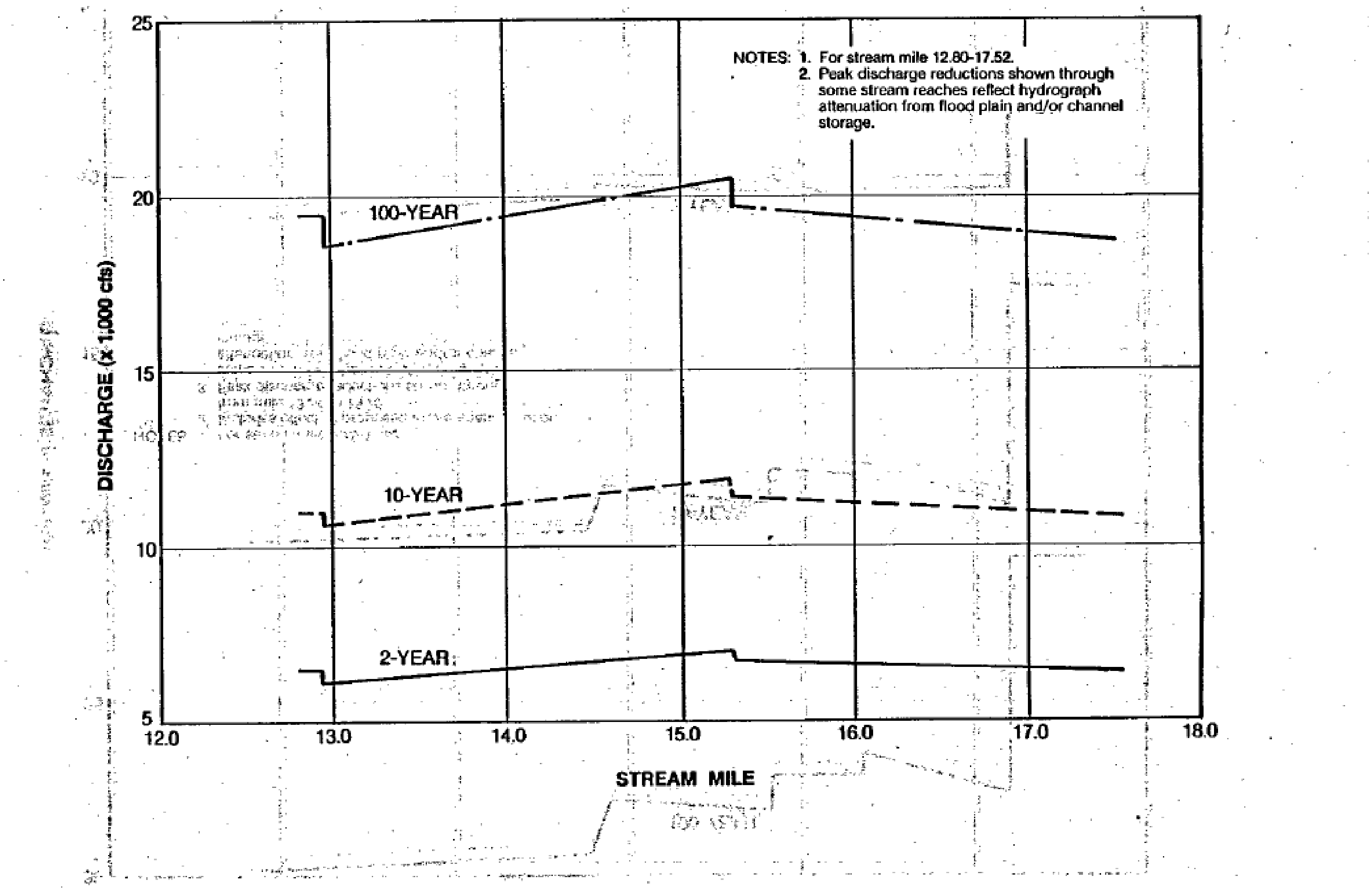
Reference: TVA (1986).

FIGURE 4-27
Flow Frequency Estimates for Huntsville Spring Branch, Existing Channel



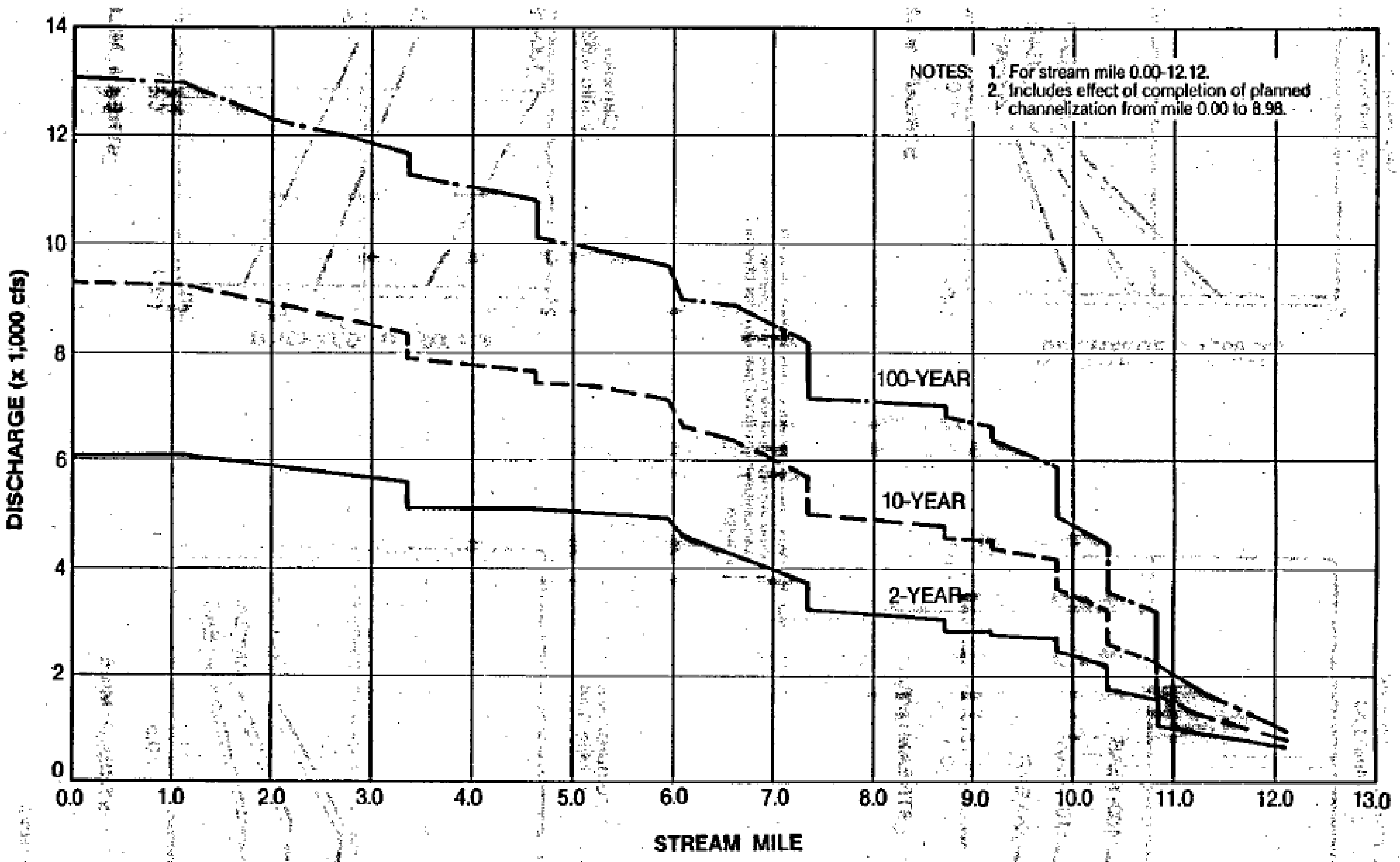
Reference: TVA (1986).

FIGURE 4-28
Flow Frequency Estimates for Huntsville Spring Branch, Proposed Channel



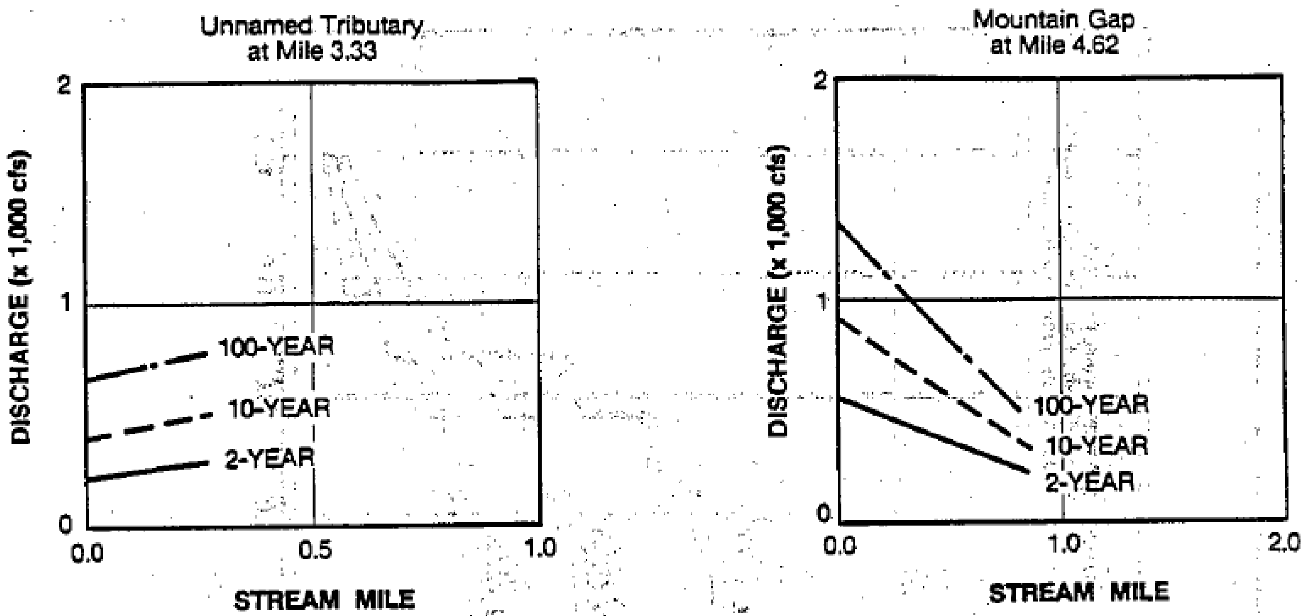
Reference: TVA (1986).

FIGURE 4-29
Flow Frequency Estimates for Indian Creek

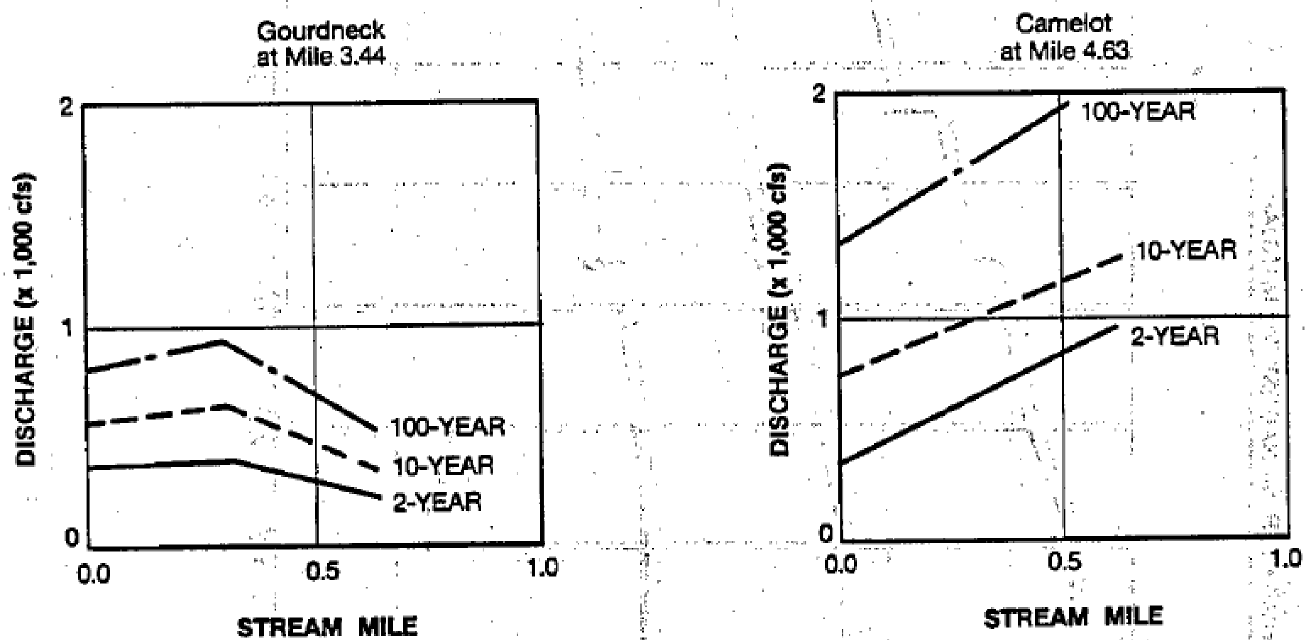


Reference: TVA (1986).

FIGURE 4-30
Flow Frequency Estimates for Aldridge Creek

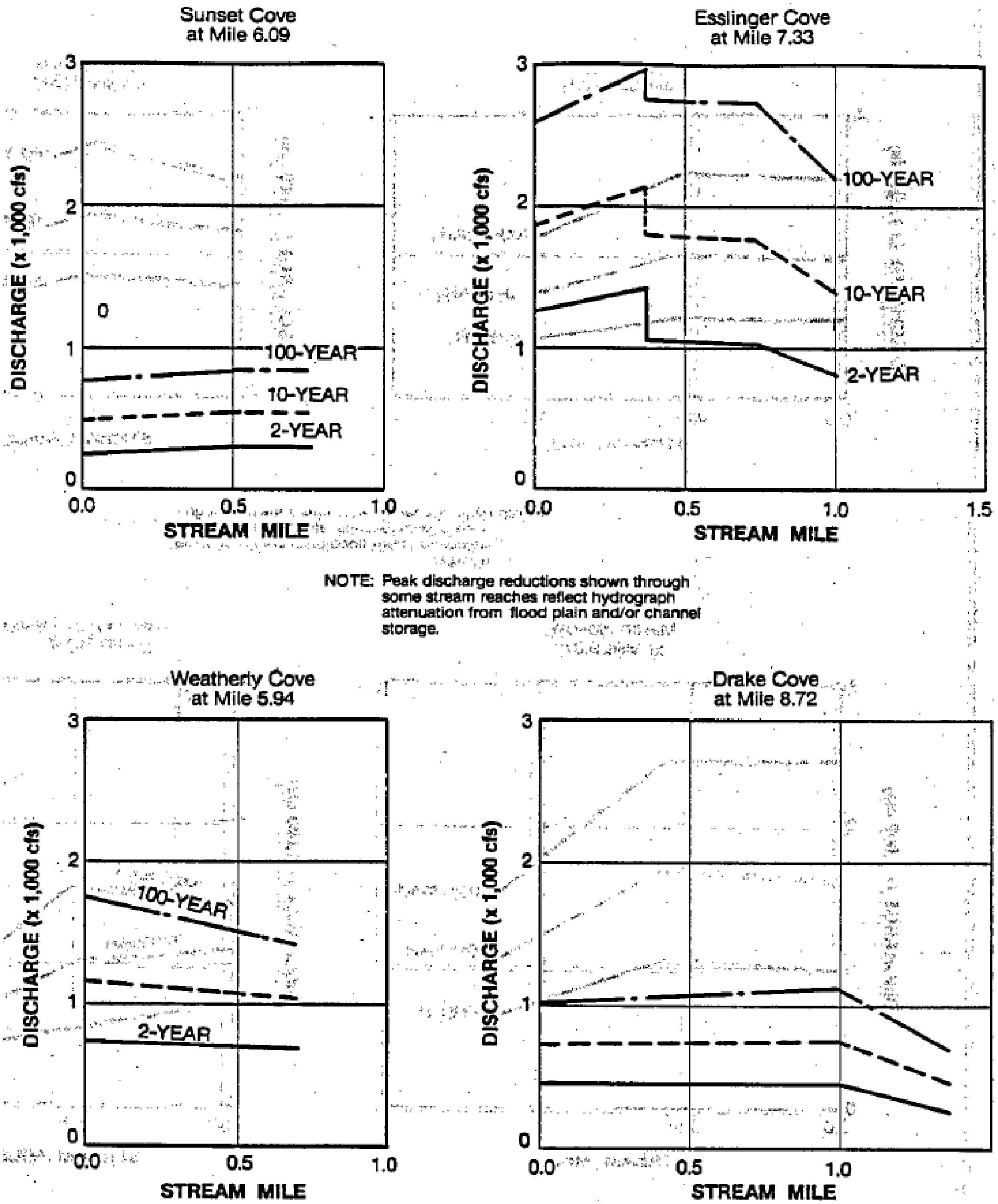


NOTE: Peak discharge reductions shown through some stream reaches reflect hydrograph attenuation from flood plain and/or channel storage.



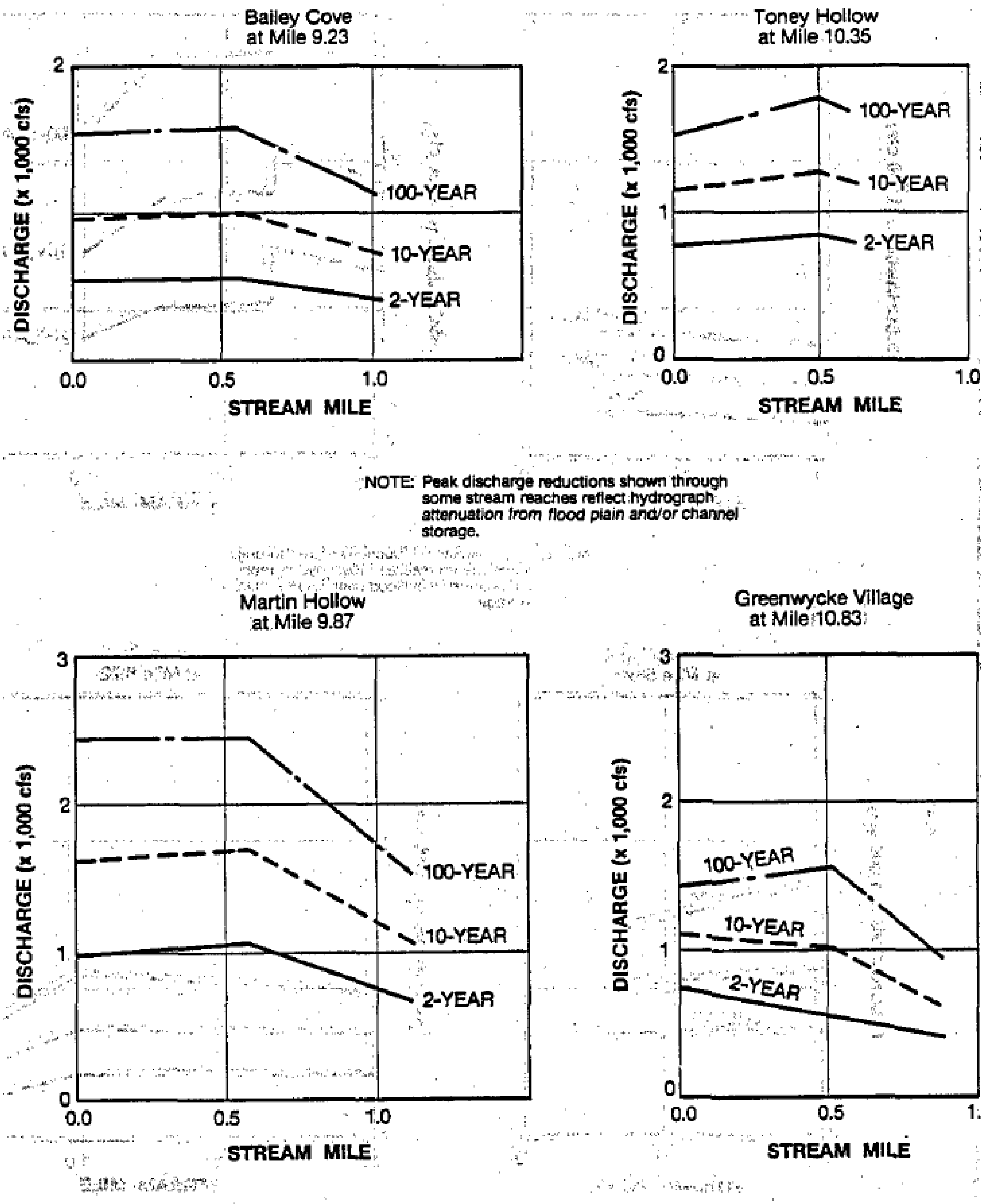
Reference: TVA (1986).

FIGURE 4-31
Flow Frequency Estimates for Aldridge Creek Tributaries



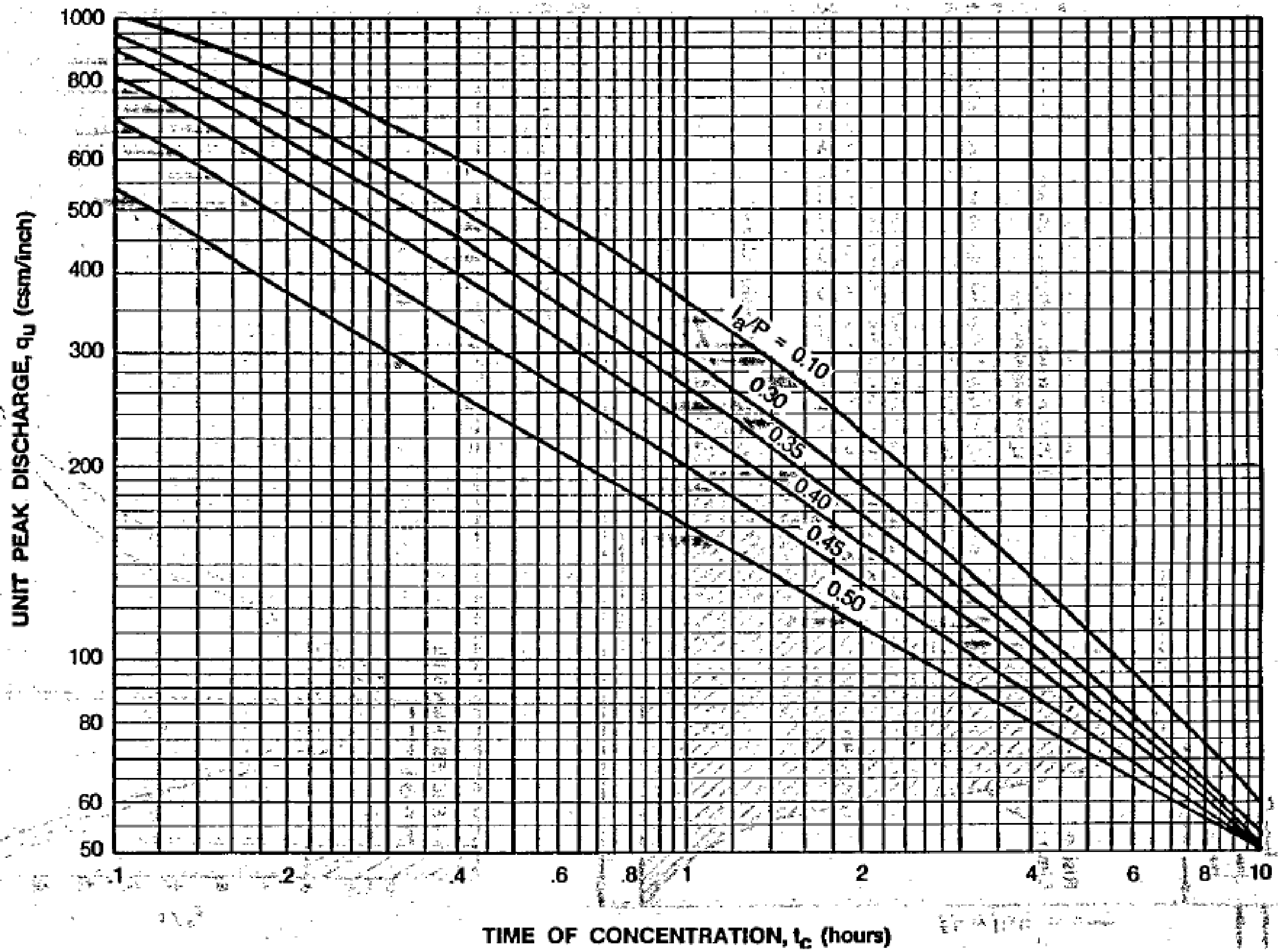
Reference: TVA (1986).

FIGURE 4-32
Flow Frequency Estimates for Aldridge Creek Tributaries



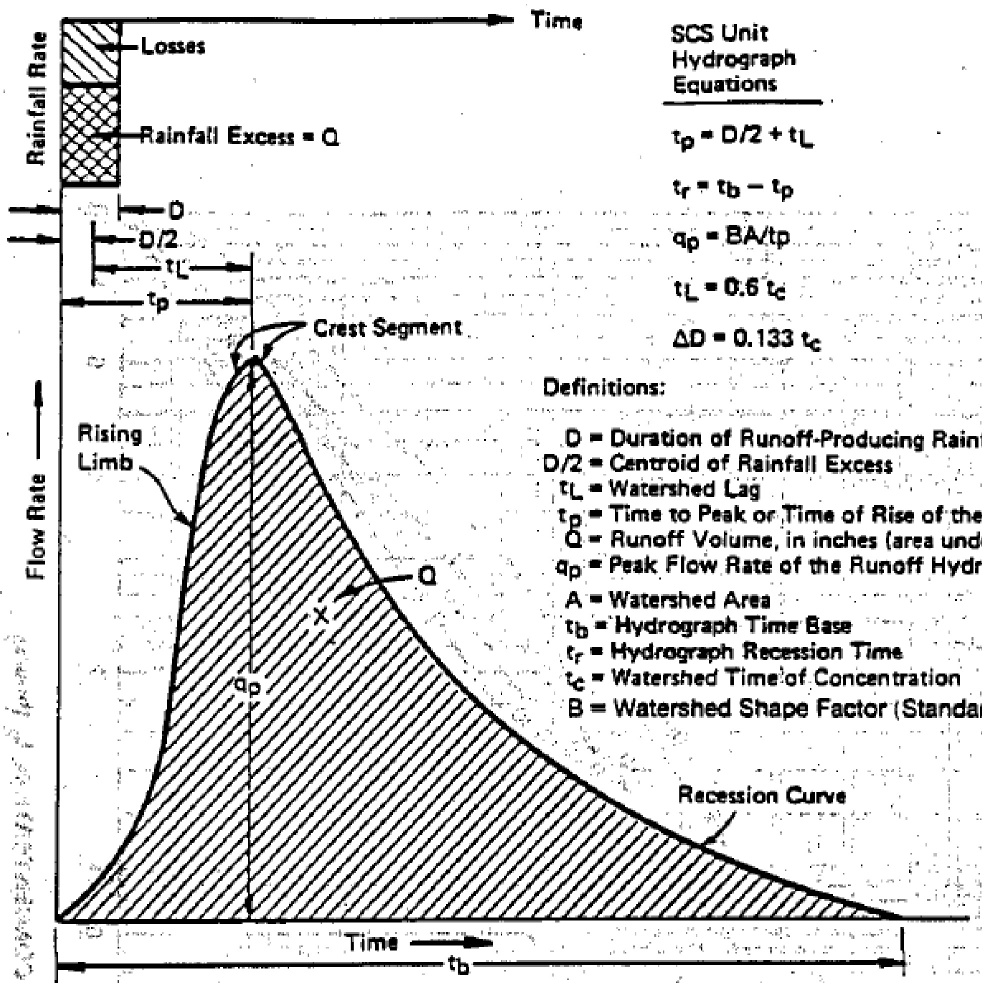
Reference: TVA (1986).

FIGURE 4-33
Flow Frequency Estimates for Aldridge Creek Tributaries



Reference: USDA, SCS, TR-55 (1986).

FIGURE 4-34
Unit Peak Discharge (q_u) for SCS Type II Rainfall Distribution



SCS Unit Hydrograph Equations

$t_p = D/2 + t_L$

$t_r = t_b - t_p$

$q_p = BA/tp$

$t_L = 0.6\ t_c$

$\Delta D = 0.133\ t_c$

Definitions:

- D = Duration of Runoff-Producing Rainfall in Hours
- $D/2$ = Centroid of Rainfall Excess
- t_L = Watershed Lag
- t_p = Time to Peak or Time of Rise of the Runoff Hydrograph
- Q = Runoff Volume, in inches (area under the curve)
- q_p = Peak Flow Rate of the Runoff Hydrrograph in cfs
- A = Watershed Area
- t_b = Hydrograph Time Base
- t_r = Hydrograph Recession Time
- t_c = Watershed Time of Concentration
- B = Watershed Shape Factor (Standard SCS $B = 484$)

FIGURE 4-35
General Hydrograph Terminology

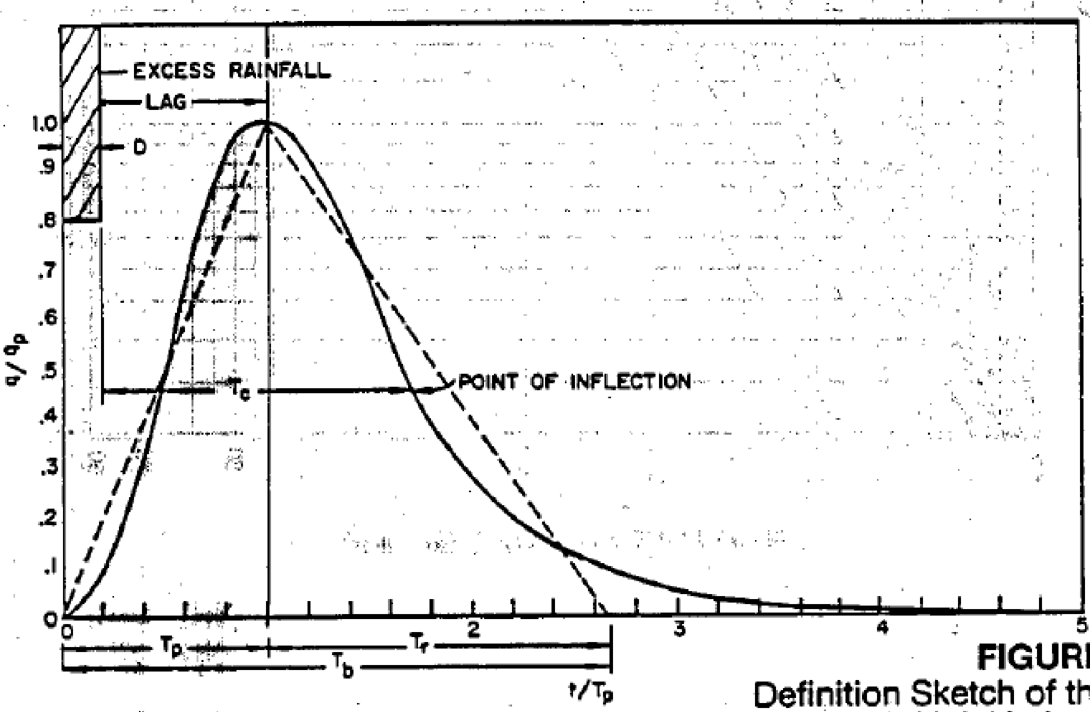
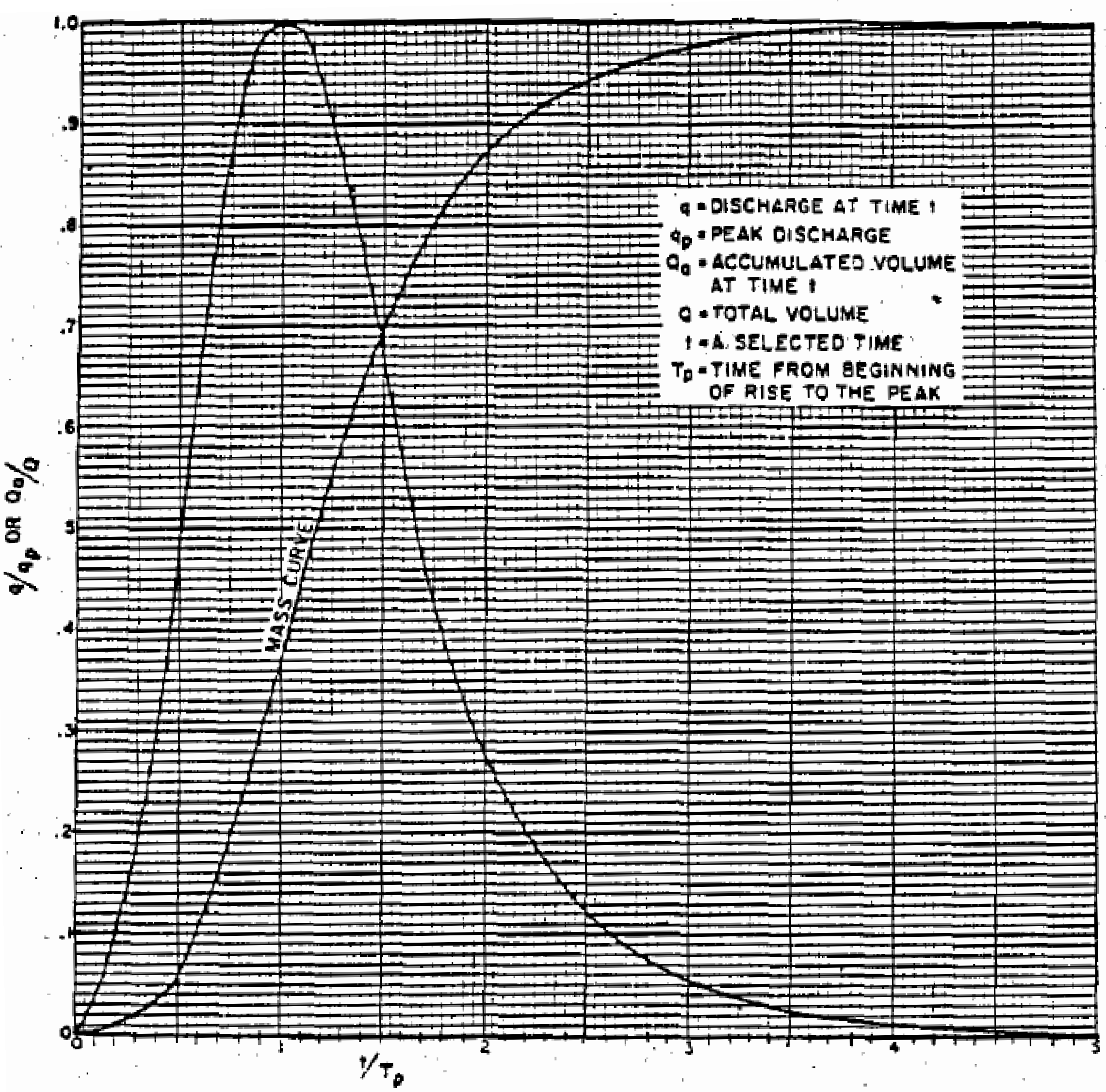


FIGURE 4-36
Definition Sketch of the SCS Dimensionless Unit Hydrographs

Reference: USDA, SCS, NEH-4 (1972).



Reference: USDA, SCS, NEH-4 (1972).

FIGURE 4-37
SCS Dimensionless Unit Hydrograph and Mass Curve

CHAPTER 5
OPEN CHANNEL HYDRAULICS

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OPEN CHANNEL HYDRAULICS

5.1 LININGS

Open channel linings are divided into the following four main classifications:

1. Vegetative (grass with mulch and sod)

2. Flexible (rock riprap and geotextile or interlocking concrete grids)

3. Rigid (concrete or grouted riprap)

4. Temporary (synthetic or natural materials to protect soil until permanent Stabilization is established)

5.1.1 <u>VEGETATIVE</u>

A healthy vegetative lining can stabilize the body of the channel, consolidate the soil mass of the bed, check erosion on the channel surface, and control the movement of soil particles along the channel bottom. Conditions under which a vegetative lining may not be acceptable, however, include but are not limited to:

1. Flow conditions in excess of the velocity limitations presented in Section 5.2.2

2. Standing or continuous flowing water

3. Lack of the regular maintenance necessary to prevent domination by taller topsoil

4. Lack of nutrients and inadequate topsoil

5. Excessive shade

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of a healthy growth of grass. Soil testing should be performed and the results evaluated by an agronomist to determine soil treatment requirements for parameters such as pH, nitrogen, phosphorus, potassium, and other factors (see Table 10-1). In many cases, temporary erosion control measures are required to provide time for the seeding to establish a viable vegetative lining (see Chapter 10). Sodding should be staggered, to avoid seams in the direction of flow.

5.1.2 <u>FLEXIBLE</u>

Flexible linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. Rock riprap including rubble is the most common type of flexible lining. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocities. The use of flexible lining may be restricted where right-of-way is limited, because the higher roughness values create layer cross sections.

5.1.3 RIGID

Rigid linings are generally constructed in places with limited right-of-way, where a smooth lining offers a higher capacity for a given cross-sectional area. Higher velocities, however, create a potential for scour at channel lining transitions. A rigid lining can be destroyed by flow undercutting the lining, channel headcutting, or the buildup of hydrostatic pressure behind the rigid surfaces. Filter fabric is required to prevent soil loss through pavement cracks.

5.1.4 TEMPORARY

Synthetic (e.g., geotextile) or natural (e.g., straw mulch) materials are required to protect exposed soil until permanent stabilization is established. The layout for temporary runoff control measures should be consistent with the layout of permanent facilities.

5.2 DESIGN CRITERIA

5.2.1 RETURN PERIOD

The design storm return period for open channel systems shall be 25 years. 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 transport in the channel.

5.2.2 VELOCITY LIMITATIONS

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for selected lining categories are presented in Table 5-1. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 5-2. Vegetative lining calculations are presented in Section 5.3.5 and riprap procedures are presented in Section 5.3.6.

5.2.3 MANNING'S n VALUES

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and riprap linings are presented in Table 5-3. Recommended values for vegetative linings should be determined using Figure 5-1, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 5-4). Figure 5-1 is used iteratively as described in Section 5.3.5.

For natural stream channels, Manning's n value should be estimated using Cowan's Equation (Cowan, 1956) as presented below:

$$n = (n_0+n_1+n_2+n_3+n_4)m_5 \qquad (5\text{-}1)$$

where:

n	=Manning's roughness coefficient for a natural or excavated channel
n_0	=Coefficient associated with channel lining material
n_1	=Coefficient associated with the degree of channel irregularity
n_2	=Coefficient associated with variations of the channel cross section
n_3	=Coefficient associated with the relative effect of channel obstructions
n_4	=Coefficient associated with channel vegetation
m_5	=Coefficient associated with the degree of channel meandering

Coefficients for Equation 5-1 can be determined using information in Table 5-5. Additional information is presented in FHWA-TS-84-204 (USDOT, FHWA, 1984), including procedures for determining Manning's values for floodplains.

5.2.4 SIDE SLOPES

Maximum open channel side slopes shall not be steeper than the following values, unless slope stability calculations, approve by the Director of City Engineering, indicate that steeper slopes are stable:

Lining Material	Maximum Side Slope
Concrete, Grouted Riprap	1:1
Ungrouted Riprap, Rock Lined, Solid Sod in Non-Residential Subdivision	2:1
Vegetative in Non-Residential Subdivision,	3:1
Vegetative in Residential Subdivision, Includes Solid Sod	4:1

5.2.5 RIPRAP LINING

Guidelines for specifying riprap gradation, thickness, and filter material are as follows:

1. Graduation: The size distribution curve should be smooth; the following range of particle sizes is acceptable:

$$\frac{D_{100}}{D_{50}} \text{ or } \frac{D_{50}}{D_{20}} \leq 3.0 \text{ and } \geq 1.5$$

where:

$\underline{D_{100}}$ = 100% of the material is smaller than this size

$\underline{D_{50}}$ = 50% of the material is smaller than this size, or mean particle size

$\underline{D_{20}}$ = 20% of the material is smaller than this size

2. Thickness: The thickness of riprap lining shall be greater than or equal to the diameter of the largest rock in the gradation, or 1.5 times the mean rock diameter, whichever is greater.

3. Filter Material: Filter material underlying rock riprap can be either granular filter layer or a geotextile fabric. Major concerns include soil retention, water permeability, and clogging. In general, the permeability of the filter material should be greater than the permeability of the native soil.

5.3 UNIFORM FLOW CALCULATIONS

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

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \quad (5\text{-}2)$$

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \quad (5\text{-}3)$$

$$S = \left(\frac{Qn}{1.49\ A\ R^{2/3}} \right)^2 \quad (5\text{-}4)$$

where:

V = Average channel velocity, in feet/second
Q = Discharge rate for design conditions, in cfs
n = Manning's roughness coefficient (see Section 5.2.3)
A = Cross-sectional area, in square feet
R = Hydraulic radius A/P, in feet
P = Wetted perimeter, in feet
S = Slope of the energy grade line, in feet/foot

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line and channel bottom can be assumed to be the same.

5.3.1 GEOMETRIC RELATIONSHIPS

Mathematical expressions are presented in Figure 5-2 for calculating the following parameters:

1. Area
2. Wetted perimeter
3. Hydraulic radius
4. Channel top width
5. Critical depth section factor

The selected cross sections include the following types:

1. Trapezoidal
2. Rectangular
3. Triangular
4. Parabolic
5. Circular

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 roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subjections is not counted as wetted perimeter (U.S. Geological Survey, 1976a,b).

5.3.2 DIRECT SOLUTIONS

When the hydraulic radius, cross-sectional area, slope and roughness coefficient of an open channel are known, discharge can be calculated directly from Equation 5-3. Design aids for the direct solution of Manning's Equation are presented in the form pf tables, charts, and nomographs.

A constant parameter, which depends on only the geometry and channel roughness, can be used to simplify the direct solution of Manning's Equation as follows:

$$Q = C_1 S^{1/2} \tag{5-5}$$

$$S = (Q/C_1)^2 \tag{5-6}$$

where:

Q = Discharge for design conditions, in cfs

$$C_1 = \frac{1.49}{n} AR^{2/3} \tag{5-7}$$

A = Cross-sectional area, in square feet
R = Hydraulic radius, A/P, in feet
P = Wetted perimeter, in feet
S = Slope of the energy grade line, in feet/foot

Tables: Tables of C_1 Values are provided for concrete pipe with the following cross sections:

Circular	Table 5-6
Elliptical	Table 5-7
Arch	Table 5-8
Precast Box	Table 5-9

Using the C_1 value obtained from the appropriate table, the capacity or slope can be obtained directly by solving Equation 5-5 or 5-6.

Charts: Full flow capacity charts for the following pipe shapes and roughness coefficients are duplicated from the American Concrete Pipe Association (1980) as the figures listed:

Arch, n = 0.011, Figure 5-3
Arch, n = 0.012, Figure 5-4
Arch, n = 0.013, Figure 5-5
Box, n = 0.012, sizes 3'x2' to 8'x8', Figure 5-6
Box, n = 0.012, sizes 9'x5' to 12'x12', Figure 5-7
Box, n = 0.013, sizes 3'x2' to 8'x8', Figure 5-8
Box, n = 0.013, sizes 9'x5' to 12' x 12', Figure 5-9

These charts provide a direct graphical solution for the capacity solution to Manning's Equation (Equation 5-3) when all but one variable in the equation are known.

Nomographs: Direct solutions to Manning's Equation can be obtained using the nomographs presented in Figures 5-10 and 5-11. Part A of Figure 5-10 provides a general solution for the velocity form of Manning's Equation (Equation 5-2), while Part B provides capacity and velocity solutions for circular pipe. Figure 5-11 provides a nomograph solution for trapezoidal channels.

General: The following steps are used for the general solution nomograph in Part A of Figure 5-10:

1. Determine open channel data, including slope in feet/foot, hydraulic radius in feet, and Manning's n value.

2. Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

3. Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

4. Extend the line from Step 3 to the velocity scale to obtain the velocity in feet/second.

Circular. The following steps are used for the circular pipe nomograph in Part B of Figure 5-10:

1. Determine input data, including slope in feet/foot, Manning's n value, and pipe diameter in inches or feet.

2. Connect a line from the slope scale, Point 1, to the Manning's n scale, Point 2, and note the point of intersection on the turning line, Point 3.

3. Connect a line from the pipe diameter, Point 5, to the point of intersection obtained in Step 2, Point 3.

4. Extend the line from Step 3 to the discharge and velocity scaled to read the discharge at Point 4 and the velocity at Point 6.

Trapezoidal. The trapezoidal channel nomograph solution to Manning's Equation in Figure 5-11 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

1. Determine input data, including the slope in feet/foot, Manning's n value, bottom width in feet, and side slope in feet/foot.

2. a. Given the design discharge, find the product of Q times n, connect a line from slope scale to the Qn scale, and find the point of intersection on the turning line.

   b. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.

   c. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.

   d. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.

3. a. Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z, = 0 scale.

   b. Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.

   c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Qn scale.

   d. Divide the value of Qn obtained in Step 3c by the n value to ding the design discharge, Q.

5.3.3 TRIAL AND ERROR SOLUTIONS

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

$$AR^{2/3} = \frac{Qn}{1.49\ S^{1/2}} \tag{5-8}$$

where:

A = Cross-sectional area, in square feet

R = Hydraulic radius (A/P), in feet

P = Wetted perimeter, in feet

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 5.2.3)

S = Slope of the energy grade line, in feet/foot

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 5-8 is satisfied such that the design flow is conveyed within the banks of the desired channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figures 5-12 through 5-15, which are described below.

Trapezoidal Channels

For trapezoidal channels, Figure 5-12 can be used to make capacity calculations as follows:

1. Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.

2. Calculate the trapezoidal conveyance factor using the equation:

$$K_T = \frac{Qn}{b^{8/3}\ S^{1/2}} \tag{5-9}$$

where:

K_T = Trapezoidal open channel conveyance factor

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 5.2.3)

b = Bottom width, in feet

S = Slope of the energy grade line, in feet/foot

3. Enter the x-axis of Figure 5-12 with the value of K_T calculated in Step 2 and draw a line vertically to the curve that corresponds to the appropriate z 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, d/b.

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

<u>Circular Pipe</u>

For partial flow in a circular pipe, Figure 5-13 can be used for capacity calculations as follows:

1. Determine input data, including design discharge, Q, Manning's n value, pipe diameter, D, and channel slope, S.

2. Calculate the circular pipe conveyance factor using the equation.

$$K_p = \frac{Qn}{D^{8/3} S^{1/2}} \tag{5-10}$$

where:

K_p = Circular pipe open channel conveyance factor

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient

D = Pipe diameter, in feet

S = Slope of the energy grade line, in feet/foot

3. Enter the x-axis of Figure 5-13 with the value of Kp calculated in Step 2 and draw a line vertically to the curve.

4. From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read a value of the normal depth of flow over the pipe diameter, d/D.

5. Multiply the d/D value from Step 4 by the pipe diameter, D, to obtain the normal depth of flow.

An alternative procedure for evaluating partial flow at any depth in circular pipe is to adjust the full pipe capacity using Figure 5-14.

Pipe Arch

The capacity of pipe arch at any depth can be estimated using Figure 5-15 to adjust the full pipe capacity. Full pipe capacity for pipe arch can be estimated using date in Table 5-8 or capacity charts in Figures 5-3, 5-4, 5-5.

5.3.4 CRITICAL FLOW CALCULATIONS

A design flow at or near critical depth ($\pm$10 percent) should be avoided, because such flow conditions are not stable. The general equation for determining critical depth is expressed as:

$$\frac{Q^2}{g} = \frac{A^3}{T} \tag{5-11}$$

where:

Q = Discharge rate for design conditions, in cfs

g = Acceleration due to gravity, 32.2 feet/second^2

a = Cross-sectional area, in square feet

T = Top width of water surface, in feet

It is important to note that critical depth depends only on the discharge rate and channel geometry. A trial and error procedure is required to solve Equations 5-11. Equations (as presented in Table 5-10) or section factors (as presented in Figure 5-2) can be used to simplify trial and error critical depth calculations, The following equation from Chow (1959) is used to determine critical depth using an appropriate critical flow section factor, Z:

$$Z = Q / \sqrt{g} \tag{5-12}$$

where:

Z = Critical flow section factor (see Table 5-10)

Q = Discharge rate for design conditions, in cfs

G = Acceleration due to gravity, 32.2 feet/second^2

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

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

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

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

4. A normal depth of uniform flow within about 10 percent of critical depth is unstable and

should be avoided in design, if possible.

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

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

$$Fr = v / (gA/T)^{0.5}$$

where:

Fr = Froude number, dimensionless

v = velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

A = Cross-sectional area of flow, in square feet

T = Top width of flow, in feet

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

5.3.5 VEGETATIVE DESIGN

A two-part procedure, adapted from Chow (1959) and presented below is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 5-4. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 5-4. If temporary lining is to be used during construction, vegetative retardance Class E should be used for design stability calculations.

Temple et al. (1987) present an alternative procedure for designing grass-lined channels that is acceptable but not duplicated in the manual.

If the channel slope exceeds 10 percent, or a combination of channel linings wil1 be used, additional procedures not presented below are required. References include HEC-15 (USDOT; FHWA; 1986) and HEC-14 (USDOT; FHWA; 1983).

Design Stability

1. Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

2. Use Table 5-2 to assign a maximum velocity, V_m, based on vegetation type and slope range.

3. Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 5-1. Use retardance Class D for permanent vegetation and E for temporary construction.

4. Calculate the hydraulic radius using the equation:

$$R = \frac{(VR)}{V_m} \tag{5-14}$$

where:

R = Hydraulic radius of flow, in feet

vR = Value obtained from Figure 5-1 in Step 2

v_m = Maximum velocity from Step 2

5. Use the following form for Manning's Equation to calculate the value of vR:

$$vR = \frac{1.49\ R^{5/3}\ S^{1/2}}{n} \tag{5-15}$$

where:

vR = Calculated value of vR product

R = Hydraulic radius value from Step 4, in feet

S = Channel bottom slope, in feet/foot

n = Manning's n value assumed in Step 3

6. Compare the vR product value obtained in Step 5 to the value obtained from Figure 5-1 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

7. Use the final Manning's n value from Step 6 to select channel dimensions that give a hydraulic radius close to the final value from Step 6. For trapezoidal channels, the flow depth can be estimated using Figures 5-11 or 5-12. The depth of flow for other channel shapes can be evaluated using the trial and error procedures in Section 5.3.3.

8. If bends are considered, calculate the length of downstream protection, L_p, for the bend using Figure 5-16. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

Design Capacity

1. Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 5-2 for equations).

2. Divide the design flow rate, obtained using appropriate procedures from Chapter 2, by the waterway area from Step 1 to find the velocity.

3. Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

4. Use Figure 5-1 to find the Manning's n value for retardance Class C based on the vR value from Step 3.

5. Use Manning's Equation (Equations 5-2) or Figure 5-10 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

6. Compare the velocity values from Steps 2 and 5. If the values are not reasonably close return to Step 1 and repeat the calculations.

7. Add an appropriate freeboard to the final depth from Step 6. Generally, 20 percent is adequate.

8. If bends are considered, calculate superelevation of the water surface profile at the bend using the equation:

$$\Delta d = \frac{v^2 T}{g R_c} \tag{5-16}$$

where:

Δd = Superelevation of the water surface profile due to the bend, in feet.

v = Average velocity from Step 6, in feet/second

T = Top width of flow, in feet

G = Acceleration due to gravity, 32.2 feet/second^2

R_c = Mean radius of the bend, in feet

Add freeboard consistent with the calculated Δd.

5.3.6 RIPRAP DESIGN

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

1. Minimum riprap thickness equal to d_{100}.
2. The value of d_{85}/d_{15} less than 4.6.
3. Froude number less than 1.2.
4. Side slopes up to 2:1.
5. A safety factor of 1.2.
6. Maximum velocity less than 18 feet per second.

If significant turbulence is caused by boundary irregularities, such as installations near obstructions or structures, this procedure is not applicable.

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 5.3.3). Manning's n values for riprap can be calculated from the equation:

$$n = 0.0395\,(d_{50})^{1/6} \qquad (5\text{-}17)$$

where:

n = Manning; roughness coefficient for stone riprap

d_{50} = Diameter of stone for which 30 percent, by weight, of the gradation is finer, in feet.

2. If the rock is to be placed at the outside of a bend, multiply the velocity

determined in Step 1 by the bend correction coefficient, C_b, given in Figure 5-17 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, R_b.

3. If the specific weight of the stone varies from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 5-18.

4. Determine the required minimum d_{30} value from Figure 5-19, which is based on the equation:

$$d_{30}/D = 0.193\ Fr^{2.5} \qquad (5\text{-}18)$$

where:

d_{30} = Diameter of stone for which 30 percent, by weight, of the gradation is finer, in feet.

D = Depth of flow above stone, in feet

Fr = Froude number (see Equation 5-13), dimensionless

v = Mean velocity above the stone in feet/second

g = Acceleration of gravity, 32.2 feet/second^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 time the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$W = 0.5236\ \Upsilon_s\ d^3 \qquad (5\text{-}19)$$

where:

W = Stone weight, in pounds

D = Selected stone diameter, in feet

Υ_S = Specific weight of stone, in pounds/cubic foot

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 included 50 percent for underwater placement.

6. If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 5-20 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.
7. Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

5.3.7 EXAMPLE PROBLEMS

Example 5-1. Direct Solution of Manning's Equation

Use Manning's Equation to find the full flow capacity, Q, of a 48-inch inside diameter, circular concrete pipe storm sewer with an n value of 0.012 and slope of 0.006 feet/foot.

1. Solve using Part B of Figure 5-10.

   d. Connect a line between the slope scale at 0.006 and the roughness scale at 0.012 and note the intersection point on the turning line.

   e. Connect a line between that intersection point and the pipe diameter scale at 48 inches and read the pipe capacity of 120 cfs from the discharge scale.

   f. Proceed to the velocity scale and read a value of 9.6 feet/second.

2. Solve using Table 5-6.

   a. Look under the column n = 0.012 for a 48-inch diameter pipe and find C_1 = 1556.

   b. For S = 0.006, calculate $S^{½}$ = 0.07746.

   c. For Q, use Equation 5-5 to find (1556) (0.07746) = 121 cfs.

d. Use Table 5-6 to find A=12.566 square feet. Since Q = Av, v = 121/12.566 = 9.6 feet/second.

Example 5-2. Grassed Channel Design Stability

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 feet/foot. Find the channel dimensions required to comply with design stability criteria (retardance Class D) for a grass mixture.

1. From Table 5-2, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5 percent is 4 feet/second.

2. Assume an n value of 0.035 and find the value of vR from Figure 5-1.
   vR = 5.4

3. Use equation 5-14 to calculate the value of R.
   $$R = \frac{5.4}{4} = 1.35 \text{ feet}$$

4. Use Equation 5-15 to calculate the value of vR.
   $$vR = \frac{1.49\,(1.35)^{5/3}\,(0.015)^{½}}{(0.035)} = 8.60$$

5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed N value	vR (Figure 5-1)	R (Equation 5-14)	vR (Equation 5-12)
0.035	5.4	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.57
0.040	3.2	0.80	3.15

   Select n = 0.040 for stability criteria.

6. Use Figure 5-11 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

$Qn = (50)\ (0.040) = 2.0$

$S = 0.015$

Try b = 8 feet, d = (8) (0.14) = 1.12 feet

Since R = 0.844 feet and a value of 0.80 feet is required, b must be increased.

Try b = 10 feet, d = (10) (0.098) = 0.98 feet

Since R = 0.796 feet, channel geometry is adequate.

Select b = 10 feet

Z = 3

D = 1 foot

v = 3.9 feet/second (Equation 5-2)

Fr = 0.76 (Equation 5-13)

Flow is subcritical

Design capacity calculations for this channel are presented in Example 5-3.

Example 5-3. Grassed Channel Design Capacity

Use a 10-foot bottom width for the trapezoidal channel sized in Example 5-2 and find the depth of flow for retardance Class C.

1. Assume a depth of 1.0 foot and calculate the following (see Figure 5-2):

$$A = (b + zd)\ d$$

$$A = [10+(3)\ (1)]\ (1)$$

$$A = 13.0 \text{ square feet}$$

$$R = \frac{(b + zd)\ d}{b + 2d(1 + z^2)^{1/2}}$$

$$R = \frac{[10+(3)\ (1)]\ (1)}{10+(2)\ (1)(1+3^2)^{1/2}}$$

$$R = 0.796 \text{ feet}$$

2. Find the velocity

v = 50/13.0
v = 3.85 feet/second

3. Find the value of vR.

   vR = (3.85) (0.796) = 3.06

4. Using the vR product from Step 3, find Manning's n from Figure 5-1 for retardance Class C.

   n = 0.047

5. Use Figure 5-11 or Equation 5-2 to find the velocity for S = 0.015, R = 0.796, and n = 0.047.

   v = 3.34 feet/second

6. Since 3.34 feet/second is less than 3.85 feet/second, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:

Assumed Depth (ft)	Area ft2	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 5-1)	Velocity (Eq. 5-2)
1	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.0475	3.39
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

7. Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 feet for freeboard to give a design depth of 1.3 feet. Design data for the trapezoidal channel are summarized as follows:

   Vegetation lining = grass mixture, v_m = 4 feet/second

   Q = 50 cfs

   b = 10 feet, d = 1.3 feet, z = 3, S = 0.015

   Top width = (10) + (2) (3) (1.3) = 17.8 feet

   n (stability) = 0.040, d = 1.0 foot, v = 3.9 feet/second, Froude number = 0.76 (Eqn 5-13)

   n (capacity) = 0.048, d = 1.1 feet, v = 3.45 feet/second, Froude number = 0.64 (Eqn 5-13)

Example 5-4. Grassed Channel Vegetation Comparison

A trapezoidal channel is required to carry 97 cfs at a bottom slope of 0.0275 foot/foot. Compare channel dimensions required for a grass mixture and bermudagrass.

1. Design stability calculations for a grass mixture are summarized below:

   V_m = 4 feet/second (From Table 5-2)

   Retardance Class = D

Assumed N Value	vR (Fig. 5-1)	R (Eq. 5-14)	vR (Eq. 5-15)
0.040	3.2	0.80	4.26
0.041	2.85	0.71	3.43
0.042	2.6	0.65	2.87
0.043	2.4	0.60	2.45

   Select m = 0.043 for stability criteria.

2. To obtain a hydraulic depth of 0.6 feet, a 39-foot bottom width is required. Depth calculations are summarized as follows:

Assumed Depth (ft)	Area (ft2)	R (ft)	$AR^{2/3}$	$\frac{Q/n}{1.49S^{0.5}}$	v (ft/sec)
0.40	16.08	0.387	8.54	16.88	3.05
0.50	20.25	0.480	12.42	16.88	3.52
0.60	24.48	0.572	16.87	16.88	3.96

Find normal depth = 0.60 feet, v = 3.96 feet/second

3. Design capacity calculations for the grass mixture are summarized as follows:

   Retardance Class = C

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (cfs)	vR	n (fig 5-1)	v (ft/sec)
0.60	24.48	0.572	3.96	2.27	0.054	3.15
0.65	26.62	0.617	3.64	2.25	0.054	3.32
0.70	28.77	0.662	3.37	2.23	0.054	3.48
0.69	28.34	0.653	3.42	2.23	0.054	3.45

Select n = 0.054 and d = 0.69 feet.

4. Design stability calculations for bermudagrass are summarized below:

v_m = 6 feet/second (from Table 5-2)

Retardance Class = D

Assumed n Value	vR (Fig 5-1)	R (Eq 5-14)	vR (Eq 5-15)
0.035	5.4	0.90	5.92
0.036	4.8	0.80	4.73

Select n = 0.036 for stability criteria.

5. To obtain a hydraulic depth of 0.8 feet, a 15-foot bottom width is required. Depth calculations are as follows:

Assumed Depth (ft)	Area (ft^2)	R (ft)	$AR^{2/3}$	$1.49S^{0.5}$	v (ft/sec)
0.8	13.92	0.694	10.91	14.13	5.38
0.9	15.93	0.770	13.38	14.13	5.77
0.93	16.54	0.792	14.17	14.13	5.88

Find normal depth = 0.93 feet, v = 5.88 feet/second.

6. Design capacity calculations for bermudagrass using retardance Class C are summarized as follows:

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (cfs)	vR	n (fig 5-1)	v (ft/sec)
0.98	17.58	0.829	5.52	4.58	0.040	5.45
1.0	18.00	0.844	5.39	4.55	0.040	5.52
1.02	18.42	0.859	5.27	4.52	0.042	5.32

Select n = 0.042 and d = 1.02 feet.

7. Vegetative linings are compared as follows:

	Grass Mixture	Bermudagrass
Bottom width	39	15
Side Slope	3	3
n (stability)	0.043	0.036
d (stability)	0.60	0.93
v (stability)	3.96	5.87
Fr (capacity)	0.919	1.15
n (capacity)	0.054	0.042
d (capacity)	0.69	1.02
d (d/freeboard)	0.84	1.22
v (capacity)	3.44	5.29
Fr (capacity)	0.75	1.00
Top Width (w/freeboard)	44.0	22.3

Example 5-5. Riprap Design

A natural channel has an average backfill channel velocity of 8 feet per second with a top width of 20 feet and a bend radius of 50 feet. The depth over the toe of the outer bank is 5 feet. Available stone weight is 170 pounds per cubic foot. Stone placement is on a side slope of 2:1 (horizontal: vertical).

1. Use 8 feet per second as the design velocity, because the reach is short and the bend in not protected.

2. Determine the bend correction coefficient for the ration of R_b/T = 50/20 = 2.5. From Figure 5-17, C_b = 1.55. The adjusted effective velocity is (8)(1.55) = 12.4 feet per second.

3. Determine the correction coefficient for the specific weight of 170 pounds from Figure 5-18 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 feet per second.

4. Determine minimum d_{30} from Figure 5-19 or Equation 5-18 as about 10 inches.

5. An available gradation has a minimum d_{30} size of 12 inches and is acceptable. It has enough fines that a filter course will not be required.

6. (Optional) Another gradation is available with a d_{30} of 8 inches. The ratio of desired to standard stone size is 8/10 or 0.8. From Figure 5-20, this gradation would be acceptable if the blanket thickness were increased if the blanket thickness were increased from the original d_{100} thickness by 35 percent (a ratio of 1.35 on the horizontal axis).

7. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 5-16.

5.4 GRADUALLY VARIED FLOW

The most common occurrence of gradually carried flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. 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 program, HEC-RAS, was developed by the U.S. Army Corps of Engineers (1988) and is recommended for floodwater profile computations. This program can be used to computer water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the Direct Step Method, as presented by Chow (1959). For an irregular nonuniform channel, the Standard Step Method, also presented by Chow (1959), is recommended, although it is a more tedious and iterative process. The use of HEC-RAS is recommended for standard step calculations.

5.5 RAPIDLY VARIED FLOW

Rapidly varied flow common to storm drainage systems occurs at flow control structures, hydraulic jumps, and bridges. Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions and equations for broad-crested weirs, v-notch weirs, and orifices are presented in Chapter 9. The hydraulic jump and bridges are briefly discussed below.

5.5.1 HYDRAULIC JUMP

A hydraulic jump can occur when flow passes rapidly from supercritical to subcritical depth and can be designed to dissipate highly erosive velocities. The evaluation of a hydraulic jump should consider the high energy loss and erosive forces that are associated with the jump. For rigid-lined facilities such as pipes or concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity. For grass-lined channels, unless the erosive forces are controlled, serious damage will result. Control is usually obtained by check dams or grade control structures that confine the erosive forces to a protected area. Flexible material, such as riprap, rock, or rubble, usually affords the most effective protection.

The analysis of the hydraulic jump inside storm sewers must be approximate, because of the lack of data for circular, elliptical, or arch sections. The jump can be approximately located by intersecting the energy grade line of the supercritical and subcritical flow reaches. The primary concerns are whether the pipe can withstand the forces, which may separate the joint or damage the pipe wall, and whether the jump will affect the hydraulic characteristics. The effect on pipe capacity can be determined by evaluating the energy grade line, taking into account the energy lost by the jump. In general, for Froude numbers less than 2.0, the loss of energy is less than 10 percent.

For long box culverts with a concrete bottom, the concerns about jump are the same as for storm sewers. However, the jump can be adequately defined for box culverts/sewers and for spillways using the jump characteristics of rectangular sections.

The relationship between variables for a hydraulic jump in rectangular channels can be expressed as:

$$d_2 = -\frac{d_1}{2} + \left(\frac{d_1^2}{4} + \frac{2v_1^2 d_1}{g} \right)^{1/2} \qquad (5\text{-}20)$$

where:

d_2 = Depth below jump, in feet

d_1 = Depth above jump, in feet

v_1 = Velocity above jump, in feet/second

g = Acceleration due to gravity, 32.2 feet second^2

A nomograph for solving Equation 5-20 is presented in Figure 5-21. Additional details on evaluating hydraulic jumps can be found in publications by French (1985) USDOT, FHWA (HEC-14, 1983), Chow (1959), and Peterska (1978).

5.5.2 BRIDGES

Water surface profile calculations in the vicinity of bridges are generally performed using a computer program such as HEC-RAS or WSPRO (USDOT, FHWA, HY-7, 1986). The procedure selected should be consistent with results from previous studies, if available. For example, if changes to regulatory flood evaluations and floodways are to be evaluated, the program and data files for the original study should be used.

Table 5-1
MAXIMUM VELOCITIES FOR
COMPARING LINING MATERIALS

Materials	Maximum Velocity[a] (feet/second)
Bare Soil	1.50
Silt or fine sand	1.75
Sandy loam	1.75
Silt loam	2.00
Stiff clay	3.75
Sod	4.0
Lapped sod	5.5
Vegetation	Use Table 5-2
Rigid[a]	10

[a] Higher velocities may be acceptable for rigid linings if energy dissipation is provided (see USDOT, FHWA, HEC-14, 1983)

Table 5-2
MAXIMUM VELOCITIES FOR VEGETATION CHANNEL LININGS

Vegetation Type	Slope Range (%)	Maximum Velocity[a] (feet per second)
Bermudagrass	0-5	6
	5-10	5
Kentucky bluegrass	0-5	5
Buffalo grass	5-10	4
Grass Mixture	0-5	4
	5-10	3
Lespedeza sericea Kudzu, alfalfa	0-5	2.5
Annuals	0-5	2.5

Reference: USDA, TP-61 (1947).

[a] Based on erosive soils.

Table 5-3
RECOMMENDED MANNING'S n VALUES FOR ARTIFICIAL CHANGES

		n-value Depth Ranges		
Lining Category	Lining Type	0-0.5 ft	0.5-2.0 ft	2.0 ft
Rigid	Concrete (Broom or Float Finish)	0.015	0.020	0.020
	Gunite	0.022	0.020	0.020
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglas Roving	0.028	0.021	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch (2.5-cm) D_{50}	0.044	0.033	0.030
	2-inch (5-cm) D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch (15 cm) D_{50}	0.104	0.069	0.035
	12-inch (30 cm) D_{50}	N/A	0.078	0.040

Reference: USDOT, FHWA, HEC-15 (1986).

Table 5-4
CLASSIFICATION OF VEGETAL COVERS AS TO DEGREE OF RETARDANCE

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (average 30”) (76 cm)
	Yellow bluestem Ischaemum	Excellent stand, tall (average 36”) (91 cm)
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12”) (30 cm)
	Native grass mixture (little bluestem, bluestem, blue gamma, and other long and short Midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall, (average 24”) (61 cm)
	Lespedeza sericea	Good stand, not woody, tall (average 19”) (48 cm)
	Alfalfa	Good stand, uncut (average 11”), (28 cm)
	Weeping lovegrass	Good stand, unmowed (average 13”) (33 cm)
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 11”) (28 cm)
C	Crabgrass	Fair stand, uncut (10 to 48”) (25 to 120 cm)
	Bermuda grass	Good stand, mowed (average 6”) (15 cm)
	Common lespedeza	Good stand, uncut (average 11”) (28 cm)
	Grass-legume mixture—summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 to 8 inches) (15 to 20 cm)
	Centipedegrass	Very dense cover (average 6 inches) (15 cm)
	Kentucky bluegrass	Good stand, headed (6 to 12 inches) (15-30 cm)
D	Bermuda grass	Good stand, cut to 2.5-inch height (15 cm)
	Common lespedeza	Excellent stand, uncut (average 4.5”) (11 cm)
	Buffalo grass	Good stand, uncut (3 to 6 inches) (8 to 15 cm)
	Grass-legume mixture fall, spring (orchard grass, redtop, Italian ryegrass,	Good stand, uncut (4 to 5 inches) (10 to 13 cm)
	and common lespedeza)	After cutting to 2-inch height (5 cm)
	Lespedeza serices	Very good stand before cutting
E	Bermuda grass	Good stand, cut to 1.5-inch height (4 cm)
	Bermuda grass	Burned stubble

Reference: USDA, TP-61 (1947).

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

Table 5-5
COEFFICIENTS FOR COMPUTING MANNING'S n VALUES
FOR NATURAL OR EXCAVATED CHANNELS USING COWAN'S EQUATION[a]

Channel Conditions		Values[b]	
Material Involved	Earth	n_0	0.020
	Rock Cut		0.025
	Fine Gravel		0.024
	Coarse Gravel		0.028
Degree of Irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of Channel Cross Section	Gradual	n_2	0.000
	Alternating Occasionally		0.005
	Alternating Frequently		0.010-0.015
Relative Effect of Obstructions	Negligible	n_3	0.000
	Minor		0.010-0.015
	Appreciable		0.020-0.030
	Severe		0.040-0.060
Vegetation	Low	n_4	0.005-0.010
	Medium		0.010-0.025
	High		0.025-0.050
	Very High		0.050-0.100
Degree of Meandering	Minor	n_5	1.000
	Appreciable		1.150
	Severe		1.300

[a]Cowan's Equation is presented as Equation 5-1

[b]From Chow (1959), Table 5-5, page 109.

D Pipe Diameter (inches)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = \frac{1.486}{n} \times A \times R^{2/3}$			
			n=0.010	n=0.011	n=0.012	n=0.013
8	0.349	0.167	15.8	14.3	13.1	12.1
10	0.545	0.208	28.4	25.8	23.6	21.8
12	0.785	0.250	46.4	42.1	38.6	35.7
15	1.227	0.312	84.1	76.5	70.1	64.7
18	1.767	0.375	137	124	114	105
21	2.405	0.437	206	187	172	158
24	3.142	0.500	294	267	245	226
27	3.976	0.562	402	366	335	310
30	4.909	0.625	533	485	444	410
33	5.940	0.688	686	624	574	530
36	7.069	0.750	867	788	722	666
42	9.621	0.875	1308	1189	1090	1006
48	12.566	1.000	1867	1698	1556	1436
54	15.904	1.125	2557	2325	2131	1967
60	19.635	1.250	3385	3077	2821	2604
66	23.758	1.375	4364	3967	3636	3357
72	28.274	1.500	5504	5004	4587	4234
78	33.183	1.625	6815	6195	5679	5242
84	38.485	1.750	8304	7549	6920	6388
90	44.170	1.875	9985	9078	8321	7681
96	50.266	2.000	11850	10780	9878	9119
102	56.745	2.125	13940	12670	11620	10720
108	63.617	2.250	16230	14760	13530	12490
114	70.882	2.375	18750	17040	15620	14420
120	78.540	2.500	21500	19540	17920	16540
126	86.590	2.625	24480	22260	20400	18830
132	95.033	2.750	27720	25200	23100	21330
138	103.870	2.875	31210	28370	26010	24010
144	113.100	3.000	34960	31780	29130	26890

Reference: American Concrete Pipe Association (1980).

TABLE 5-6
Full Flow C_1 Values for Circular Concrete Pipe

Pipe Size R x S (HE) S x R (VE) (Inches)	Approximate Equivalent Circular Diameter (Inches)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = \frac{1.486}{n} \times A \times R^{2/3}$			
				n = 0.010	n = 0.011	n = 0.012	n = 0.013
14 x 23	18	1.8	0.367	138	125	116	108
19 x 30	24	3.3	0.490	301	274	252	232
22 x 34	27	4.1	0.546	405	368	339	313
24 x 38	30	5.1	0.613	547	497	456	421
27 x 42	33	6.3	0.686	728	662	607	560
29 x 45	36	7.4	0.736	891	810	746	686
32 x 49	39	8.8	0.812	1140	1036	948	875
34 x 53	42	10.2	0.875	1386	1260	1156	1067
38 x 60	48	12.9	0.969	1878	1707	1565	1445
43 x 68	54	16.6	1.106	2635	2395	2196	2027
48 x 76	60	20.5	1.229	3491	3174	2910	2686
53 x 83	66	24.8	1.352	4503	4094	3753	3464
58 x 91	72	29.5	1.475	5680	5164	4734	4370
63 x 98	78	34.6	1.598	7027	6388	5856	5406
68 x 106	84	40.1	1.721	8560	7790	7140	6590
72 x 113	90	46.1	1.845	10300	9365	8584	7925
77 x 121	96	52.4	1.967	12220	11110	10190	9403
82 x 128	102	59.2	2.091	14380	13070	11980	11060
87 x 136	108	66.4	2.215	16770	15240	13970	12900
92 x 143	114	74.0	2.340	19380	17620	16150	14910
97 x 151	120	82.0	2.461	22190	20180	18490	17070
106 x 166	132	99.2	2.707	28630	26020	23860	22020
116 x 180	144	118.6	2.968	36400	33100	30340	28000

Reference: American Concrete Pipe Association (1980).

TABLE 5-7
Full Flow C_1 Values for Elliptical Concrete Pipe

Pipe Size R x S (Inches)	Approximate Equivalent Circular Diameter (Inches)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = \frac{1.486}{n} \times A \times R^{2/3}$			
				n = 0.010	n = 0.011	n = 0.012	n = 0.013
11 x 18	15	1.1	0.25	65	59	54	50
13½ x 22	18	1.6	0.30	110	100	91	84
15½ x 26	21	2.2	0.36	165	150	137	127
18 x 28½	24	2.8	0.45	243	221	203	187
22½ x 36¼	30	4.4	0.56	441	401	368	339
26⅝ x 43¾	36	6.4	0.68	736	669	613	566
31⁵⁄₁₆ x 51⅛	42	8.8	0.80	1125	1023	938	866
36 x 58½	48	11.4	0.90	1579	1435	1315	1214
40 x 65	54	14.3	1.01	2140	1945	1783	1646
45 x 73	60	17.7	1.13	2851	2592	2376	2193
54 x 88	72	25.6	1.35	4641	4219	3867	3569
62 x 102	84	34.6	1.57	6941	6310	5784	5339
72 x 115	90	44.5	1.77	9668	8789	8056	7436
77¼ x 122	96	51.7	1.92	11850	10770	9872	9112
87⅛ x 138	108	66.0	2.17	16430	14940	13690	12640
96⅞ x 154	120	81.8	2.42	21975	19977	18312	16904
106½ x 168¾	132	99.1	2.65	28292	25720	23577	21763

Reference: American Concrete Pipe Association (1980).

TABLE 5-8
Full Flow C_1 Values for Concrete Pipe Arch

TABLE 5-9
Full Flow C_1 Values for Precast Concrete Box Sections

Box Size Span x Rise (Feet)	A Area (Square Feet)	R Hydraulic Radius (Feet)	C = 1.486/n(A x $R^{2/3}$)		Box Size Span x Rise (Feet)	A Area (Square Feet)	R Hydraulic Radius (Feet)	C = 1.486/n(A x $R^{2/3}$)	
			n = 0.012	n = 0.013				n = 0.012	n = 0.013
3 X 2	5.78	0.63	524	484	9 X 5	43.88	1.67	7060	7070
3 X 3	8.78	0.78	923	852	9 X 6	52.88	1.87	9950	9180
4 X 2	7.65	0.69	743	686	9 X 7	61.88	2.05	12400	11400
4 X 3	11.65	0.90	1340	1240	9 X 8	70.88	2.20	14800	13700
4 X 4	15.65	1.04	1990	1840	9 X 9	79.88	2.33	17400	16100
5 X 3	14.50	0.98	1770	1630	10 X 5	48.61	1.73	8690	8020
5 X 4	19.50	1.16	2660	2460	10 X 6	58.61	1.95	11300	10462
5 X 5	24.50	1.30	3620	3340	10 X 7	68.61	2.14	14100	13000
6 X 3	17.32	1.04	2200	2030	10 X 8	78.61	2.31	17000	15700
6 X 4	23.32	1.25	3350	3100	10 X 9	88.61	2.46	20000	18500
6 X 5	29.32	1.42	4590	4240	10 X 10	98.61	2.59	23000	21300
6 X 6	35.32	1.56	5880	5430	11 X 4	42.32	1.52	6930	6390
7 X 4	27.11	1.33	4050	3740	11 X 6	64.32	2.02	12730	11700
7 X 5	34.11	1.52	5590	5160	11 X 8	86.32	2.41	19200	17700
7 X 6	41.11	1.68	7200	6650	11 X 10	108.32	2.72	26100	24100
7 X 7	48.11	1.82	8880	8200	11 X 11	119.32	2.85	29700	27400
8 X 4	31.11	1.39	4790	4420	12 X 4	46.00	1.55	7630	7050
8 X 5	39.11	1.60	6630	6120	12 X 6	70.00	2.08	14100	13000
8 X 6	47.11	1.78	8760	7920	12 X 8	94.00	2.50	21400	19800
8 X 7	55.11	1.94	10600	9790	12 X 10	118.00	2.83	29300	27000
8 X 8	63.11	2.07	12700	11700	12 X 12	142.00	3.11	37500	34600

Reference: American Concrete Pipe Association (1980).

Table 5-10
CRITICAL DEPTH EQUATIONS FOR UNIFORM FLOW IN SELECTED CHANNEL CROSS SECTIONS

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

where:

d_c = Critical depth, in feet
Q^c = Design discharge, in cfs
g = Acceleration due to gravity, 32.2 feet/second²
b = Bottom width of channel, in feet
z = 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

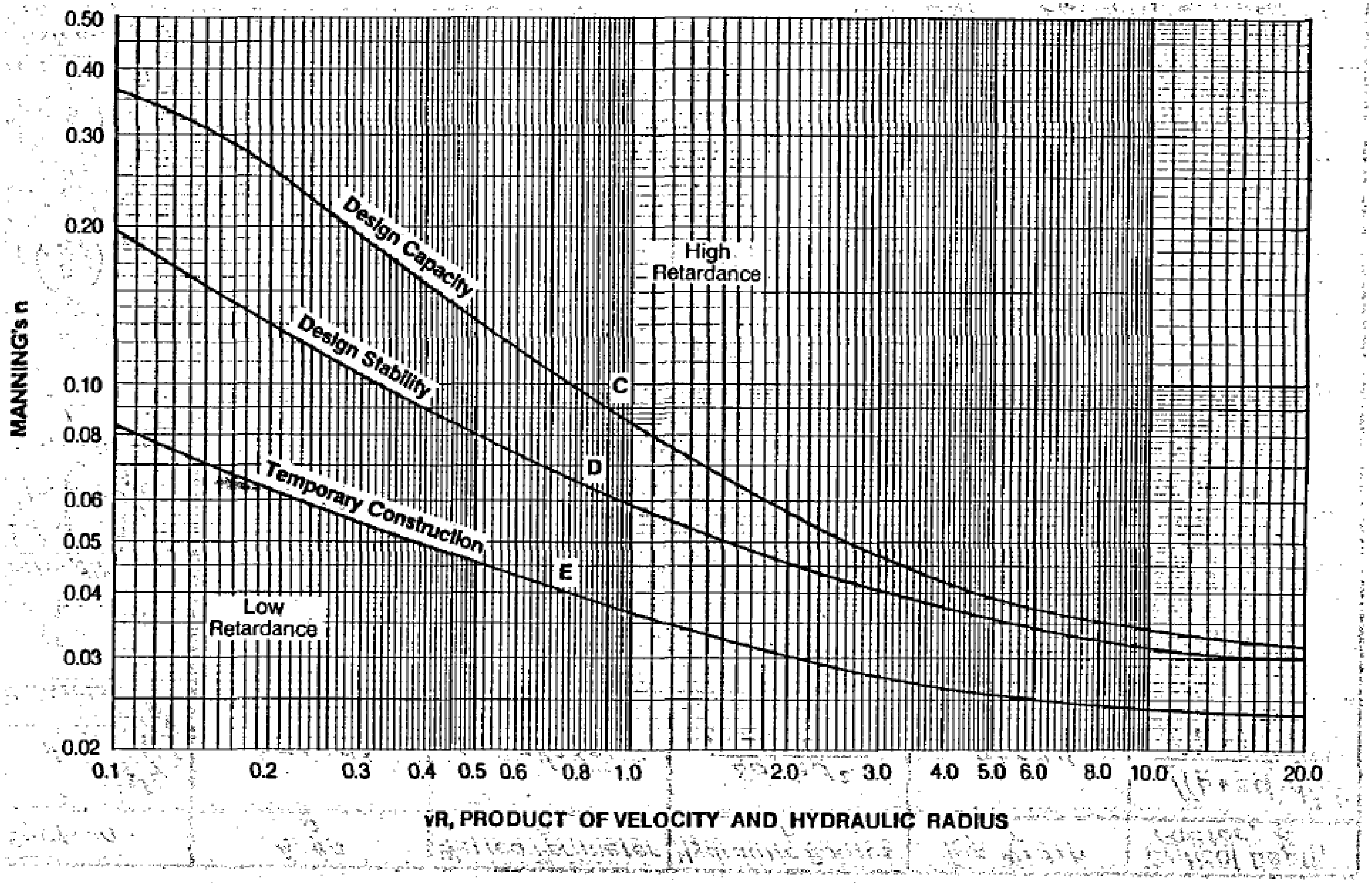
[a]See Figure 5-2 for channel sketches.

[b]Assumes uniform flow with the kinetic energy coefficient equal to 1.0.

[c]Reference: French (1985).

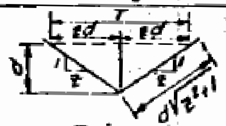
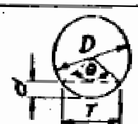
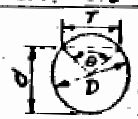
[d]Reference: USDOT, FHWA, HDS-4 (1965).

[e]Reference: Brater and King (1976).



Reference: USDA, TP-61 (1947).

FIGURE 5-1
Manning's n Values for Vegetated Channels

Section	Area a	Wetted Perimeter P	Hydraulic Radius r	Top Width T	Critical Depth Factor, Z
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}zd^{2.5}$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
 Circle - < ½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
 Circle - > ½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

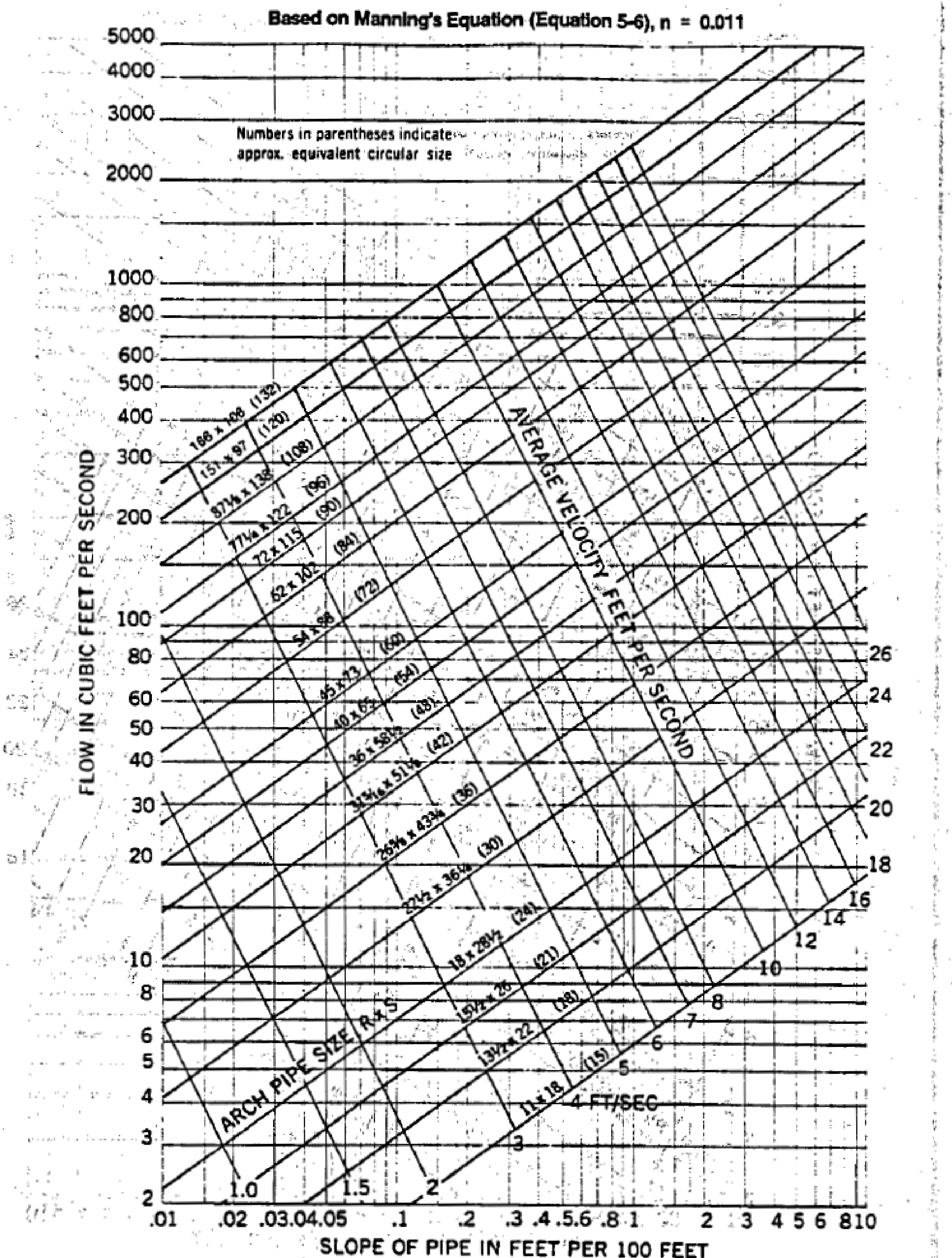
[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Reference: USDA, SCS, NEH-5 (1956).

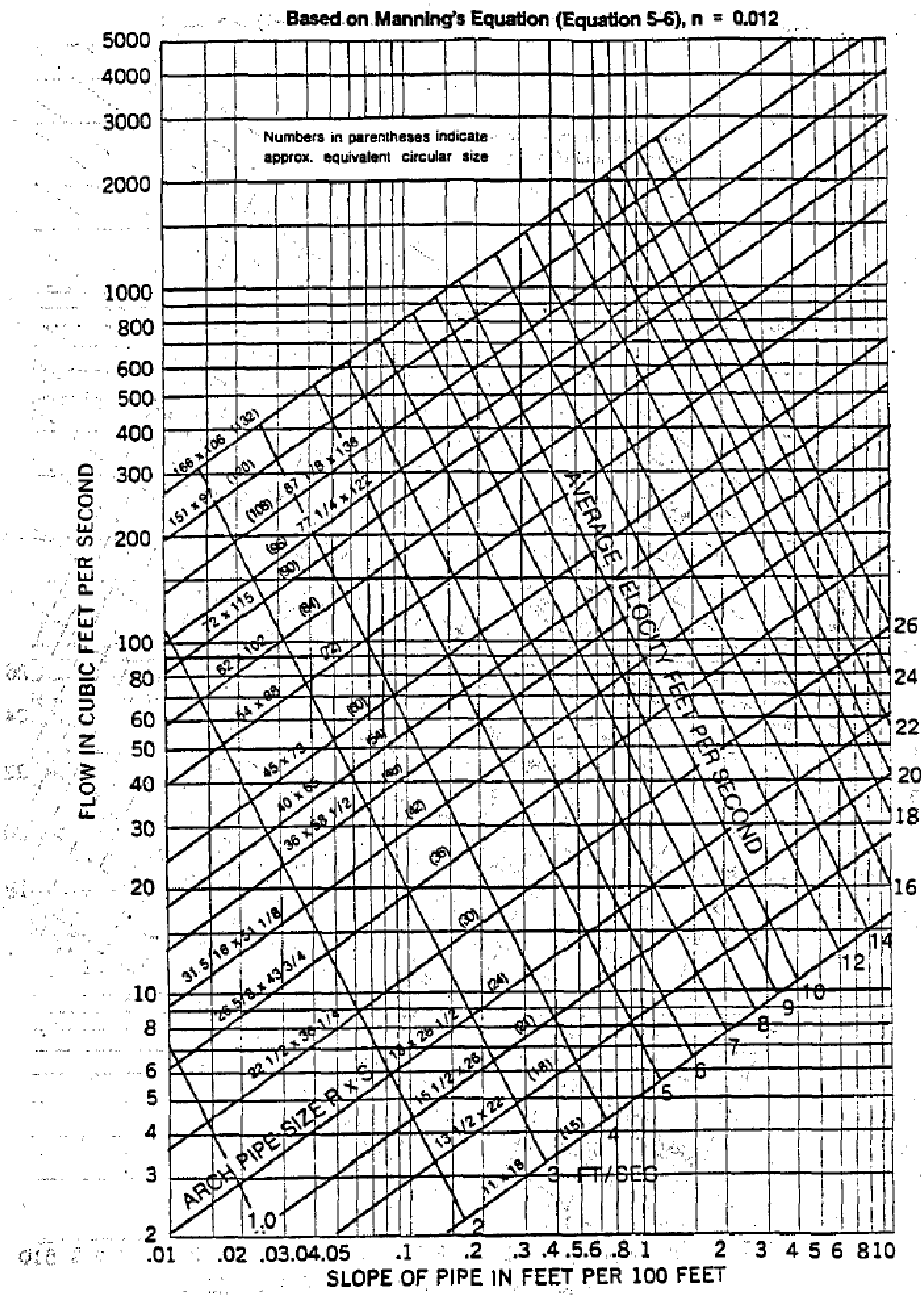
FIGURE 5-2
Open Channel Geometric Relationships for Various Cross Sections



Reference: American Concrete Pipe Association (1980).

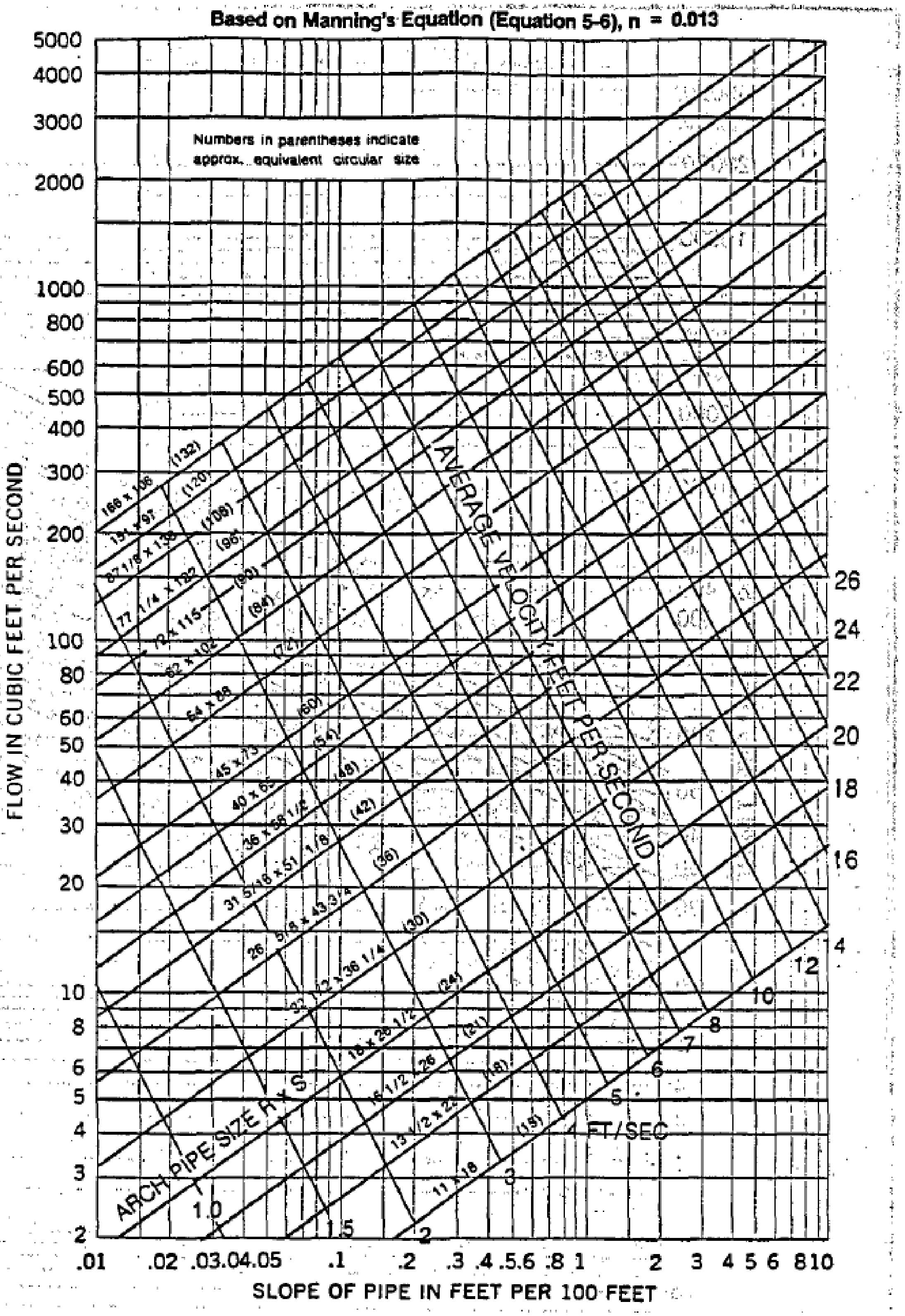
FIGURE 5-3

Capacity Chart for Pipe Arch Flowing Full, n = 0.011



Reference: American Concrete Pipe Association (1980).

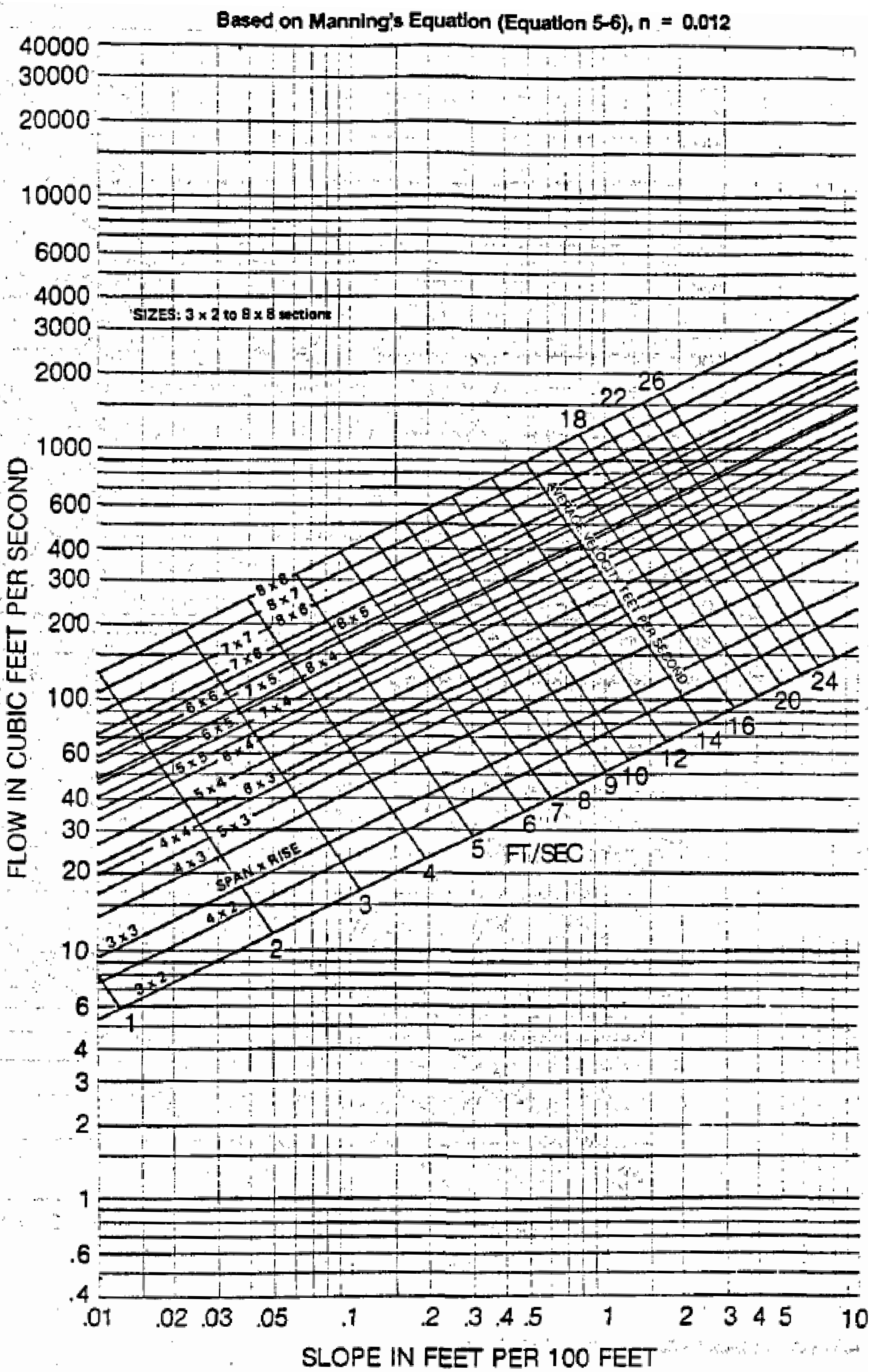
FIGURE 5-4
Capacity Chart for Pipe Arch Flowing Full, n = 0.012



Reference: American Concrete Pipe Association (1980).

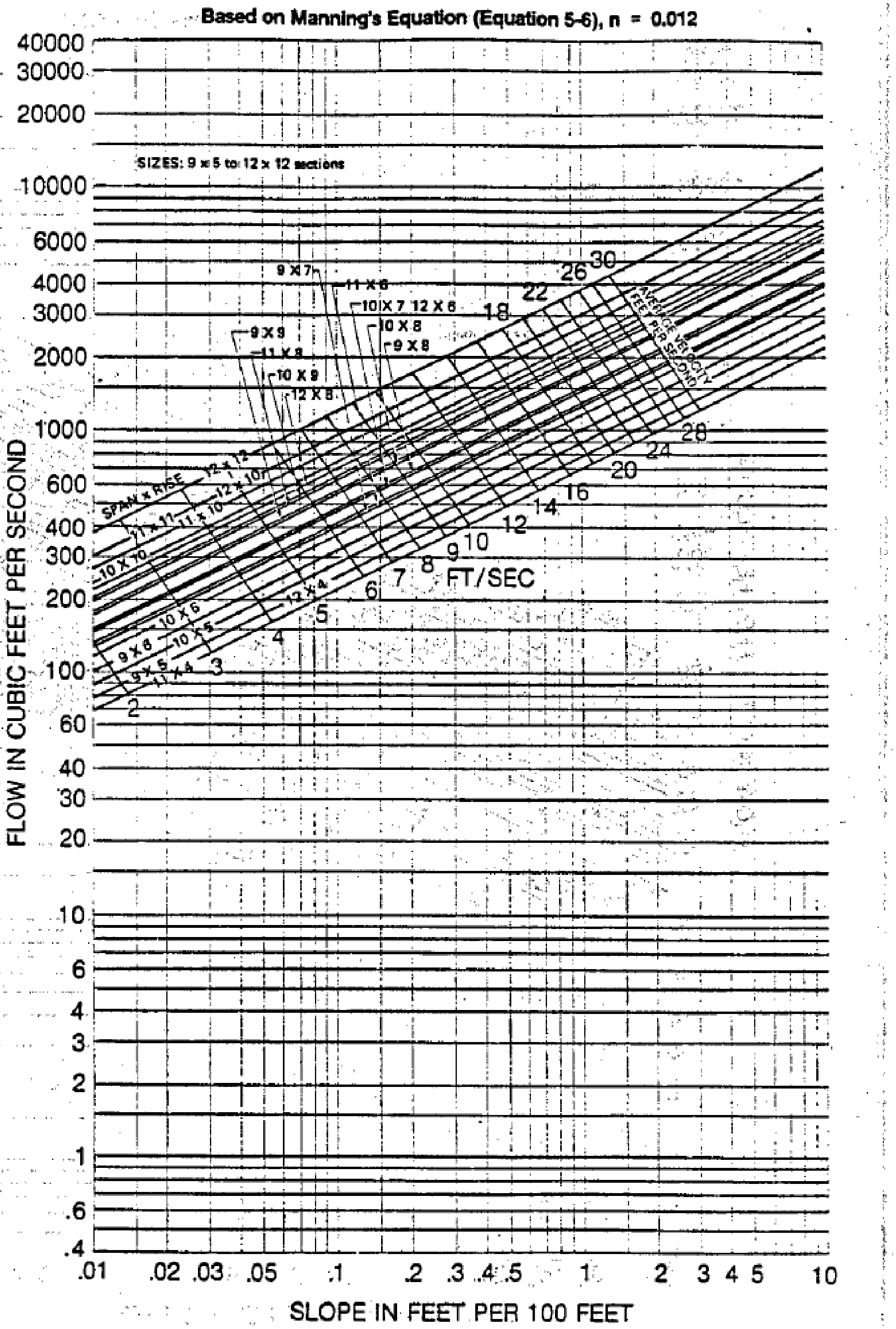
FIGURE 5-5

Capacity Chart for Pipe Arch Flowing Full, n = 0.013



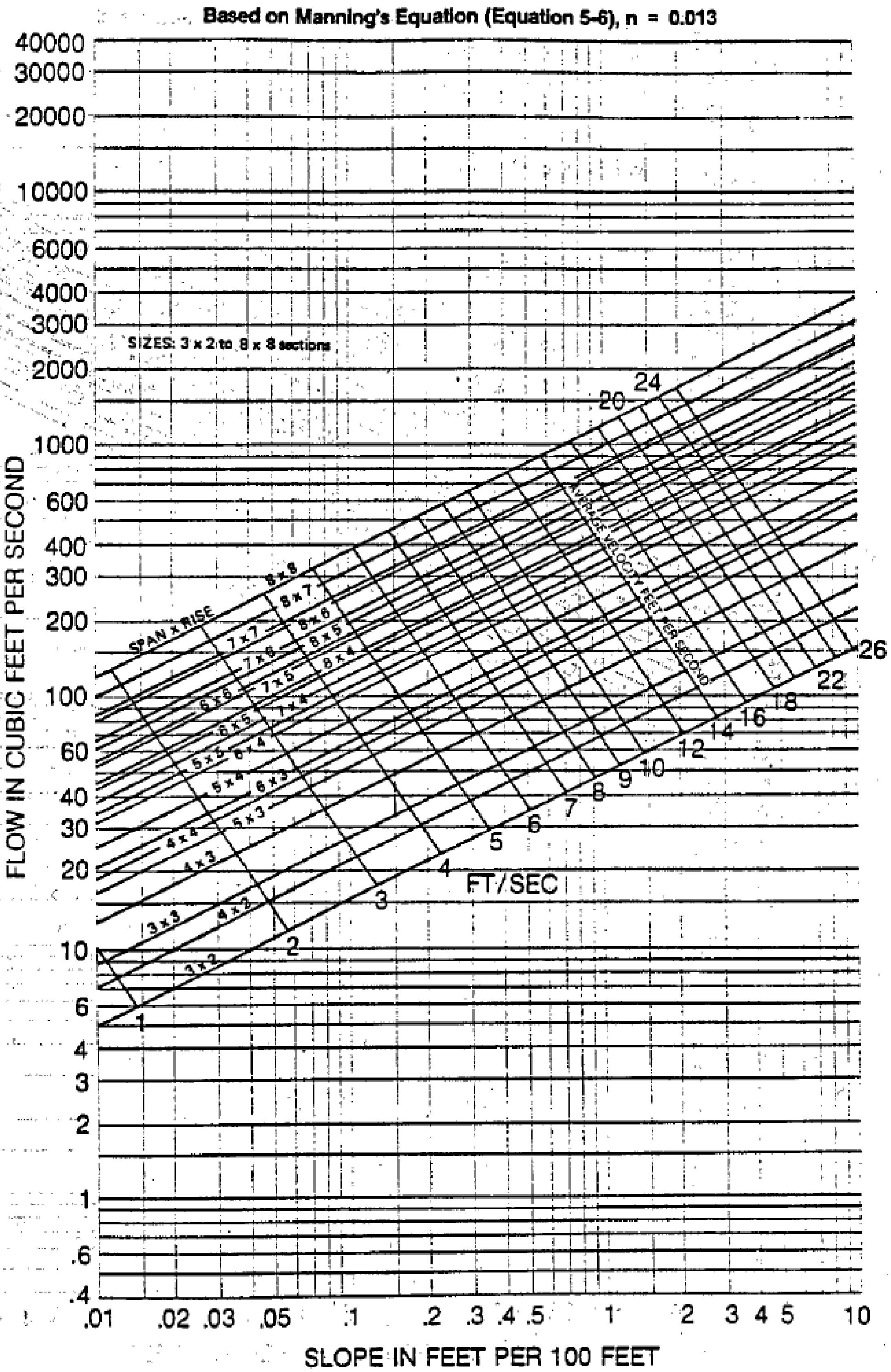
Reference: American Concrete Pipe Association (1980).

FIGURE 5-6
Capacity Chart for Box Sections Flowing Full, n = 0.012, Sizes 3' x 2' to 8' x 8'



Reference: American Concrete Pipe Association (1980).

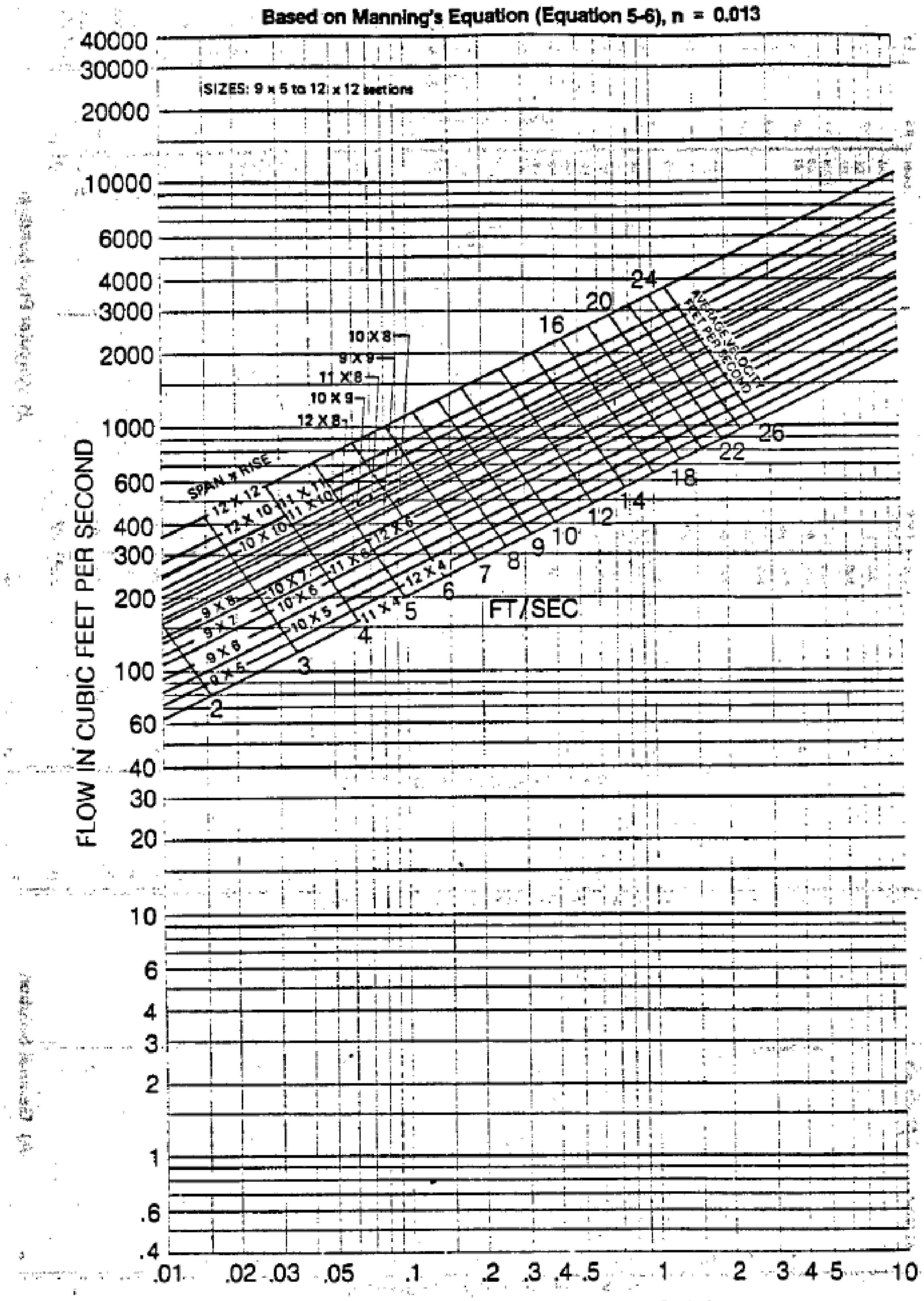
FIGURE 5-7
Capacity Chart for Box Sections Flowing Full, n = 0.012, Sizes 9′ x 5′ to 12′ x 12′



Reference: American Concrete Pipe Association (1980).

FIGURE 5-8

Capacity Chart for Box Sections Flowing Full, n = 0.013, Sizes 3′ x 2′ to 8′ x 8′



Reference: American Concrete Pipe Association (1980).

FIGURE 5-9

Capacity Chart for Box Sections Flowing Full, n = 0.013, Sizes 9′ x 5′ to 12′ x 12′

Reference: USDOT, FHWA, HDS-3 (1961).

Reference: American Concrete Pipe Association (1980).

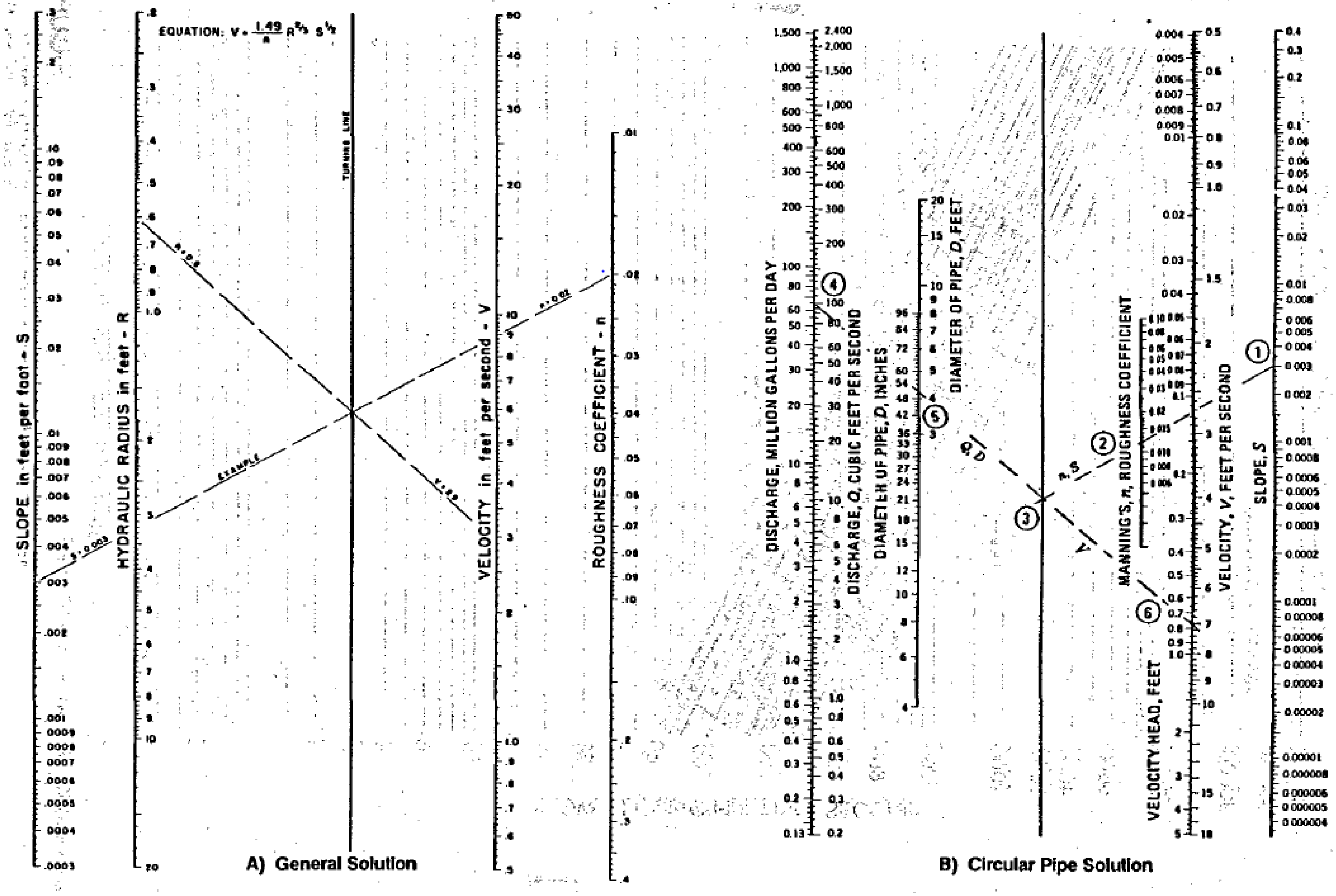
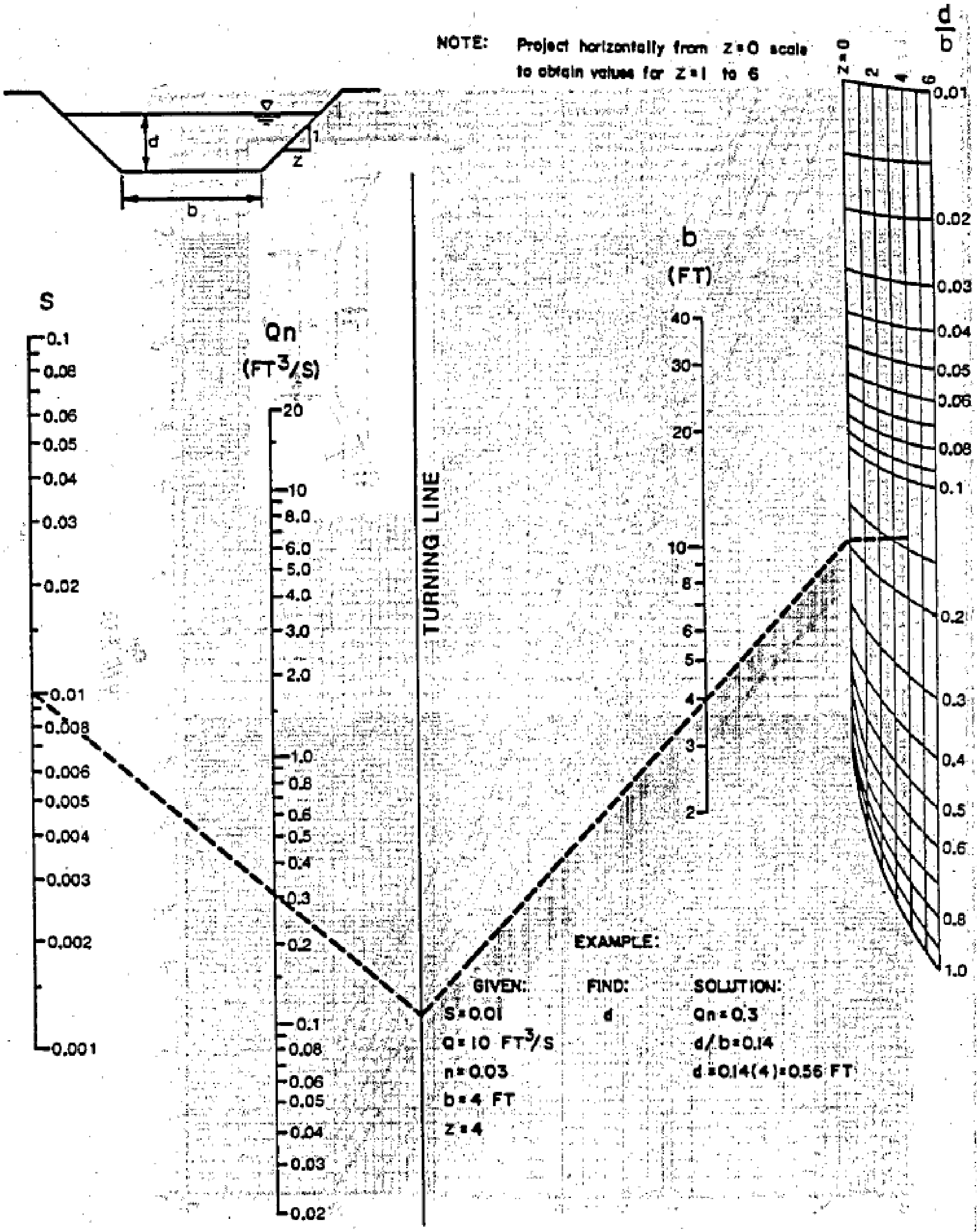


FIGURE 5-10
Nomographs for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

FIGURE 5-11
Solution of Manning's Equation for Trapezoidal Channels

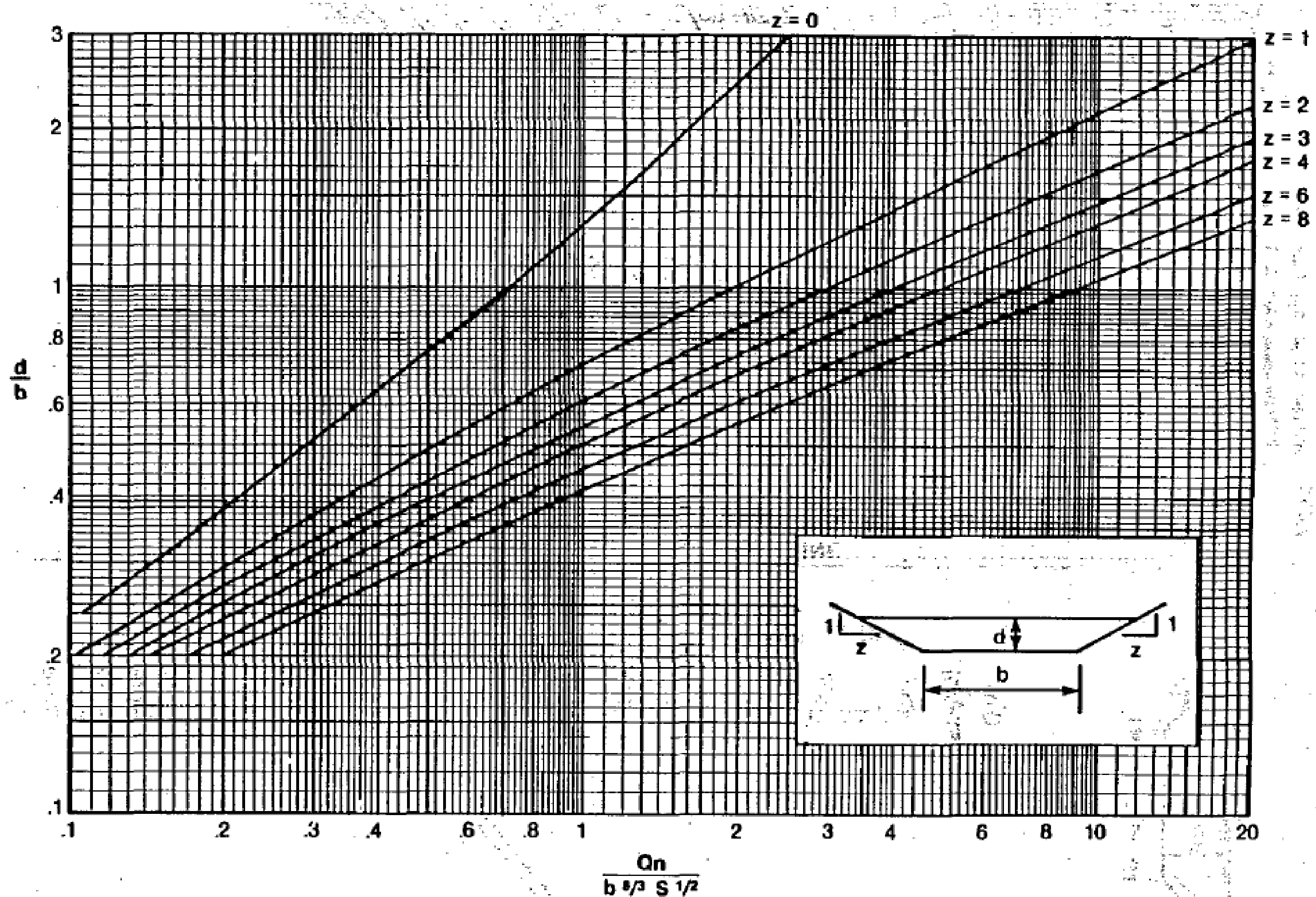


FIGURE 5-12
Trapezoidal Channel Capacity Chart

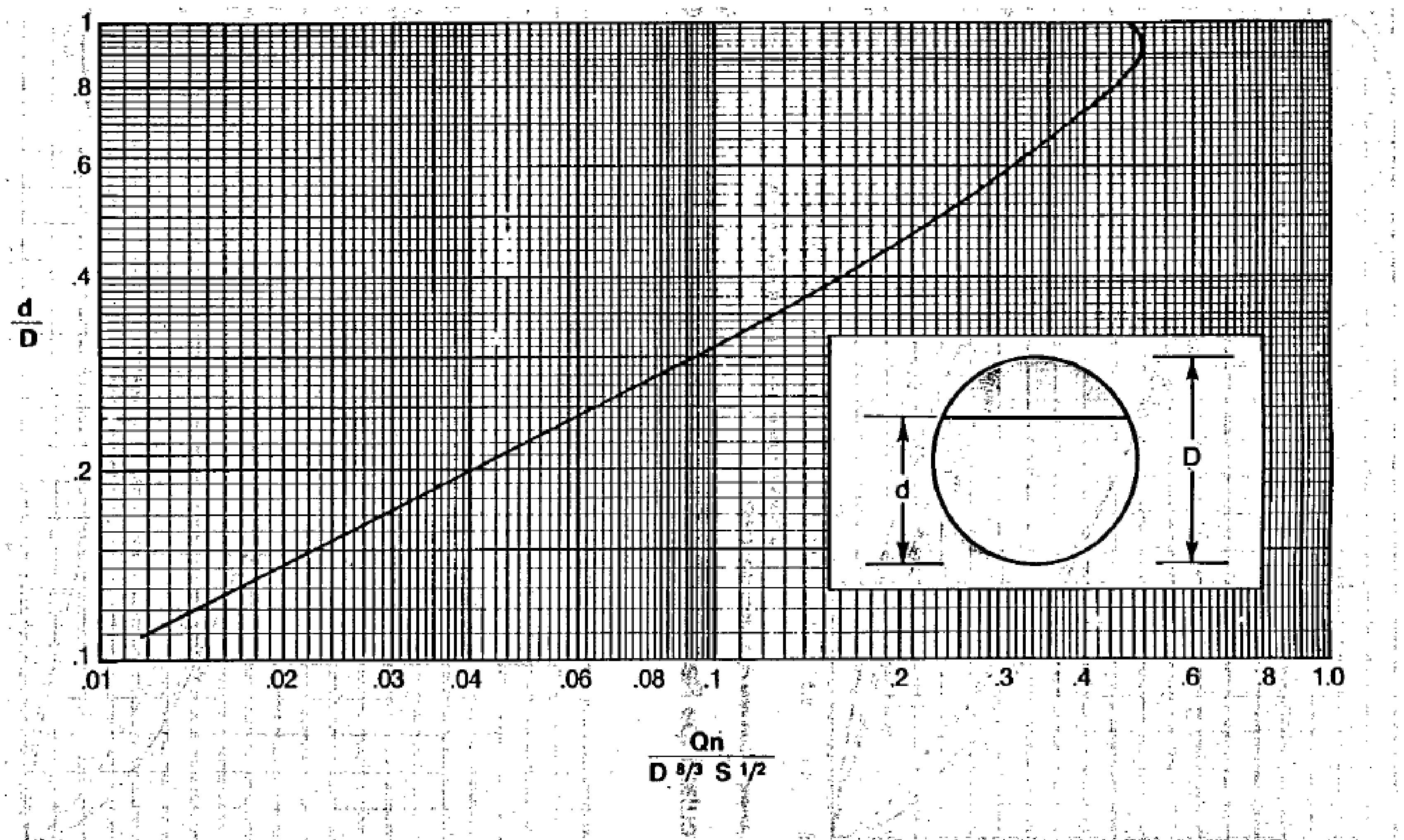
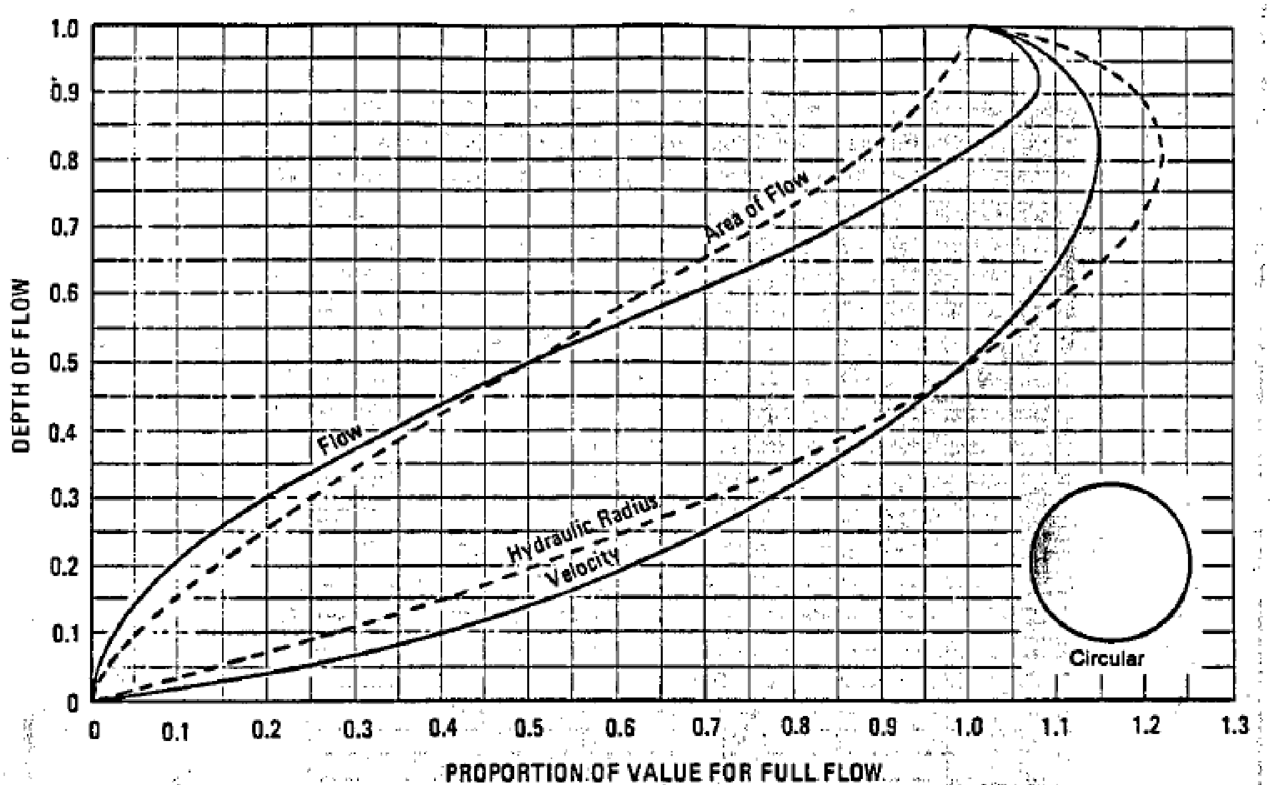


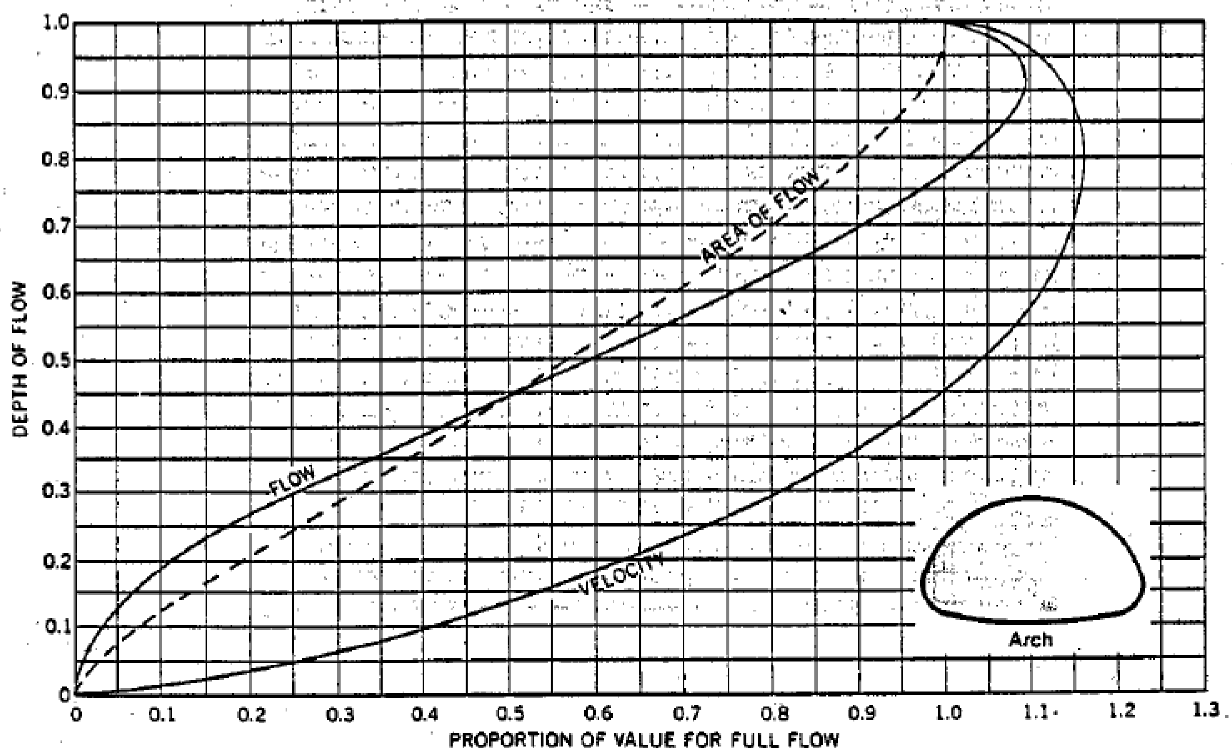
FIGURE 5-13
Circular Pipe Partial Flow Capacity Chart



Reference: American Concrete Pipe Association (1980).

FIGURE 5-14

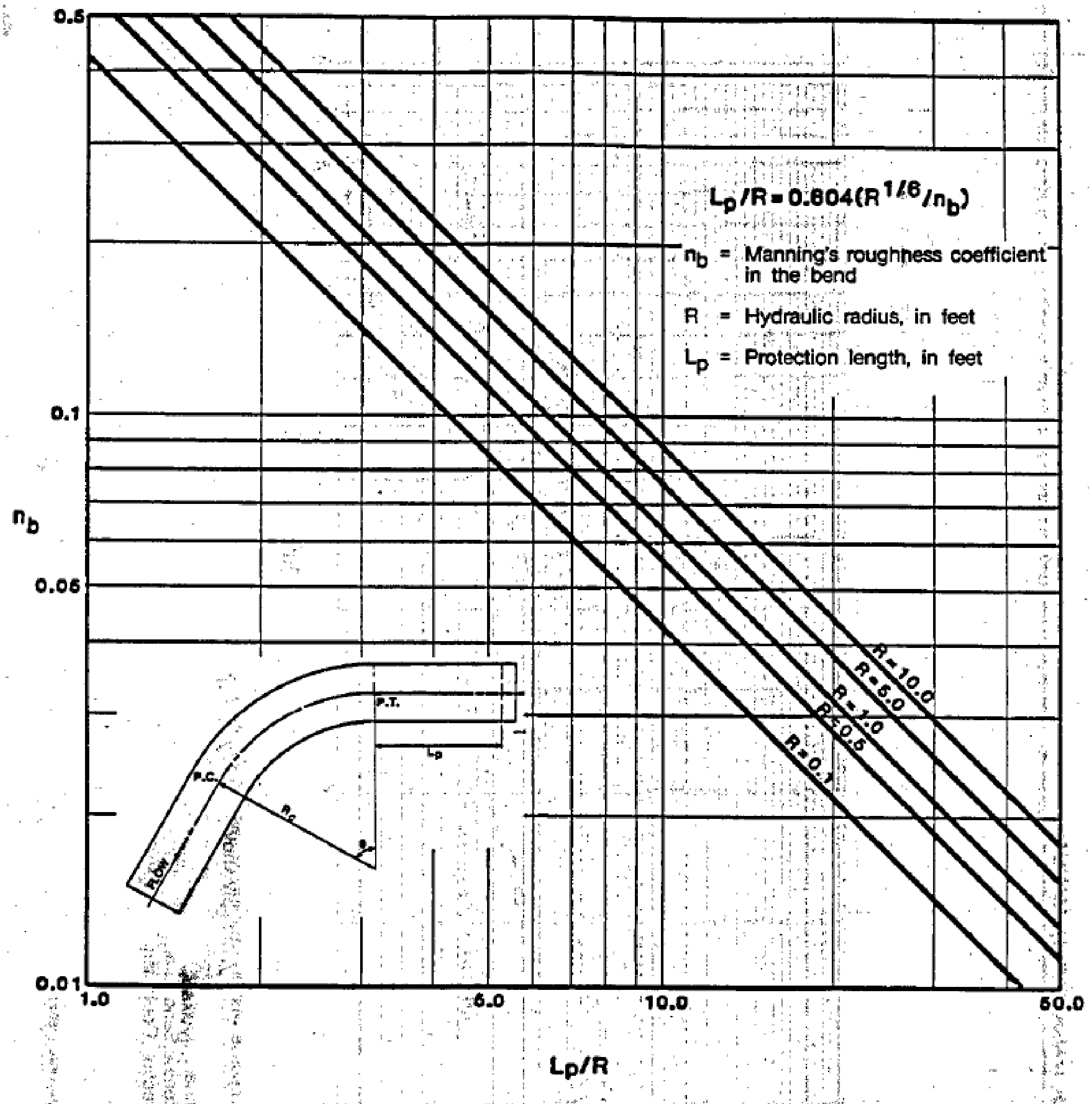
Circular Pipe Relative Flow, Area, Hydraulic Radius, and Velocity for Any Depth



Reference: American Concrete Pipe Association (1980).

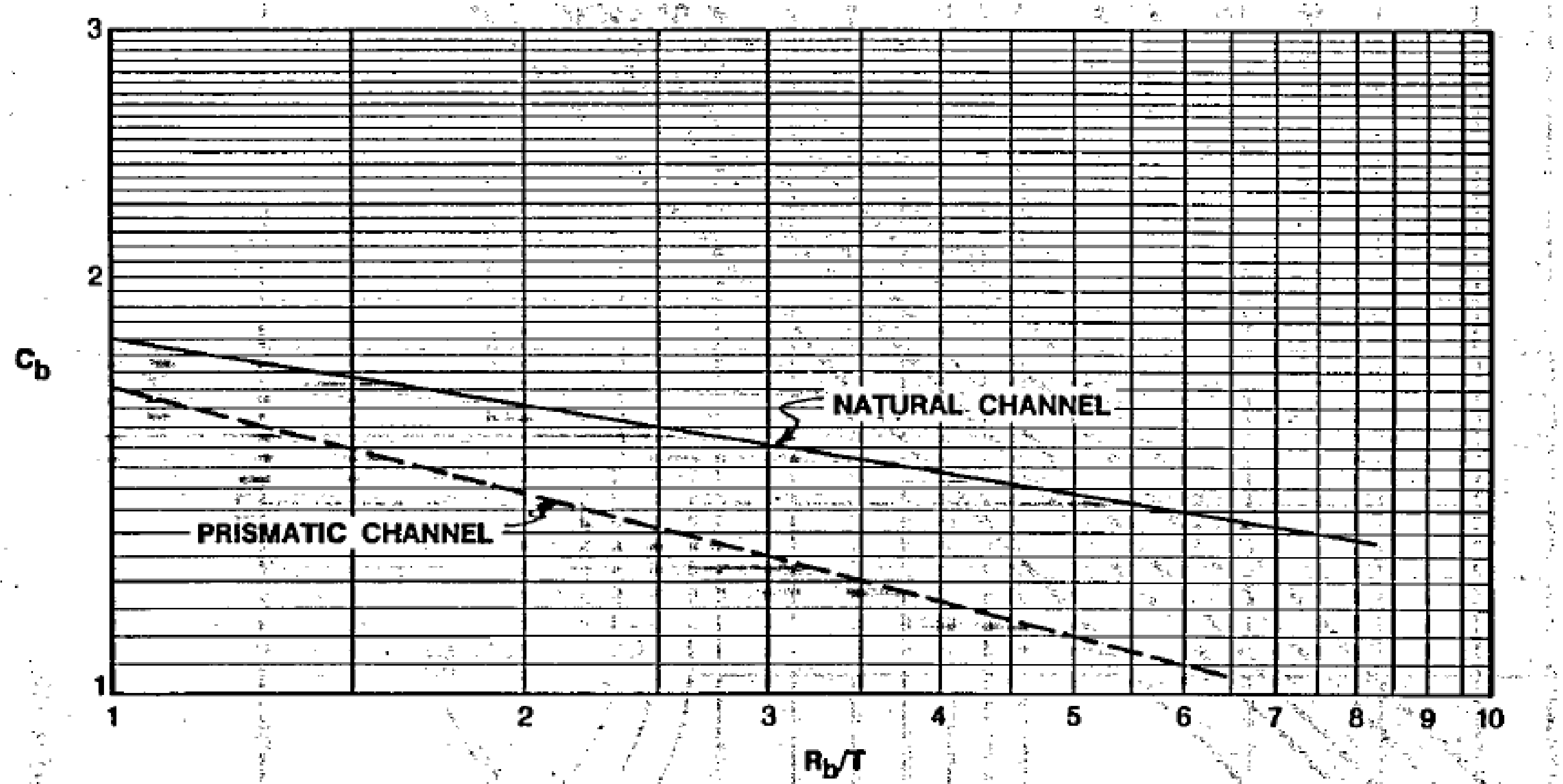
FIGURE 5-15

Pipe Arch Relative Flow, Area, and Velocity for Any Depth



Reference: USDOT, FHWA, HEC-15 (1986).

FIGURE 5-16
Protection Length, L_p, Downstream of Channel Bend

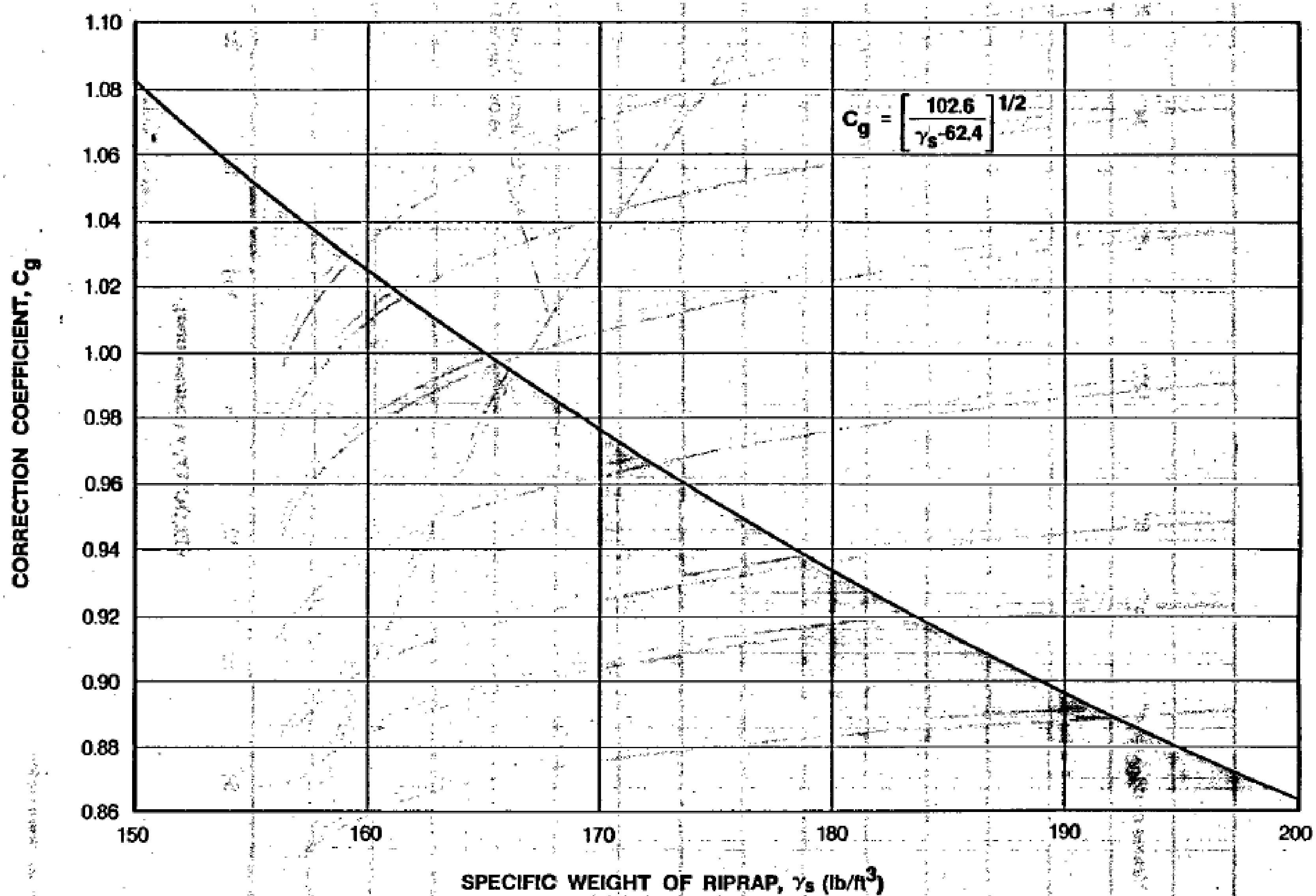


To obtain effective velocity, multiply known velocity by C_b.

T = Channel Top Width
R_b = Centerline Bend Radius
C_b = Correction Coefficient

Reference: Maynord (1987).

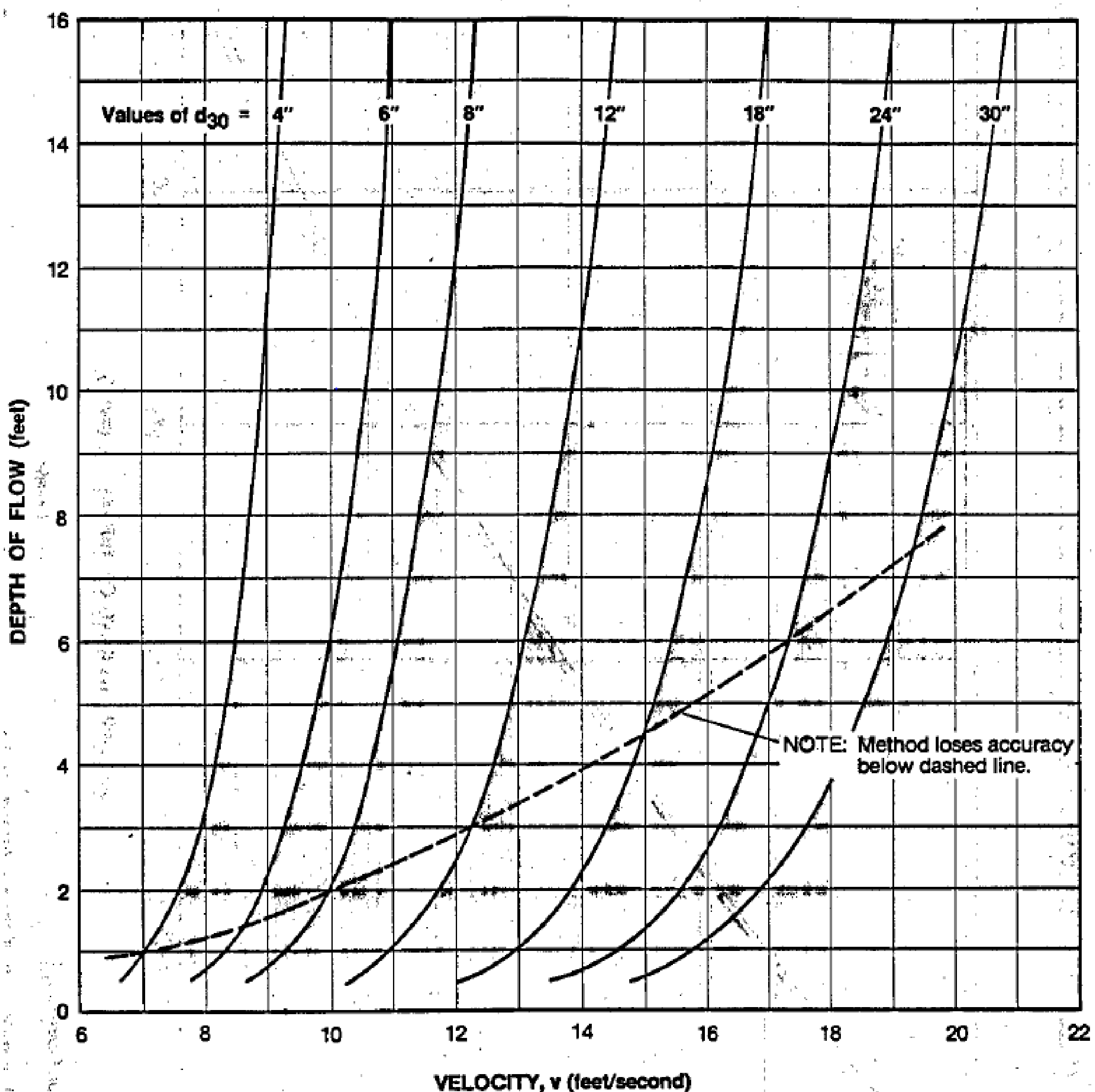
FIGURE 5-17
Riprap Lining Bend Correction Coefficient



C_g = Correction Coefficient

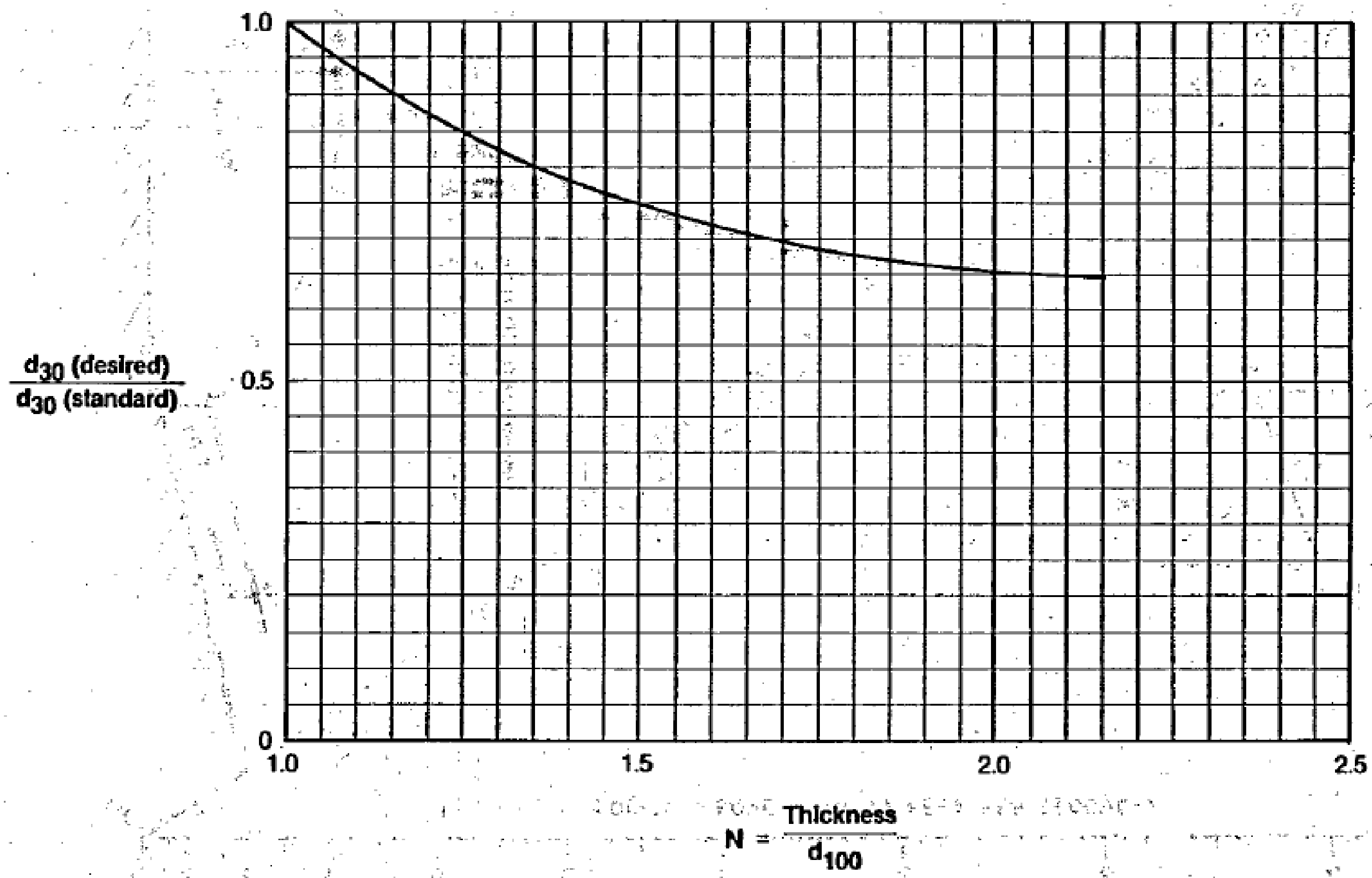
To obtain effective velocity, multiply known velocity by C_g.

FIGURE 5-18
Riprap Lining Specific Weight Correction Coefficient



Reference: Reese (1988).

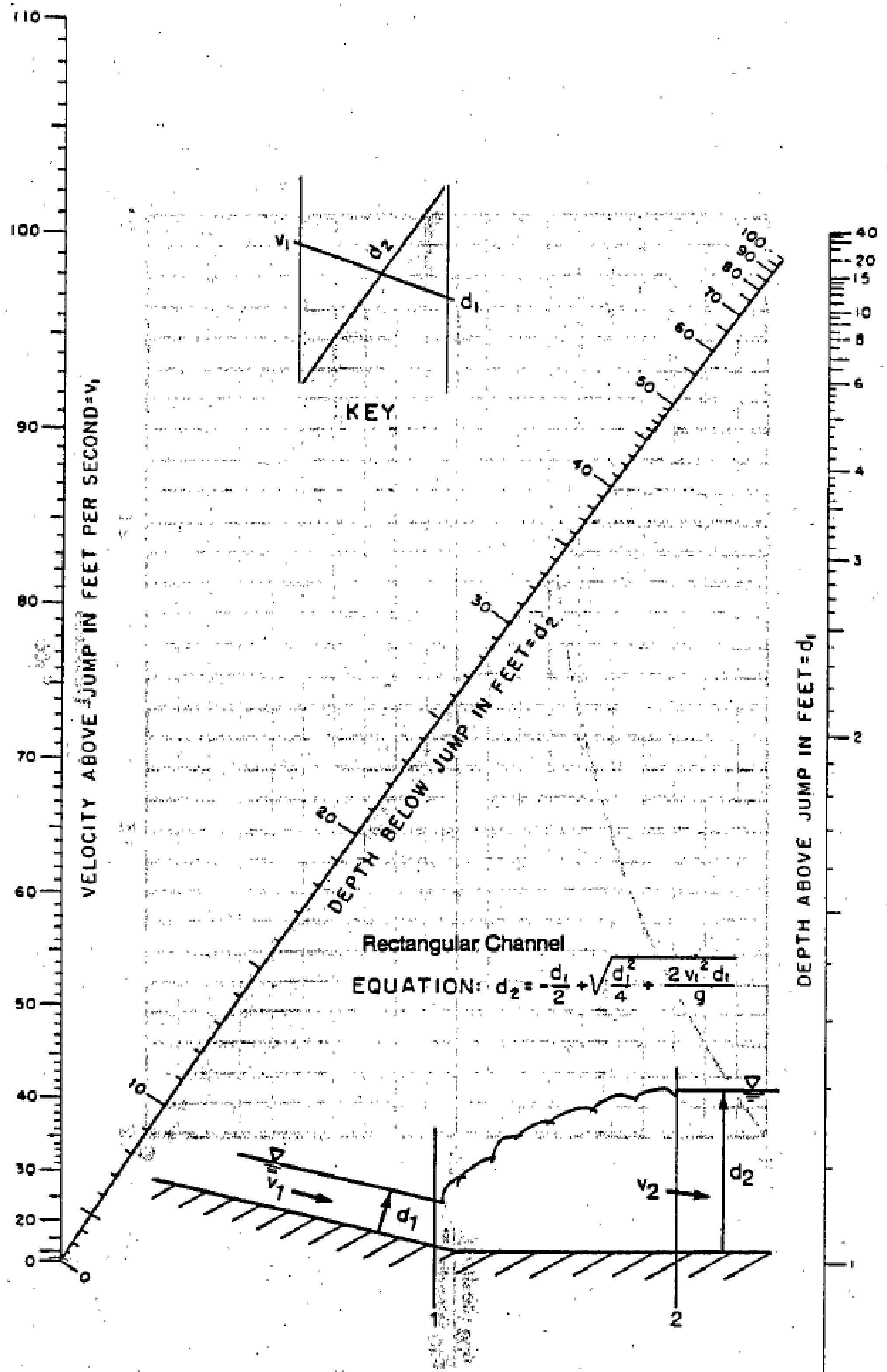
FIGURE 5-19
Riprap Lining d_{30} Stone Size as a Function of Mean Velocity and Depth



Reference: Maynord (1987).

FIGURE 5-20

Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 2.0 to 2.3



Reference: U.S. Department of the Interior (1973).

FIGURE 5-21
Nomograph for Solving the Rectangular Channel Hydraulic Jump Equation

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CHAPTER 6
CULVERT HYDRAULICS

6.1 FUNDAMENTALS

Culvert hydraulics can be classified and analyzed on the basis of a control section. A <u>control section</u> is a location where a unique relationship exists between the rate of flow and depth of flow or water surface elevation. The two basic types of control sections are <u>inlet</u> and <u>outlet</u> control.

6.1.1 <u>INLET CONTROL</u>

Inlet control exists when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section for this condition is located just inside the entrance. Critical depth occurs at or near this location and the flow in the culvert is supercritical. The following variables influence culvert performance at the inlet:

1. Headwater elevation
2. Inlet area
3. Inlet edge configuration
4. Inlet shape

Flow under inlet control may be described mathematically by either the weir formula, depending on the headwater depth. It is important to note that the tailwater elevation has no influence on capacity.

6.1.2 <u>OUTLET CONTROL</u>

Outlet control occurs when the culvert barrel or outlet has less capacity than the inlet. The control section for this situation is located at the barrel exit or downstream from the culvert. Either partially full subcritical flow or full pipe pressure flow conditions can occur. In addition to the variables influencing inlet performance listed above, the following factors can affect outlet performance:

1. Barrel roughness
2. Barrel area
3. Barrel shape
4. Barrel length
5. Barrel slope
6. Tailwater elevation

For outlet control, the difference between headwater and tailwater elevation represents the energy or head that conveys the flow through the culvert.

6.1.3 CULVERT SELECTION

In most situations, the hydraulic sizing of a culvert is a trial and error process. A trial culvert size is assumed and inlet and outlet performance are evaluated to determine if they will satisfy the conditions prevailing at the proposed location. A culvert system is selected by choosing the following items:

1. Inlet structure
2. Barrel material
3. Shape
4. Size
5. Outlet structure

The inlet and outlet structures are usually the same, to achieve a symmetrical installation. If the outlet velocity is high enough to cause erosion, protection or energy dissipation is required.

The procedures presented in the chapter were obtained from USDOT, FHWA, HDS-5 (1985).

6.2 DESIGN CRITERIA

The following parameters shall be considered when culvert hydraulic calculations are performed:

1. Discharge
2. Headwater
3. Tailwater
4. Manning's n values
5. Length and slope
6. Velocity limitations

6.2.1 DISCHARGE

The design discharge depends on an appropriate storm frequency (return period) and the protection desired for the public on the roadway or adjacent lands. The following criteria shall apply:

Major and Minor Arterials	25-year
Major Collectors	25-year
Minor Collectors	10-year
Residential and Local	10-year
Roadside Ditches	10-year

In addition to the design flow, the culvert capacity should be checked for the 100-year return period, to ensure that the overtopping flood conditions do not exceed 1 foot above the top of curb. If ponding occurs at the culvert entrance, and a reduction in discharge attributable to storage is appropriate, reservoir routing calculations can be used to estimate the reduction.

6.2.2 HEADWATER

The allowable headwater elevation is determined from an evaluation of conditions upstream of the culvert and the proposed or existing roadway elevation. The following criteria shall be analyzed:

1. Non-damaging or permissible upstream flooding elevations (e.g., existing buildings or flood elevation data from Section 3.3) should be identified. Headwater should be kept below these elevations.

2. Headwater depth for the design discharge should not exceed a height greater than 1.5 feet below the edge of the shoulder of a road.

3. Headwater depth for the design discharge should not cause water to rise above the top of approach channels adjacent to improved land or above the established floodplain (Q_{100}). Impacts to the established base flood (Q_{100}) elevation should be minimal

4. Other site-specific design considerations should be addressed as required.

The constraint that gives the lowest allowable headwater elevation shall establish the basis for hydraulic calculations.

6.2.3 TAILWATER

The hydraulic conditions downstream of the culvert site shall be evaluated to determine a tailwater depth for the design discharge. The following conditions are typical:

1. If the culvert outlet is operating in a free fall condition (e.g., a cantilever pipe), the critical depth and equivalent hydraulic grade line should be determined using procedures presented in Section 6.3.

2. For culverts that discharge to an open channel, tailwater depth is established by evaluation the normal depth of flow in the outlet channel using procedure identified in Chapter 5.

3. 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.

4. If the culvert discharges to a lake, pond, or other major waterbody, the expected high water elevation of the particular waterbody may establish the culvert tailwater.

6.2.4 MANNING'S n VALUES

All culverts shall be constructed using reinforced concrete pipe unless alternate materials of equal or greater effectiveness and strength are approved by the Director of City Engineering. Manning's n values for culvert capacity calculations shall be as follows:

Material	n Value
Circular and Arch Concrete Pipe	0.012
Circular Corrugated Metal Pipe (CMP)	0.024
Structural Plate CMP	0.0328-0.302
Box Concrete	0.012
Oval Concrete	0.012
Arch CMP	0.024
Structural Plate Arch	0.0327-0.0306

6.2.5 LENGTH AND SLOPE

The length and slope of a culvert should consider the following factors:

1. Chanel bottom of the stream being conveyed
2. Geometry of the roadway embankment
3. Skew angle of the culvert

In general, the culvert slope should be chosen to approximate existing topography with a minimum 0.5% slope on all culverts and a recommended slope of 1.0%

6.2.6 VELOCITY LIMITATIONS

A minimum velocity of 2.5 feet per second when the culvert is flowing full is recommended to ensure a self-cleaning condition during partial depth flow. When velocities below this minimum are anticipated, the installation of a sediment trap upstream of the culvert is required.

Riprap is required at all culvert outlets, unless some other material of equal or greater effectiveness and appearance is approved by the Director of City Engineering. The maximum velocity shall be consistent with channel stability requirements at the culvert outlet. As outlet velocities increase, the need for channel stabilization at the culvert outlet increases. If velocities exceed permissible velocities for the outlet lining material (see Chapter 5), energy dissipation is required.

6.3 DESIGN CALCULATIONS

A flow chart for performing culvert design calculations is provided in Figure 6-1 (see discussion in Section 6.3.1). A worksheet for performing calculations for standard culvert design is provided in Figure 6-2. Additional worksheets for tapered and mitered inlet design are provided in HDS-5 (USDOT, FHWA, 1985).

For standard culvert design, the following inlet and outlet control charts, found in HDS-5 and duplicated as the figures shown, are required culvert capacity calculations:

Standard Culvert Type	Figure Numbers Inlet Control	 Outlet Control
Circular Concrete Pipe	6-3	6-4
Circular Corrugated Metal Pipe (CMP)	6-5	6-6
Structural Plate CMP	6-5	6-7
Concrete Box	6-8	6-9
Oval Concrete Pipe – Long Axis Horizontal	6-10	6-11
Oval Concrete Pipe – Long Axis Vertical	6-12	6-11
CMP Arch	6-13	6-14
Structural Plate CMP Arch (18-inch Corner Radius)	6-13	6-15
Circular Pipe with Beveled Ring	6-16	6-4 or 6-6
Concrete Pipe Arch	6-17	6-18

6.3.1 GENERAL PROCEDURE

The following procedure, illustrated in Figure 6-1, shall be used to select a culvert size with the charts from HDS-5:

1. Perform hydrologic calculations (see Chapter 4).

2. List the following design data (a suggested tabulation form is provided in Figure 6-2, which includes a drawing labeled with design variables):

    a. Design discharge, Q, in cfs, with average return period (e.g., Q_{25}). When more than one barrel is used, show Q divided by the number of barrels.

    b. Approximate length, L, of culvert, in feet.

    c. Slope of culvert (if grade is given in percent, convert to slope in feet/ foot).

    d. Allowable headwater depth, AHW, in feet; i.e., the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert (see Section 6.2.2).

    e. Mean and maximum flood velocities in natural stream (optional).

f. Type of culvert, including barrel material, barrel cross-sectional shape, and inlet configuration.

3. Determine a trial culvert size by choosing one of the following options:

   a. Arbitrary selection.

   b. An approximating equation such as:

$$A = \frac{Q}{V} \qquad (6\text{-}1)$$

   where:

   A = Culvert area, in square feet

   Q = Design discharge, in cfs

   v = Average velocity, in feet/second

   c. Inlet control nomographs for the culvert type selected (see appropriate figures in this chapter). A trial size is determined by assuming HW/D. e.g., HW/D = 1.5, and using the given Q.

   If the trial size selected is larger than available standard culvert sizes or its use is prohibited by other physical limitations (such as limited embankment height), multiple culverts may be used by dividing the discharge equally between the number of barrels. It is also possible to consider raising the embankment height or using pipe arch and box culverts with width greater than twice the height.

4. Find inlet and outlet control headwater, HW, depths for the trial culvert size as follows:

   a. For inlet control, perform the following calculations (see Section 6.3.2 for additional details):

      (1) Use an appropriate inlet control chart from this chapter or HDS-5 and the trial size from Step 3 to find HW. Tailwater, TW, conditions are neglected in this determination. HW is found by multiplying HW/D, obtained from the nomographs, by the height of culvert D.

      (2) If HW is greater or less than allowable, try another trial size until HW is acceptable for inlet control, before computing HW for outlet control.

   b. For outlet control, perform the following calculations (see Section 6.3.3 for additional details):

(1) Approximate the depth of TW, in feet, above the invert at the outlet for the design flood condition in the outlet channel.

(2) If the TW elevation determined above is equal to or greater than the top of the culvert at the outlet, set design tailwater, DTW, equal to TW and find HW, using the following equation and the appropriate outlet control nomograph from this chapter or HDS-5:

$$HW = H + DTW - LS_o \tag{6-2}$$

where:

HW = Headwater depth for outlet control, in feet

H = Total head loss, obtained from the appropriate outlet Control nomograph from this chapter or HDS-5, in feet

DTW = Design tailwater, in feet

L = Barrel length, in feet

S_o = Barrel slope, in feet/foot

(3) If the TW elevation determined above is less than the top of the culvert at the outlet, find HW using Equation 6-2, except that DTW is the greater of the following two parameters:

$$h_o = \frac{d_c + D}{2} \tag{6-3}$$

or

TW

where:

h_o = Equivalent hydraulic elevation at outlet, in feet

dc = Critical depth, in feet (from HDS-5 charts, duplicated as figures listed below). Note d_c cannot exceed D.

D = Height of culvert opening, in feet

TW = Downstream tailwater elevation, in feet

Culvert Type	Figure
Rectangular	6-19
Circular	6-20
Oval – Long Axis Vertical	6-21
Oval – Long Axis Horizontal	6-22
Standard CMP Arch	6-23
Structural Plate CMP Arch	6-24
Concrete Pipe Arch	6-25
	6-26

Note: Headwater depth determined for this condition becomes increasingly less accurate as the headwater computed by this method falls below the value:

$$HW \leq D + (1 + k_e) \frac{v^2}{2g} \tag{6-4}$$

where:

D = Height of culvert opening, in feet

k_e = Entrance loss coefficient

v = Average velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

5. Compare the headwater values from Step 4a (inlet control) and Step 4b (outlet control). The higher headwater governs and indicates the type of flow control existing under the given conditions for the trial size and inlet configuration selected.

6. If outlet control governs and the HW is higher than the acceptable AHW, select a larger trial size and find HW as instructed under Step 4b. (Inlet control does not need to be checked, because the smaller size should be satisfactory for his control as determined under Step 4a.)

7. If desired, select an alternate culvert type or shape and determine size and HW by the above procedure.

8. If the culvert operates under inlet control, design a tapered inlet following Procedures in HDS-5, if an improved inlet is desirable.

9. If roadway overtopping occurs, calculate capacity following procedures in Section 6.3.4.

10. If storage routing is considered important, follow procedures in Chapter 9.

11. Compute outlet velocities for the culvert size and types to be considered in selection:

    a. If outlet control governs in Step 5, calculate the outlet velocity using Equation 6-1. If d_c or TW is less than the height of the culvert barrel, use the cross-sectional area corresponding to d_c or TW depth, whichever gives the greater area of flow. The total cross-sectional area, A, of the culvert barrel should not be exceeded.

    b. If inlet control governs in Step 5, the outlet velocity can be assumed to equal the mean velocity for open channel flow conditions in the barrel, computed by Manning's Equation (see Chapter 5) for the rate of flow, barrel size, roughness, and slope of culvert selected.

12. Determine to what extent channel protection is required downstream of the outlet (see Chapter 5). Properly sized riprap is required as a minimum.

13. Record final selection of culvert with size, type, required headwater, outlet Velocity, channel protection, and economic justification.

6.3.2 INLET CONTROL

Inlet control charts are presented in this manual for the following standard culvert types:

Standard Culvert Type	Figure Numbers for Inlet Control Charts
Circular Concrete Pipe	6-3
Circular CMO	6-5
Concrete Box	6-8
Oval Concrete Pipe – Long Axis Horizontal	6-10
Oval Concrete Pipe – Long Axis Vertical	6-12
CMP Arch	6-13
Structural Plate CMP Arch (18-inch Corner Radius)	6-13
Circular Pipe with Beveled Ring	6-16
Concrete Pipe Arch	6-17

Metal pipes are not allowed unless approved by the Director of City Engineering.

The following three types of calculations can be performed using the inlet control charts:

1. To determine the headwater, HW, given Q and size for selected culvert type and inlet configuration:

   a. Use a straightedge to connect the culvert diameter or height, D, scale and the discharge, Q, scale, or Q/B for box culverts. Note the point of intersection of the straightedge on the HW/D scale marked (1).

   b. If the HW/D scale marked (1) represents the inlet configuration used, read HW/D on this scale. When either of the other two inlet configurations listed on the nomograph is used, extend the point of intersection obtained in Step 1b horizontally to scale (2) or (3) and read HW/D.

   c. Compute HW by multiplying HW/D by D.

Note: The approach velocity is assumed to be zero by this procedure. If the approach velocity is considered significant, the HW can be decreased by subtracting the velocity head.

2. To determine Q per barrel, given HW and size for selected culvert type and inlet configuration:

   a. Compute HW/D for given conditions.

   b. Locate HW/D on scale for appropriate inlet configuration. If scale (2) or (3) is used, extend the HW/D point horizontally to scale (1).

   c. Use a straightedge to connect the point on HW/D scale (1) obtained above with culvert size on the far-left scale. Read Q or A/B at the intersection of this line with the middle discharge scale.

   d. If Q/B is read in Step 2c, multiply by B (span of box culvert) to find Q.

3. To determine culvert size, given Q, AHW, and type of culvert with desired inlet configuration:

   a. Using a trial size, compute HW/D.

   b. Locate HW/D on the scale for the appropriate inlet configuration. If scale (2) or (3) is used, extend the HW/D point horizontally to scale (1).

   c. Use a straightedge to connect the point on HW/D scale (1) obtained above with the given discharge on the middle scale. Read diameter, height, or size of culvert required at the intersection of this line with the culvert size scale on the far left.

   d. If D is not as originally assumed, repeat procedure with a new D.

6.3.3 OUTLET CONTROL

Outlet control charts are presented in this manual for the following standard culvert types:

Figure Numbers For Outlet Control

Standard Culvert Type	Charts
Circular Concrete Pipe	6-4
Circular CMO	6-6
Structural Plate CMP	6-7
Concrete Box	6-9
Oval Concrete Pipe – Long Axis Horizontal or Vertical	6-11
CMP Arch	6-14
Structural Plate CMP Arch (18 – inch Corner Radius)	6-15
Concrete Pipe Arch	6-18

All culverts shall be constructed using reinforced concrete pipe unless alternate materials are approved by the Director of City Engineering.

The following steps outline the use of the outlet control charts:

1. To determine H for a given culvert and Q:

    a. Locate the appropriate nomograph for the type of culvert selected. Find the entrance loss coefficient, k_e, for the inlet configuration using data from Table 6-1.

    b. Begin nomograph solution by locating the proper starting point on the length scale:

        (1) If the n value of the nomograph corresponds to that of the culvert being used, select the proper length curve for an assigned k_e value and locate the starting point at the given culvert length. If a curve is not shown for the selected k_e, see (e) below. If the n value for the culvert selected differs from that of the nomograph chart, see (3) below.

        (2) For the n value of the nomograph and k_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two scales in proportion to k_e values.

(3) For a different roughness coefficient, n_1, than that of the chart n, use the length scales shown with an adjusted length, L_1, calculated as:

$$L_1 = \frac{L\, n_1^2}{n} \tag{6-5}$$

(See Step 2 for n values)

c. Use a straightedge to connect the point on the length scale to size of the culvert barrel and mark the point of crossing on the turning line. See Step 3 for size considerations for a rectangular box culvert.

d. Pivot the straightedge on this point on the turning line and connect with the given discharge rate. For multiple barrels, divided Q by the number of barrels, divide Q by the number of barrels before using the nomograph. Read head in feet on the H scale located on the far right. For values beyond limit of the printed scales, find H by solving the equation:

$$H = \left(1 + k_e + \frac{29n^2L}{R^{1.33}}\right)\frac{v^2}{2g} \tag{6-6}$$

where:

H = Total head loss, or the elevation difference between HW and DTW, in feet (see Figure 6-2 for sketch

k_e = Entrance loss coefficient (see Table 6-1)

n = Manning's roughness coefficient (see appropriate culvert nomograph)

L = Barrel length, in feet

R = Hydraulic radius of the culvert, in feet

v = Average velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

2. Values of n, which are the basis for the nomographs, are presented on each nomograph.

3. To use the box culvert nomograph (Figure 6-9) for full flow for other than the

configurations shown:

a. Compute cross-sectional area of the rectangular box.

b. Use a straightedge to connect the proper point (see Step 1) on the length scale to the barrel area and mark the point on the turning line. Note that the area scale on the nomograph is calculated for barrel cross sections with span B twice the height D; its close correspondence with the area of square boxes ensure that it may be used for all sections intermediate between square and B = 2D or B = 1/2D. For other box proportions, use Equation 6-6 for more accurate results.

6.3.4 ROADWAY OVERTOPPING

The overall performance curve for roadway overtopping can be determined by performing the following steps:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These 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 (see Sections 6.3.2 and 6.3.3).

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 the following equations to calculate flow rates across the roadway:

$$Q_o = C_d L\ HW_r^{1.5} \qquad (6\text{-}7)$$

where:

Q_o = Overtopping flow rate, in cfs

$C_d = K_t\ C_r$ = Discharge coefficient

K_t = Submergence factor (from Figure 6-27, Part C)

C_r = Unsubmerged discharge coefficient (from Figure 6-27, Part A or B)

L = Length of roadway crest, in feet

HWr = Upstream depth, measured from the

Roadway crest to the water surface upstream
Of the weir drawdown, in feet

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

6.3.5 EXAMPLE PROBLEM

Example 6-1. Circular Concrete Culvert

Given the following data, design a concrete pipe culvert using appropriate nomographs:

Q_{25} = 160 cfs

Q_{100} = 220 cfs

L = 200 feet

S_0 = 0.01 feet/foot

AHW = 10 feet

25-year TW = 3.5 feet

100-year TW = 4.0 feet

Entrance type is a square edge with headwall, n = 0.012.

1. Select a trial culvert size assuming HW/D = 1.5; therefore:

   $D = \frac{HW}{1.5} = \frac{10}{1.5} = 6.67$ feet

   Try a 72-inch concrete pipe.

2. Find the actual headwater depth for the trial culvert size.

   a. For inlet control, using Figure 6-3, Q = 160 cfs, D = 72 inches, gives HW/D = 0.85.

   HW = (0.85) (6.0) = 5.1 feet

   Since 5.1 feet is considerably less than the AHW of 10 feet, try a 60-inch concrete pipe, which yields HW/D = 1.23 (from Figure 6-3).

   HW = (1.23) (5.0) = 6.15 feet

Q = 220 cfs, D = 60 inches, yielding HW/D = 1.70

HQ = (1.70) (5.0) = 8.5 feet

b. For outlet control, using Figures 6-4 and 6-20, TW = 3.5 and 4.0 is less than D = 5.0 feet.

From Table 6-1, k_e = 0.5, Q = 160 cfs, D = 60 inches, yielding d_c = 3.6 feet (Figure 6-20).

Since d_c is less than D = 5.0 feet, h_o is determined using Equation 6-3:

$$H_o = \frac{3.6+5.0}{2} = 4.3 \text{ feet}$$

From Figure 6-4, H = 2.2 feet. Using Equation 6-2:

HW = 2.2 + 4.3 – (0.01) (200) = 4.5 feet

Q = 220 cfs, D = 60 inches, givens d_c = 4.2 feet (Figure 6-20)

Since d_c is less than D = 5.0 feet, h_o is determined using Equation 6-3:

$$H_o = \frac{4.2+5.0}{2} = 4.6 \text{ feet}$$

From Figure 6-4, H = 4.2 feet. Using Equation 6-2:

HW = 4.2 + 4.6 – (0.01) (200) = 6.8 feet

c.

Flow	Headwater (feet)	
(cfs)	Inlet Control	Outlet Control
160	6.15	4.5
220	8.50	6.8

Since the headwater depths for inlet control are greatest under both Q_{25} and Q_{100} flow conditions, inlet control governs flow through the culvert.

3. A 54- inch pipe would be adequate for Q25, but since HW = 10.8 for Q100, it is not selected.

4. Compute the outlet velocity. Since the culvert will operate under inlet control, use Manning's Equation as expressed by Equation 5-5:

$$v = \frac{0.592}{(0.012)} (5.0)^{2/3} (0.01)^{1/2}$$

$$v = 14.4 \text{ feet/second}$$

Since this velocity exceeds permissible velocities for bare soil (see Table 5-2), erosion. Protection is required (see Chapter 10).

Table 6-1
CULVERT ENTRANCE LOSS COEFFICIENTS

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

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

[a]"End section conforming to fill slope," made of either metal or concrete, is the section commonly available from manufactures. From limited hydraulic tests, the sections are equivalent in operation to a headwall in both inlet and outlet control. End sections that incorporate a closed taper in their design have a superior hydraulic performance.

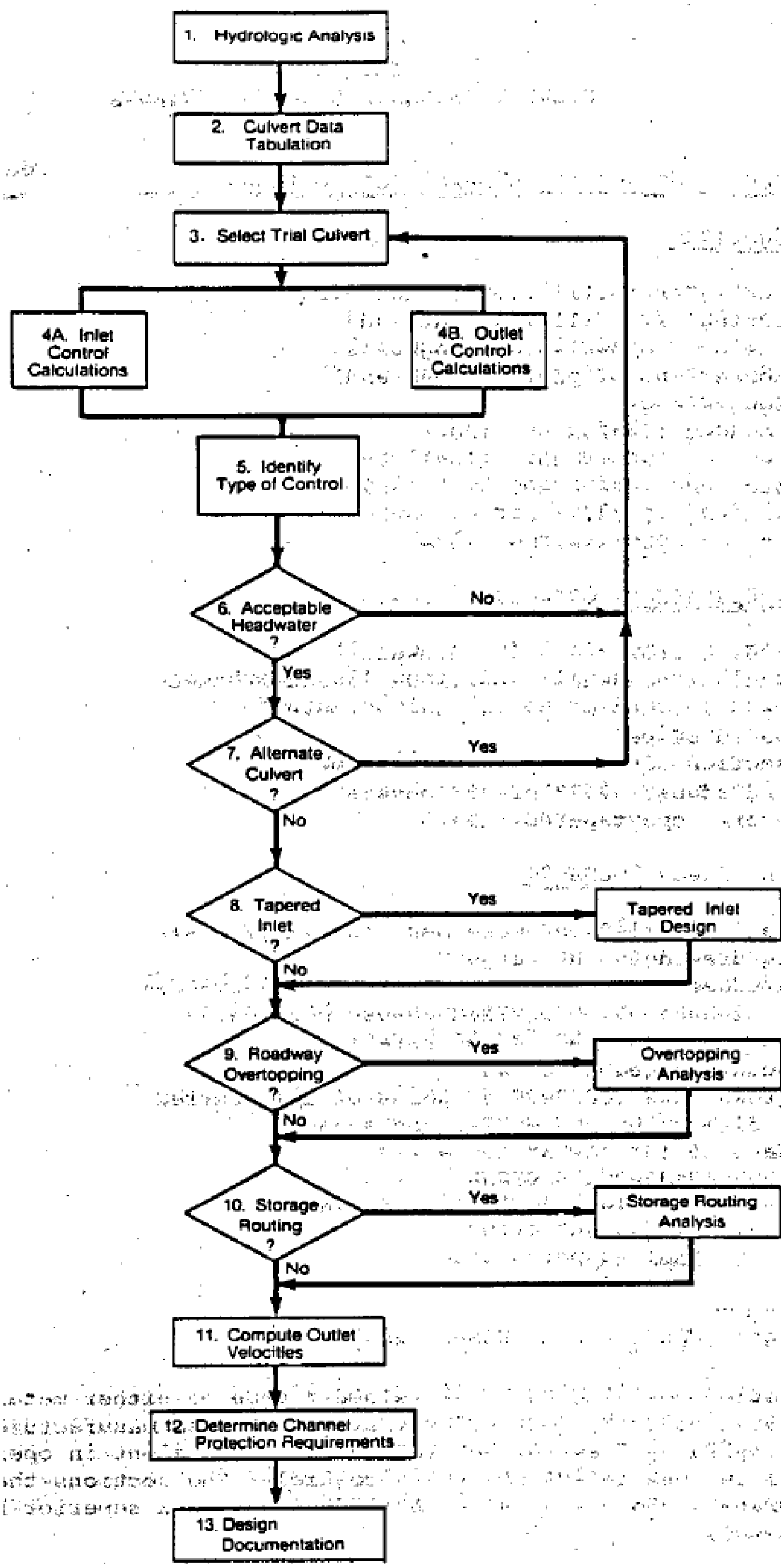


FIGURE 6-1
General Culvert Design Flow Chart

PROJECT :	STATION :	CULVERT DESIGN FORM
	SHEET ____ OF ____	DESIGNER / DATE : ____ / ____
		REVIEWER / DATE : ____ / ____

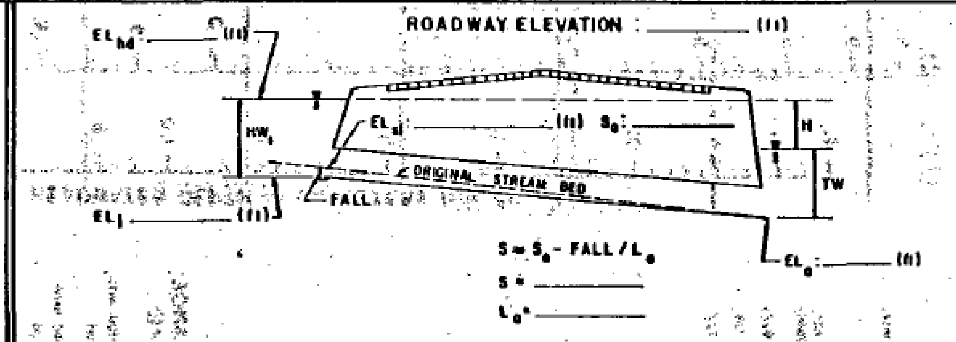
HYDROLOGICAL DATA

SEE ADD'L SHTS.

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

DESIGN FLOWS/TAILWATER

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



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

HEADWATER CALCULATIONS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

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

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

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

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

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

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

SUBSCRIPT DEFINITIONS:

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

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ____

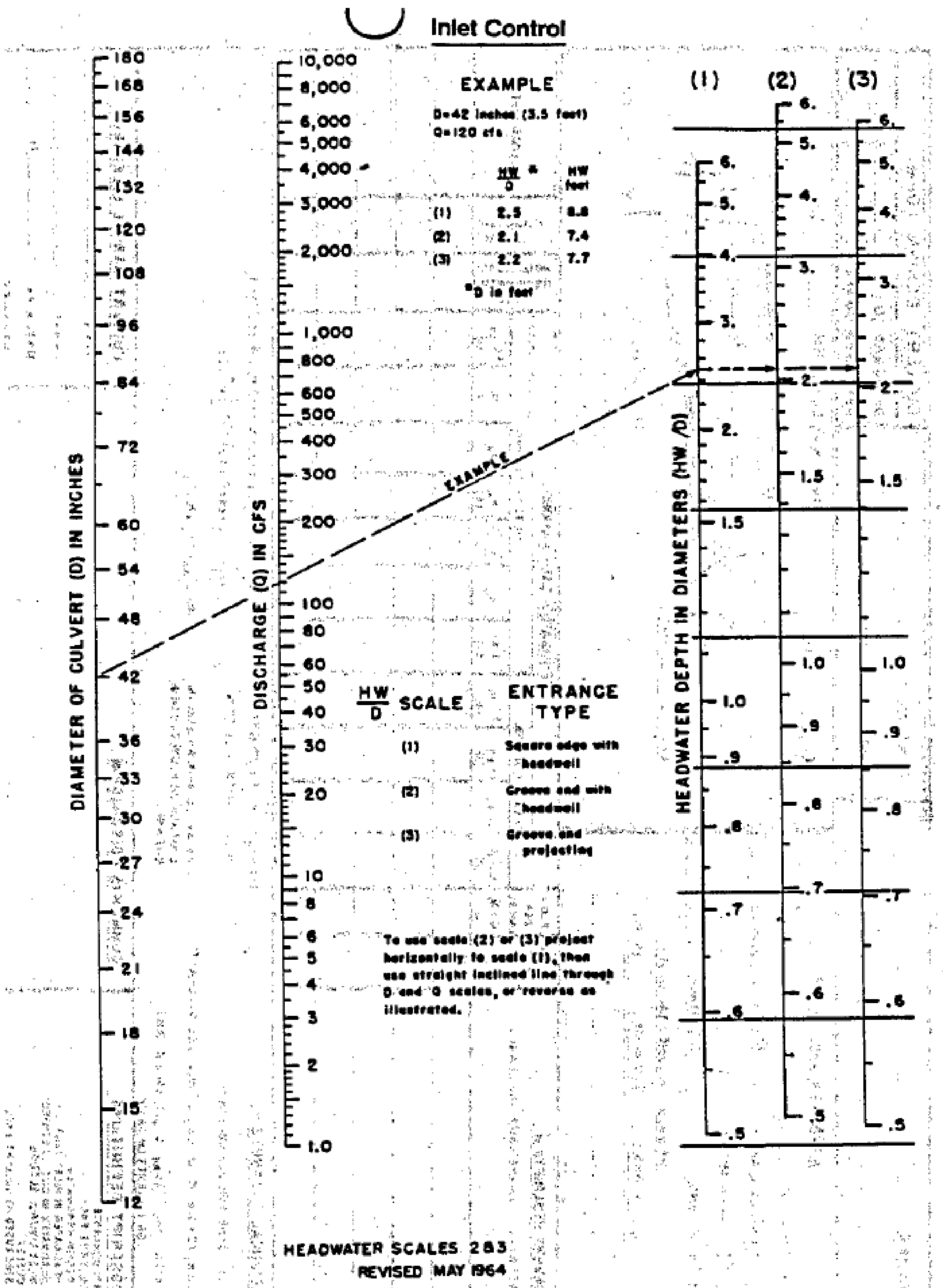
SHAPE: ____

MATERIAL: ____ n ____

ENTRANCE: ____

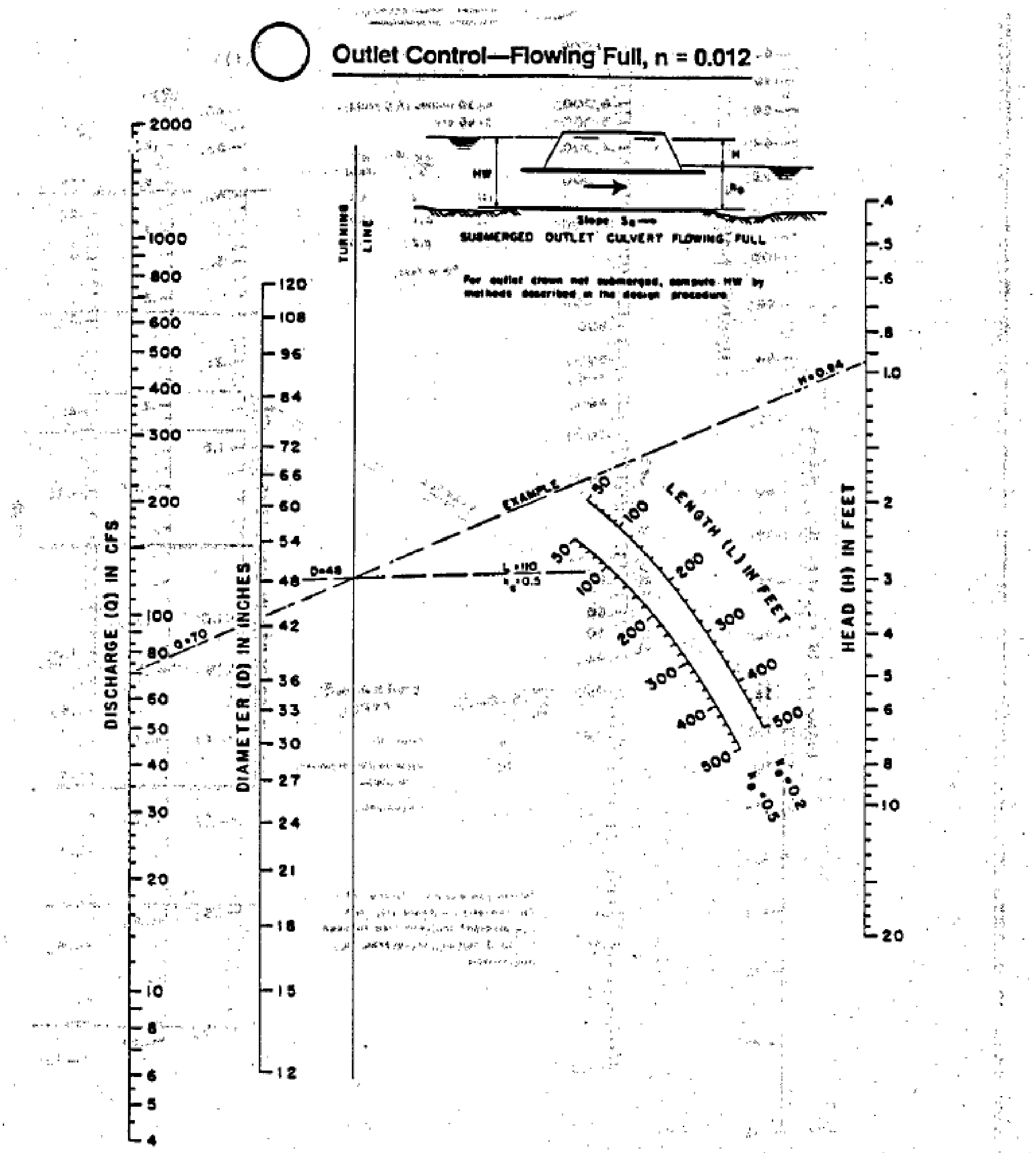
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-2
Culvert Design Form



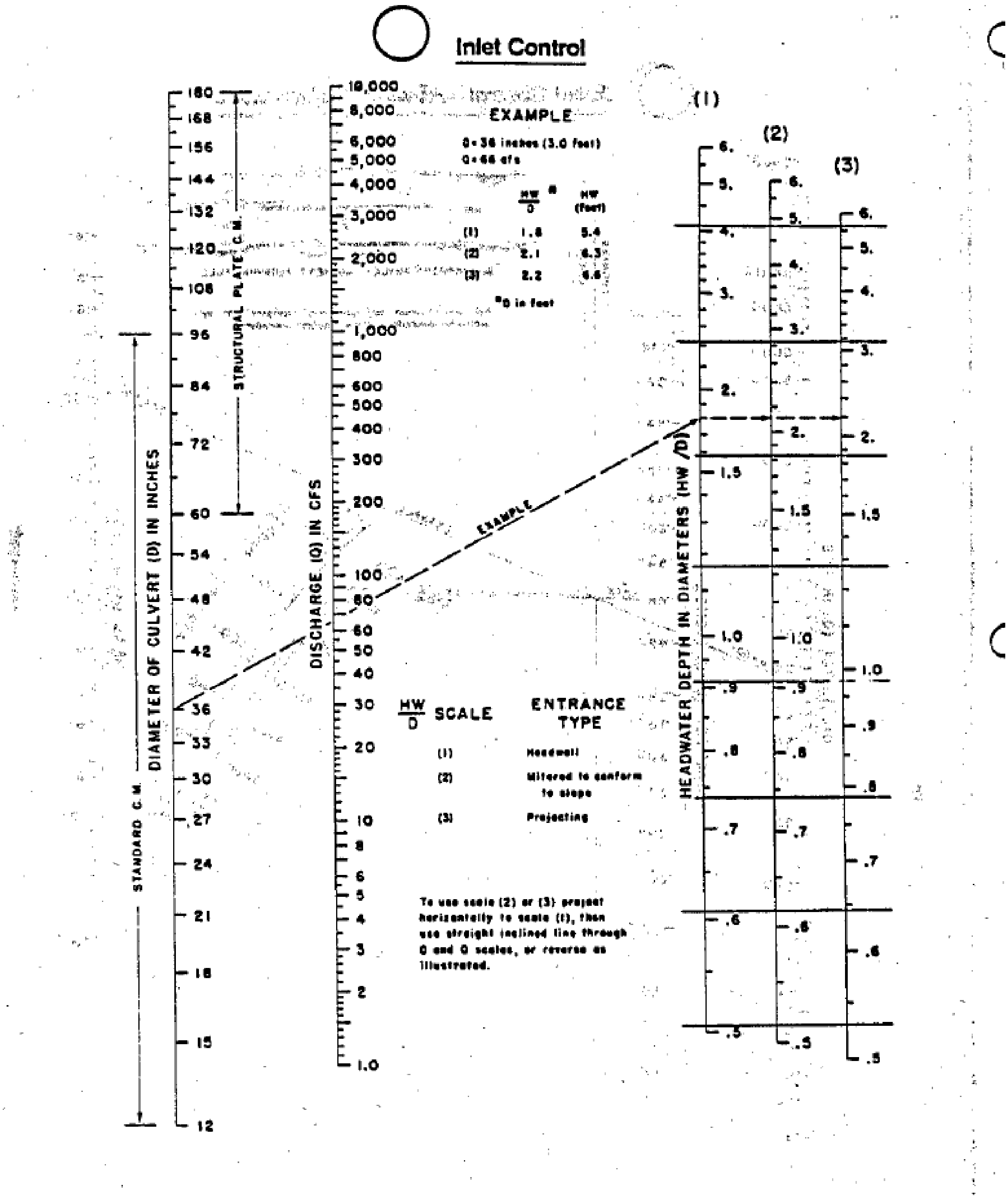
*Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-3
Inlet Control Chart for Circular Concrete Pipe Culverts



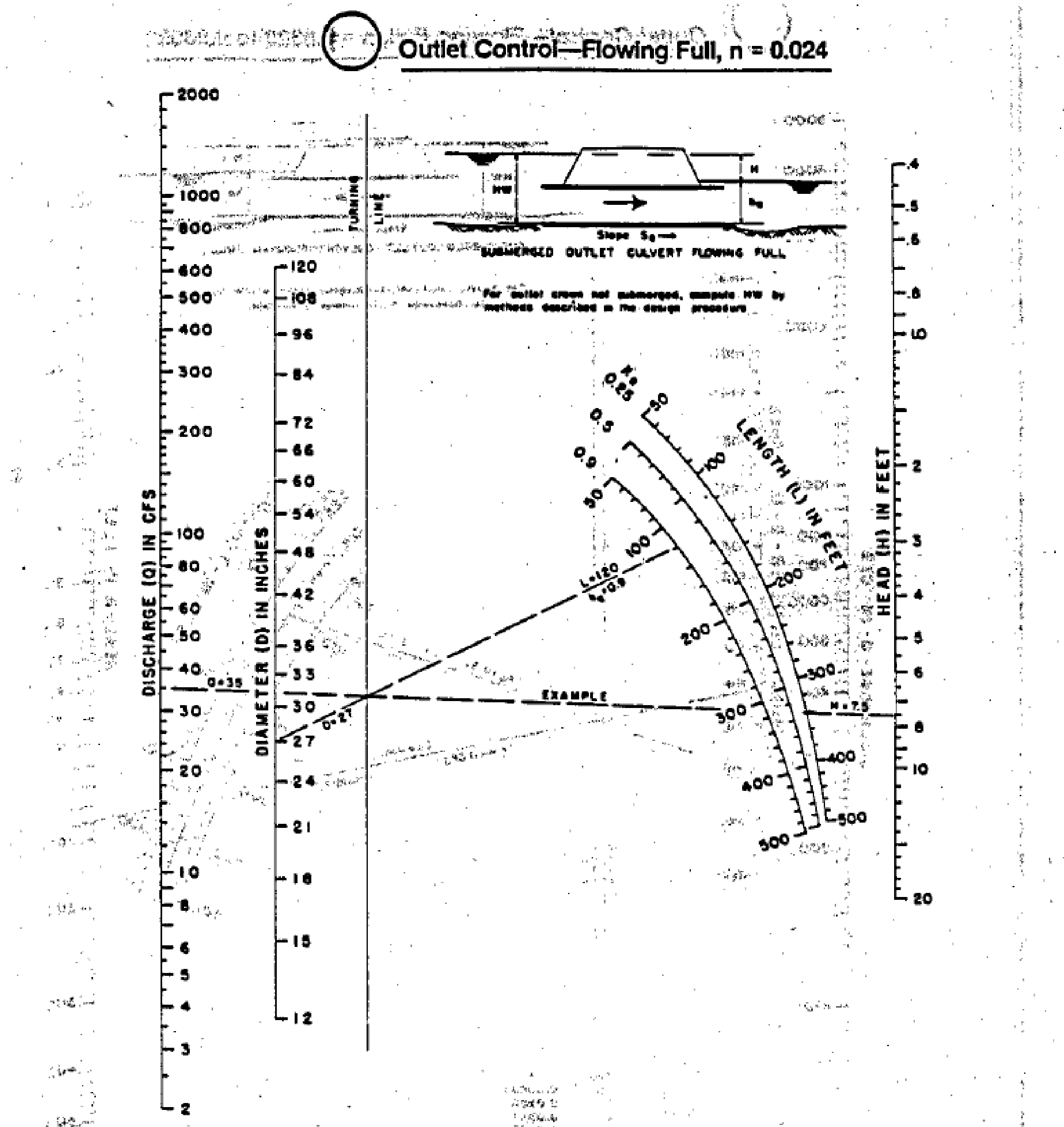
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-4
Outlet Control Chart for Circular Concrete Pipe Culverts Flowing Full



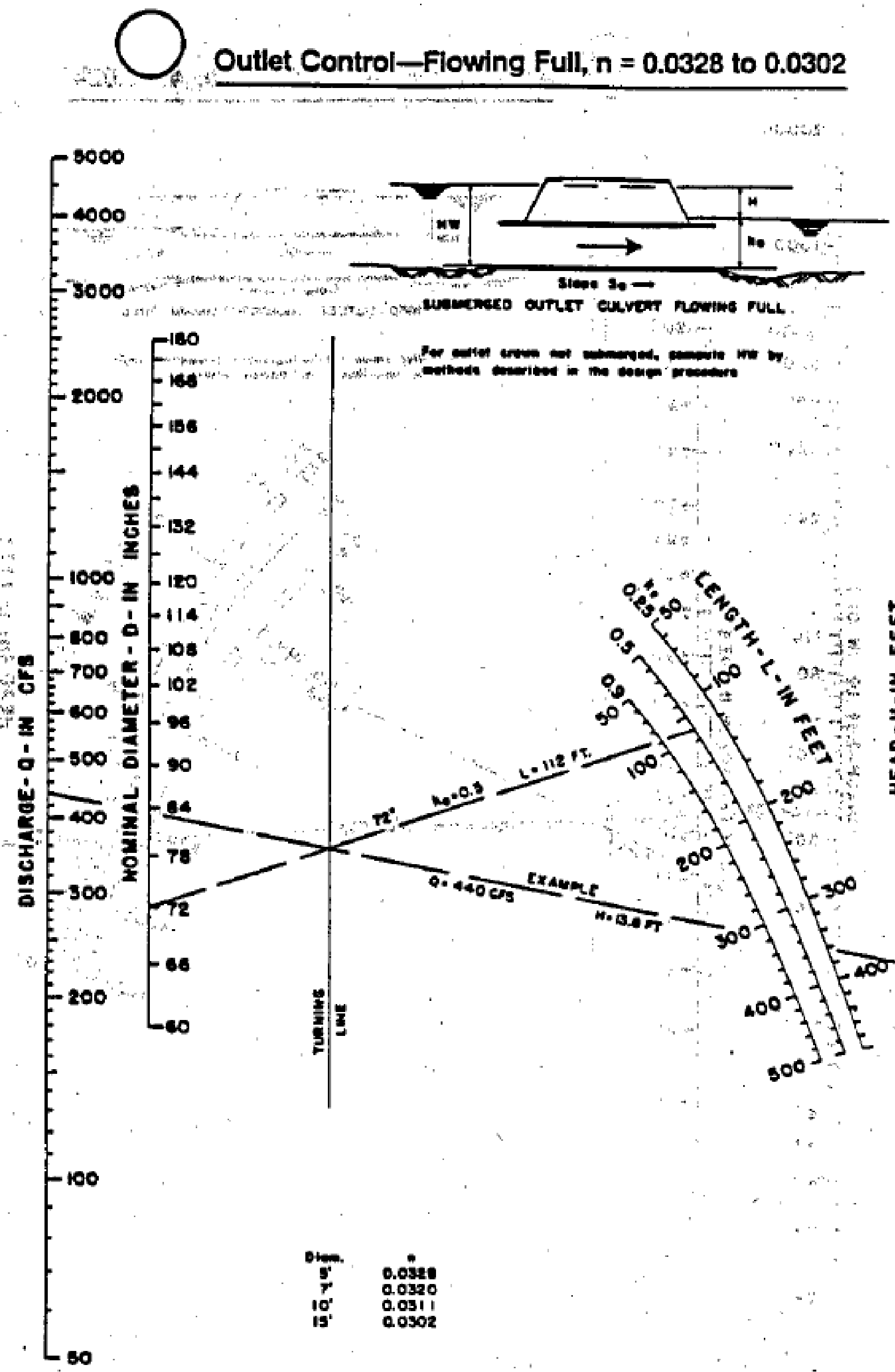
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-5
Inlet Control Chart for Circular CMP Culverts



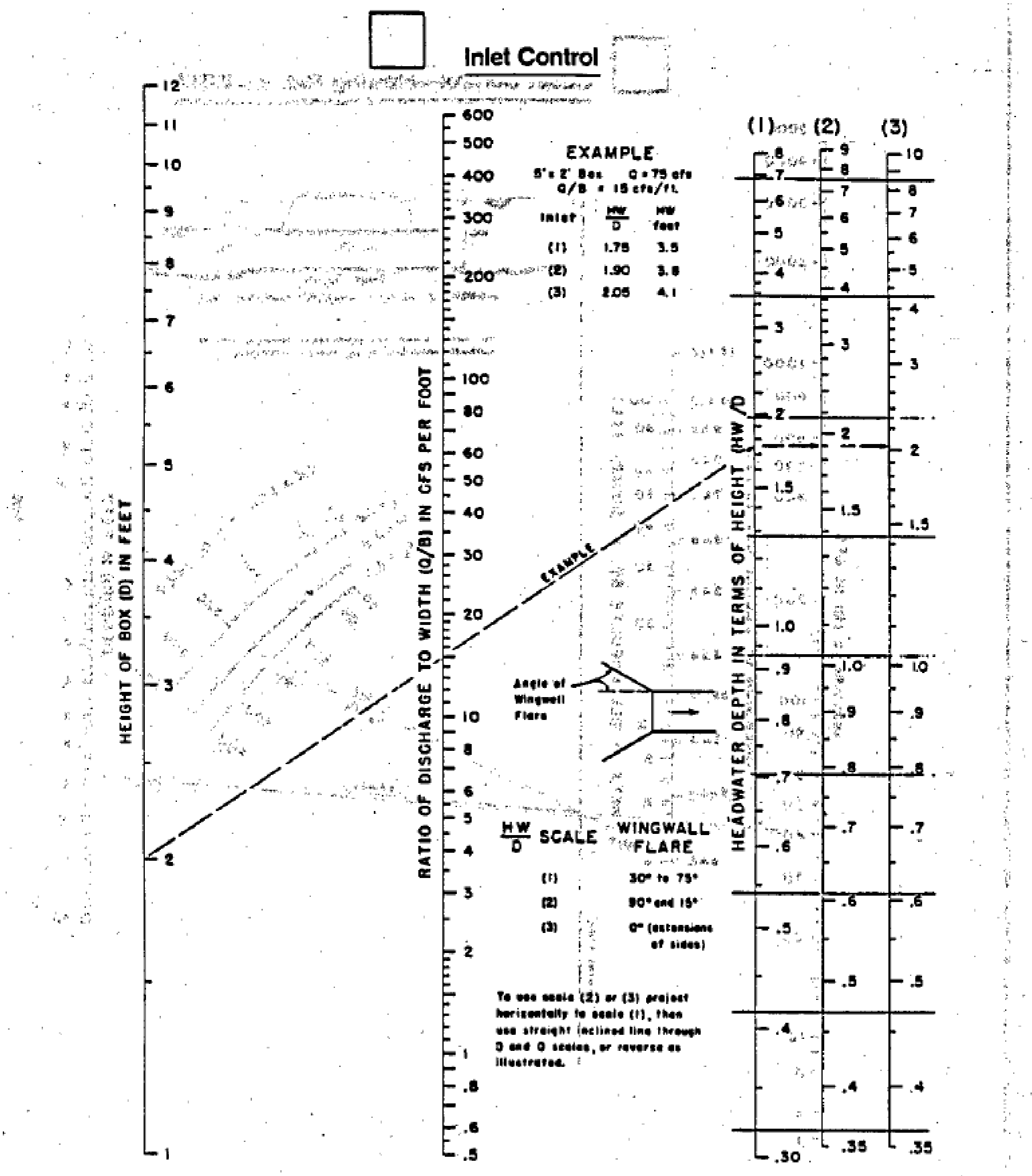
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-6
Outlet Control Chart for Circular CMP Culverts Flowing Full



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

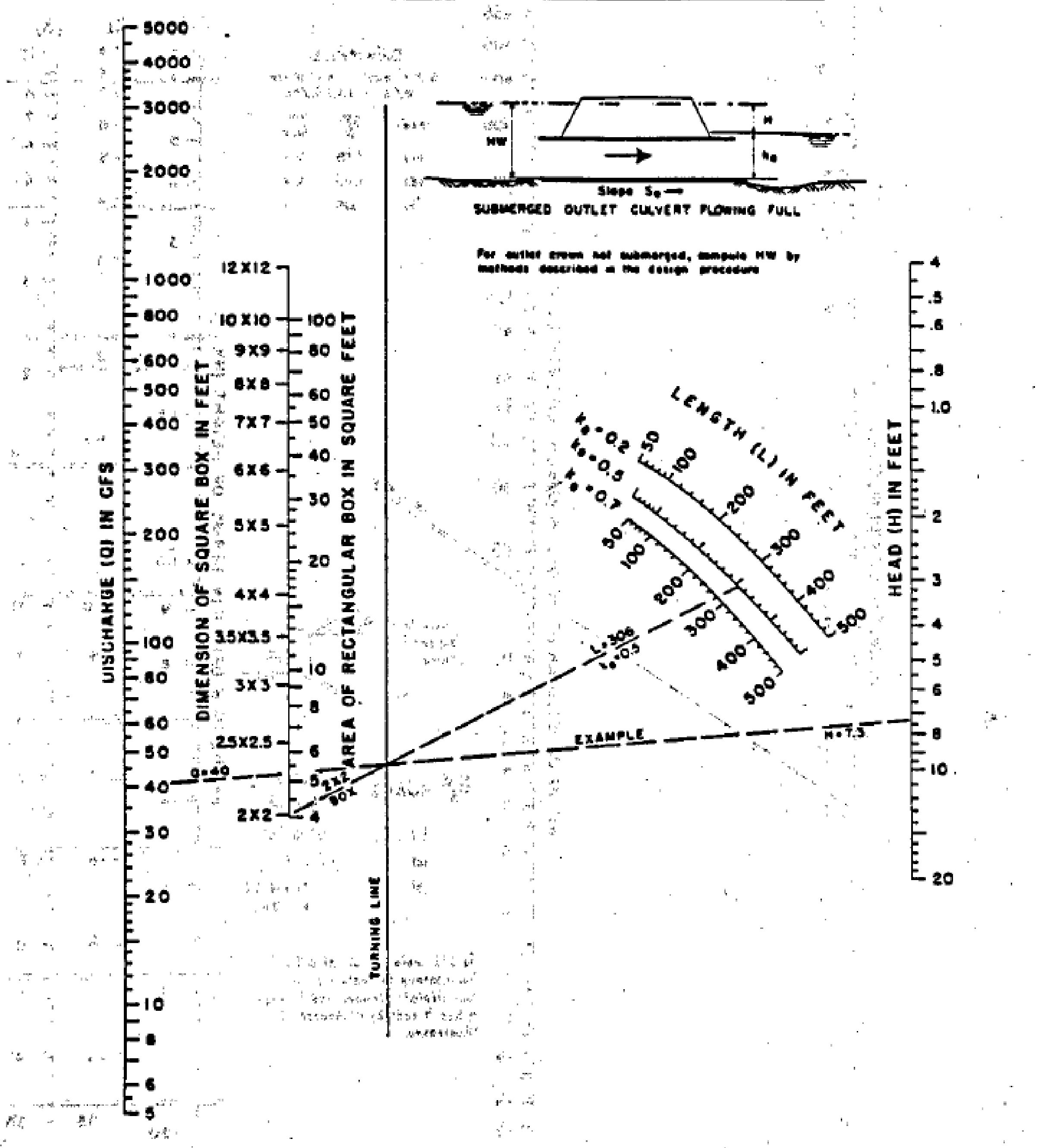
FIGURE 6-7
Outlet Control Chart for Structural Plate CMP Culverts Flowing Full



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

FIGURE 6-8
Inlet Control Chart for Concrete Box Culverts

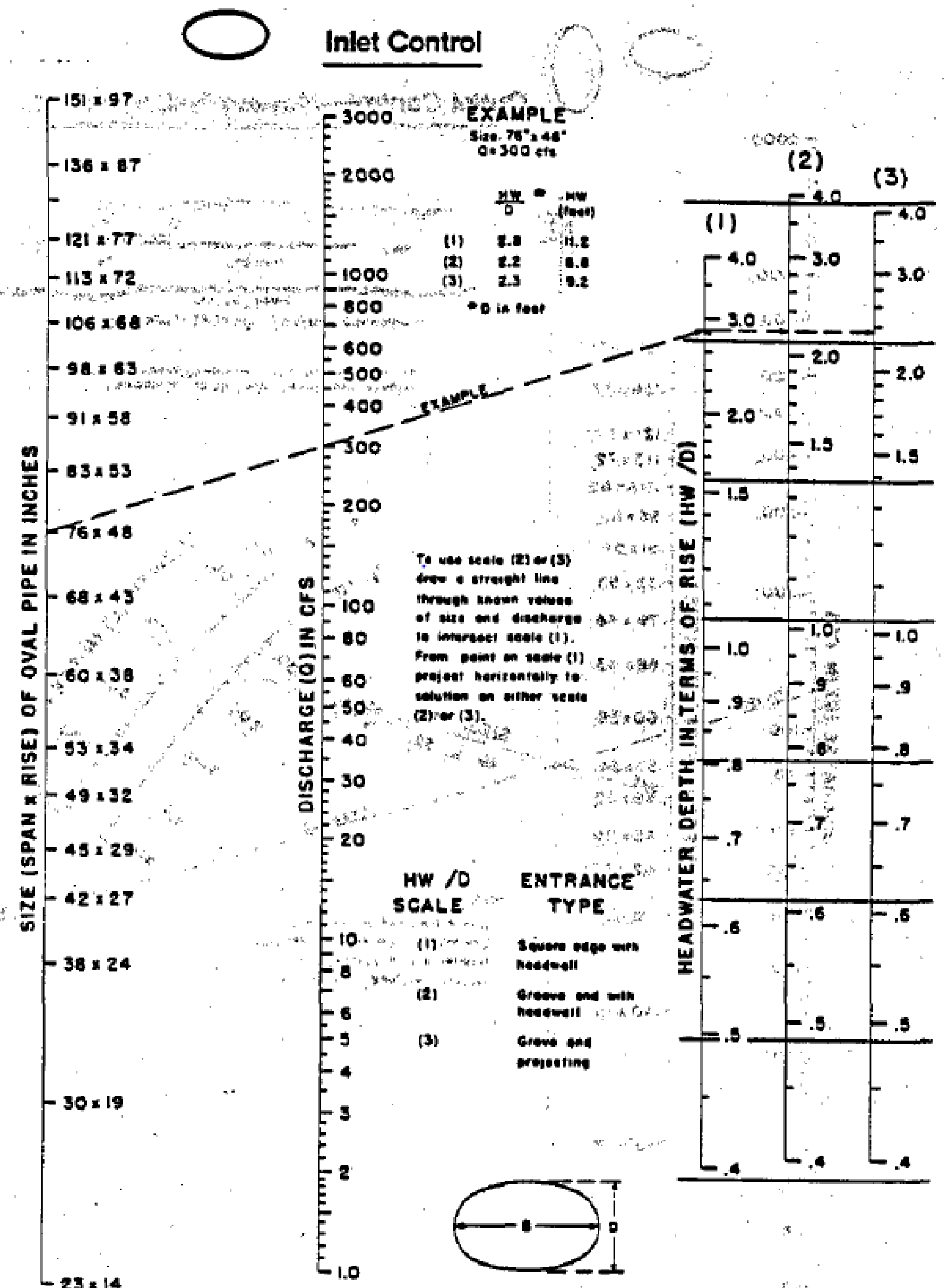
Outlet Control—Flowing Full, n = 0.012



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

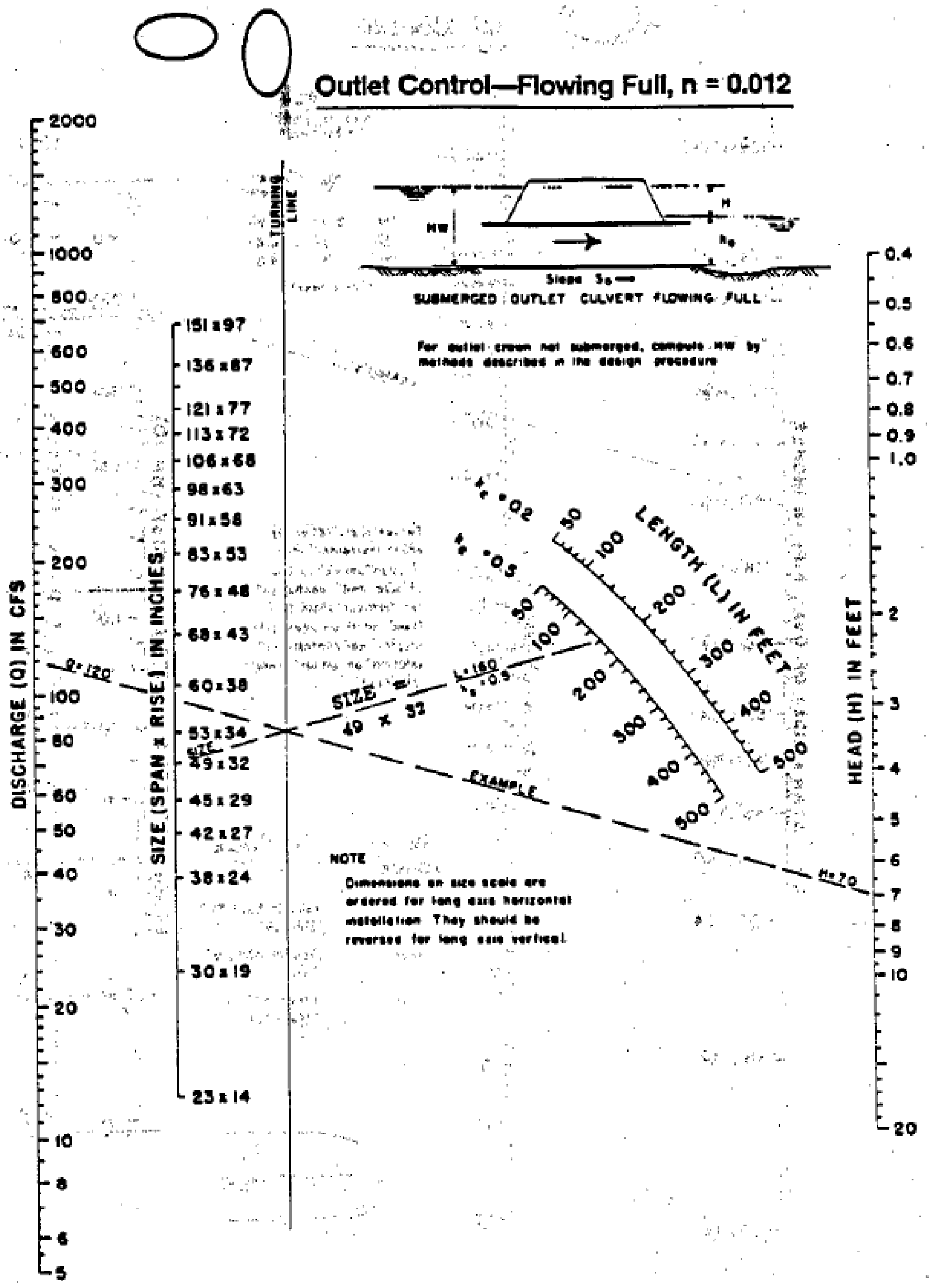
FIGURE 6-9

Outlet Control Chart for Concrete Box Culverts Flowing Full



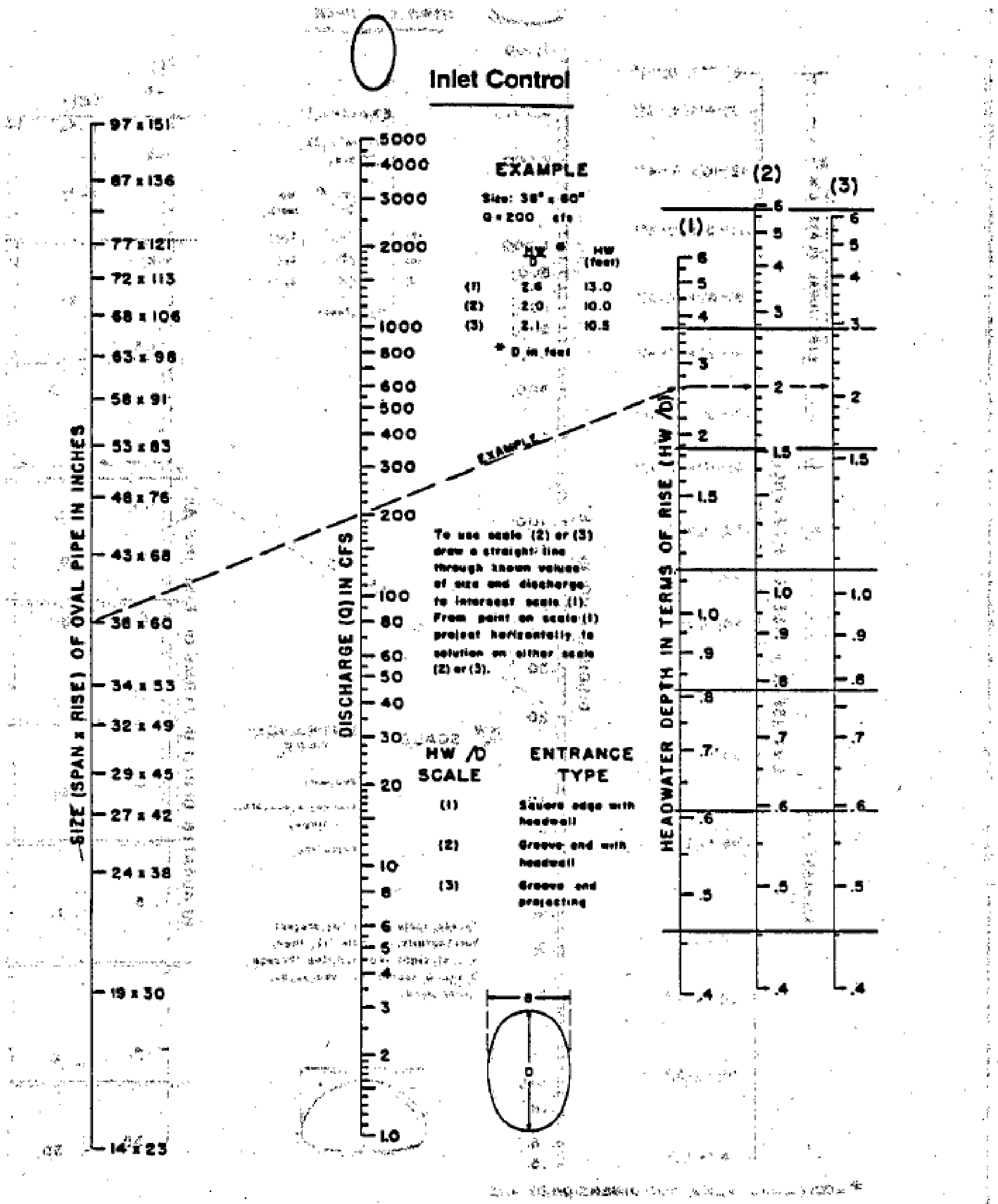
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-10
Inlet Control Chart for Oval Concrete Pipe Culverts—Long Axis Horizontal



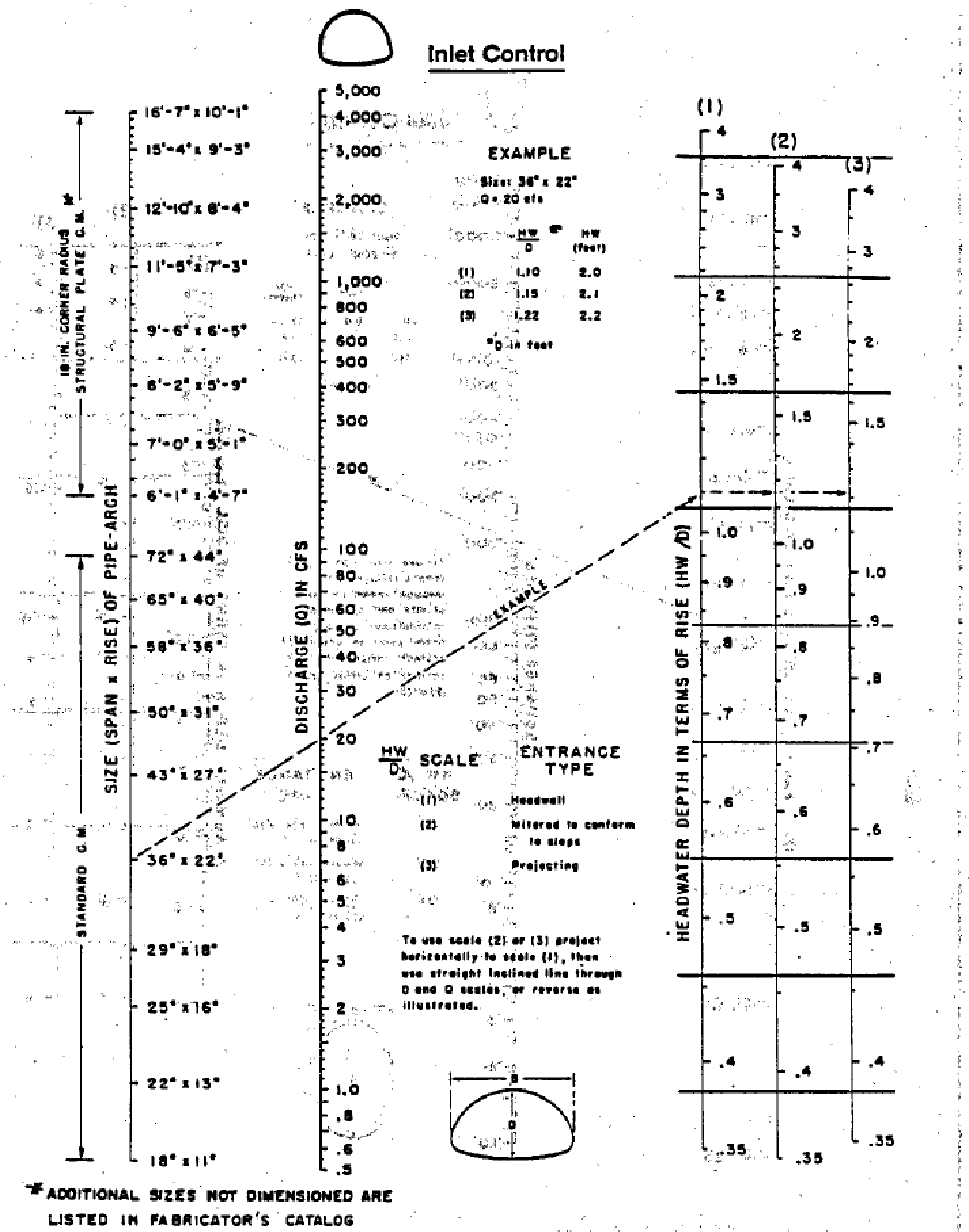
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-11
Outlet Control Chart for Oval Concrete Pipe Culverts
Flowing Full—Long Axis Horizontal or Vertical



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

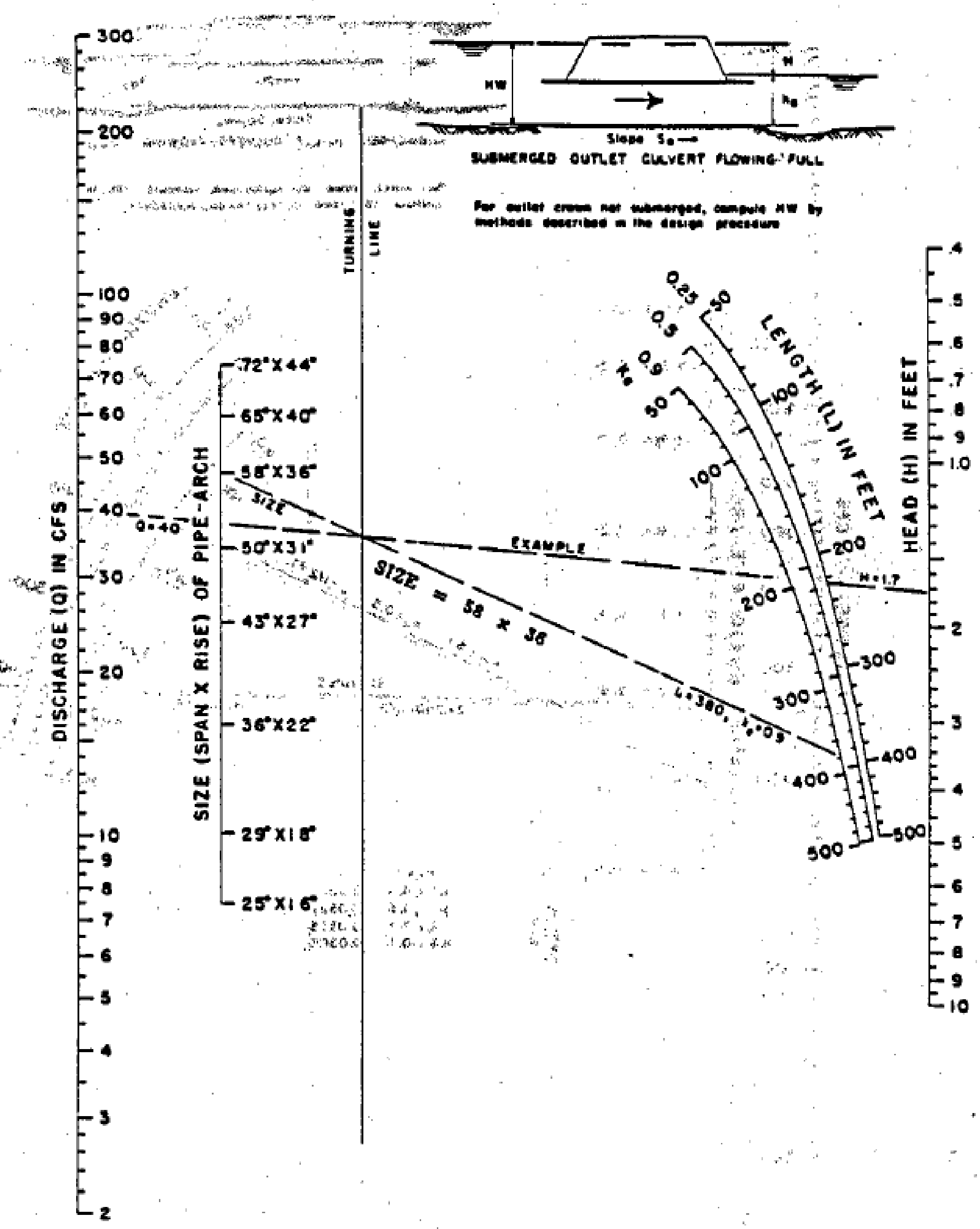
FIGURE 6-12
Inlet Control Chart for Oval Concrete Pipe Culverts—Long Axis Vertical



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

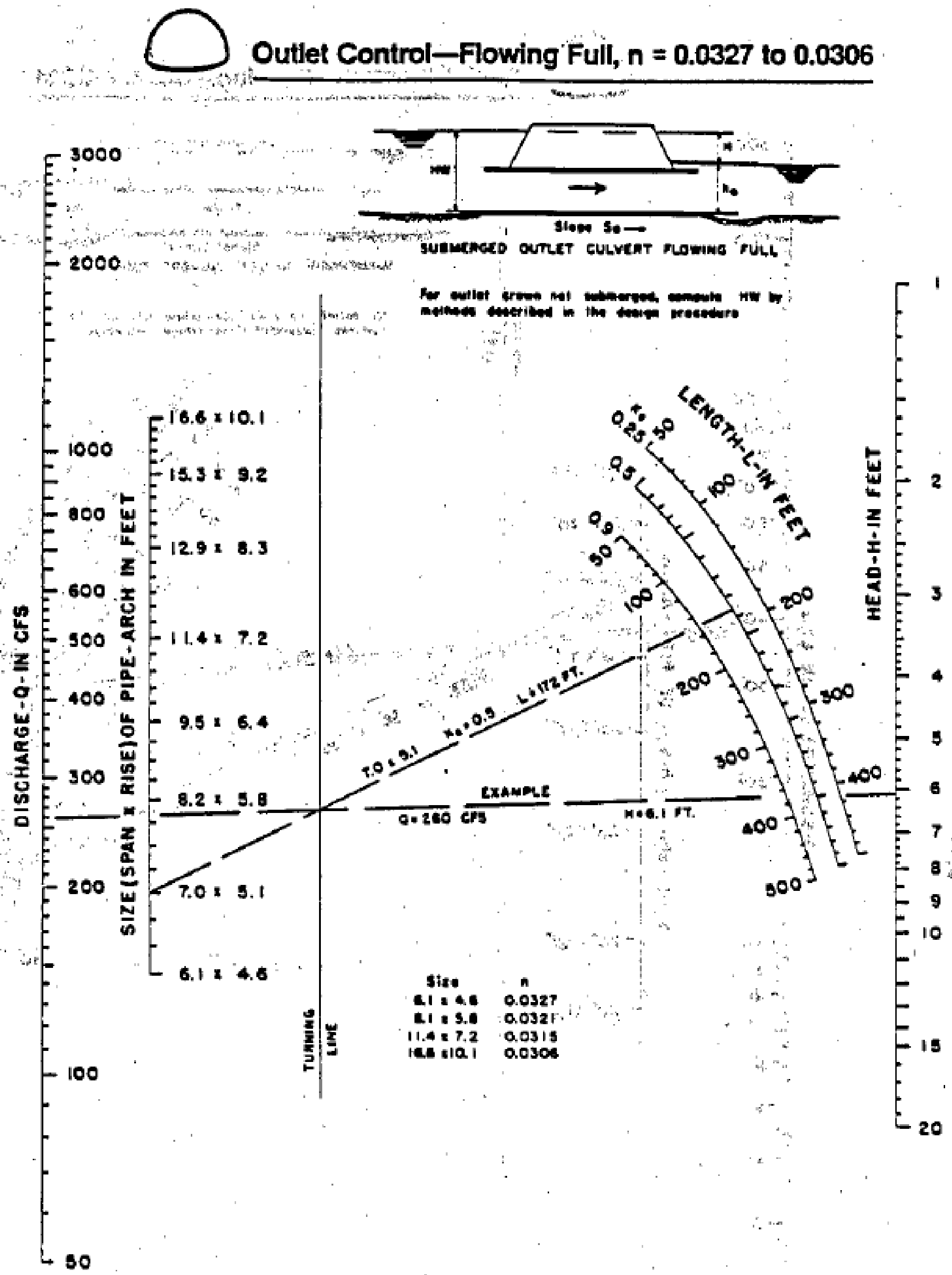
FIGURE 6-13
Inlet Control Chart for CMP Arch Culverts

Outlet Control—Flowing Full, n = 0.024



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

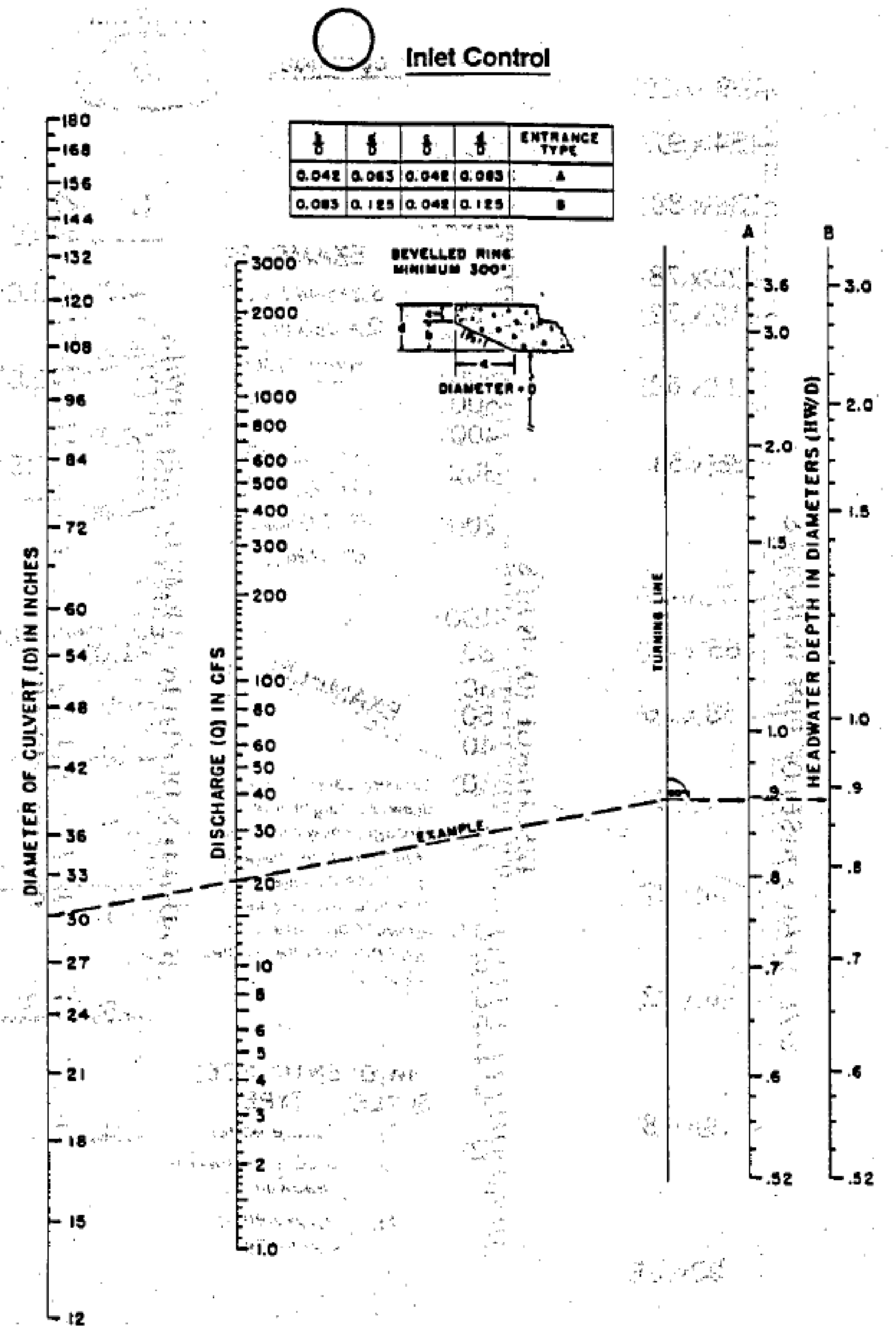
FIGURE 6-14
Outlet Control Chart for CMP Arch Culverts Flowing Full



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

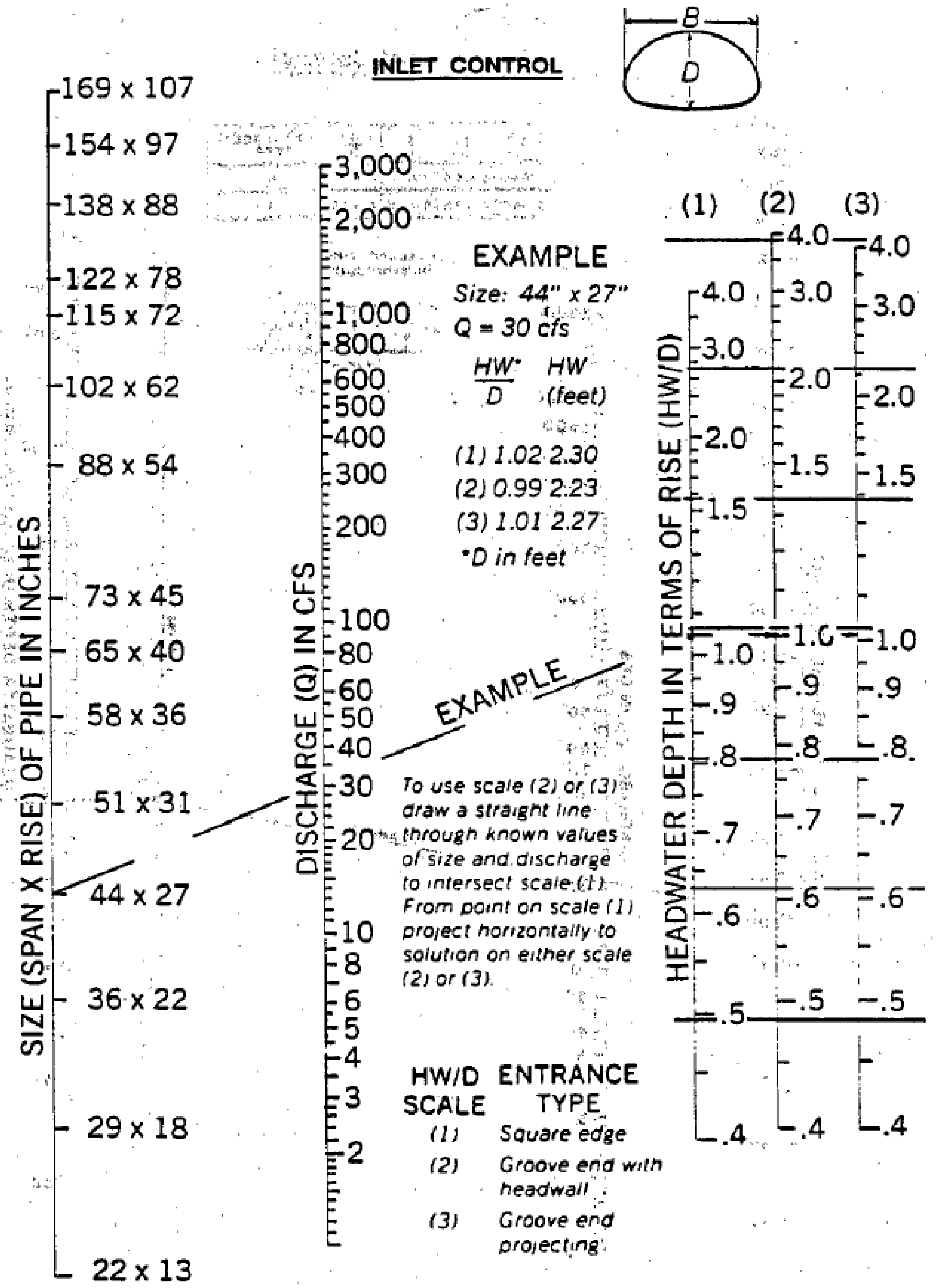
FIGURE 6-15

Outlet Control Chart for Structural Plate CMP Arch Culverts (18-inch Corner Radius) Flowing Full



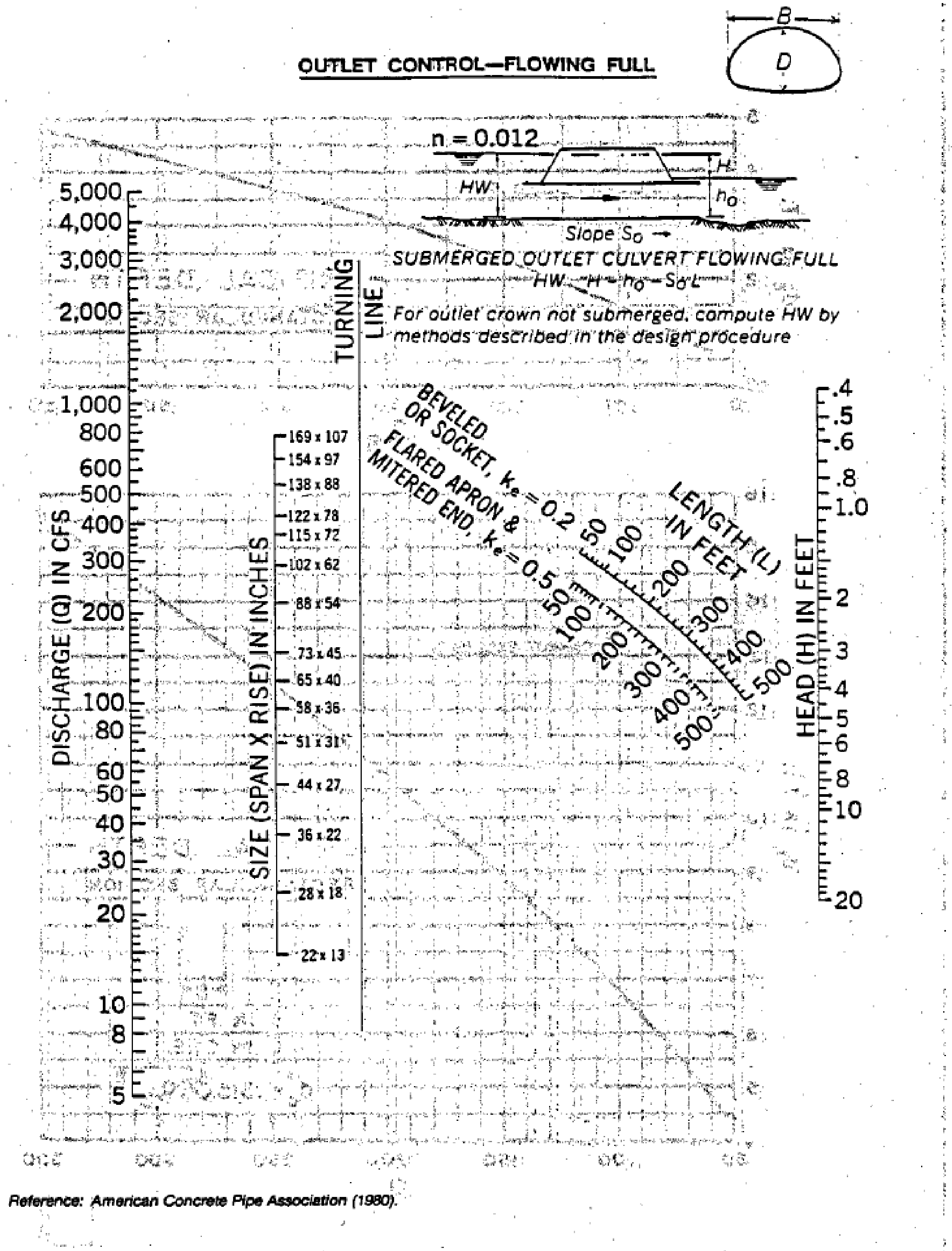
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-16
Inlet Control Chart for Circular Pipe Culverts with Beveled Ring



Reference: American Concrete Pipe Association (1980).

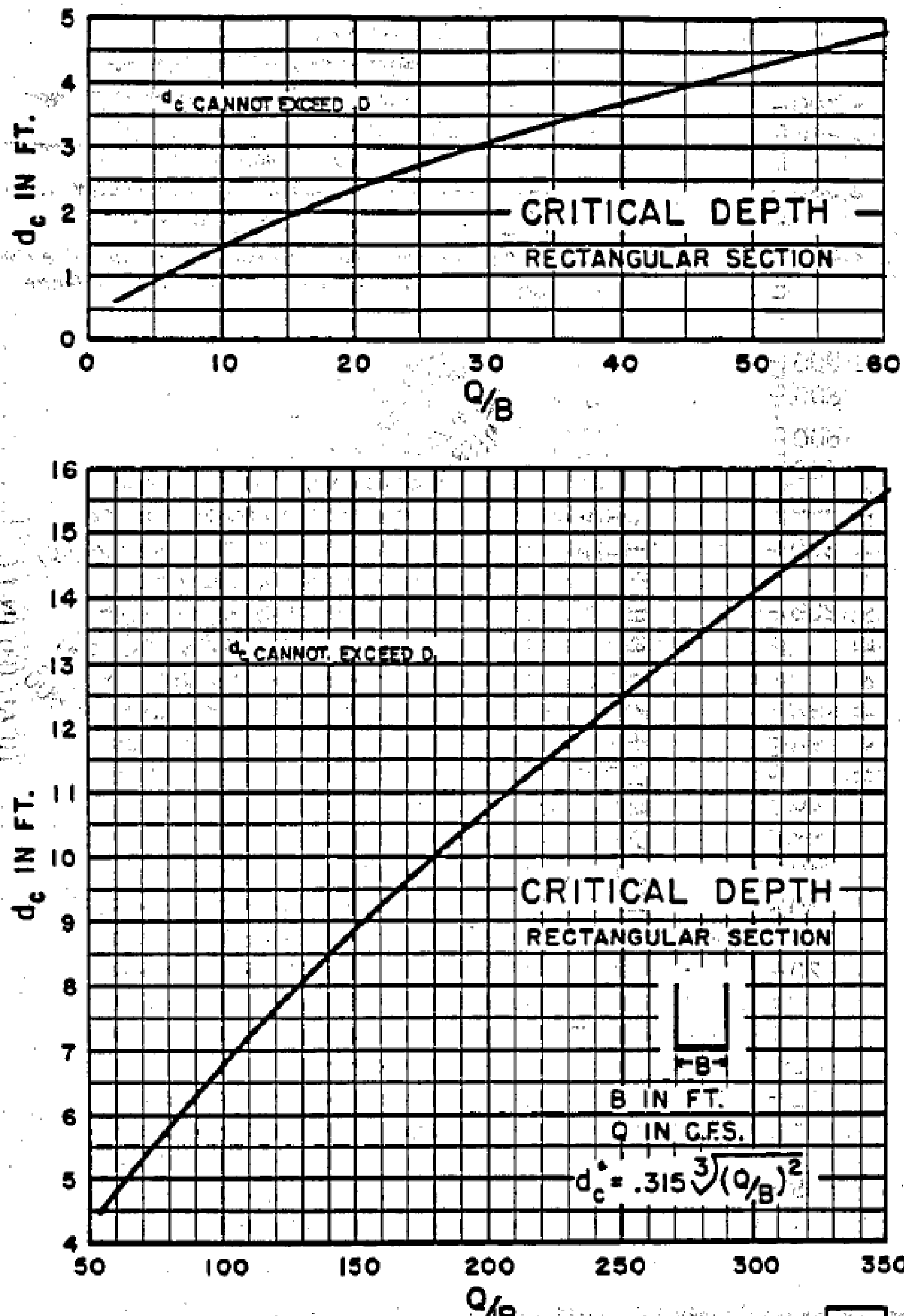
FIGURE 6-17
Inlet Control Chart for Concrete Pipe Arch Culverts



Reference: American Concrete Pipe Association (1980).

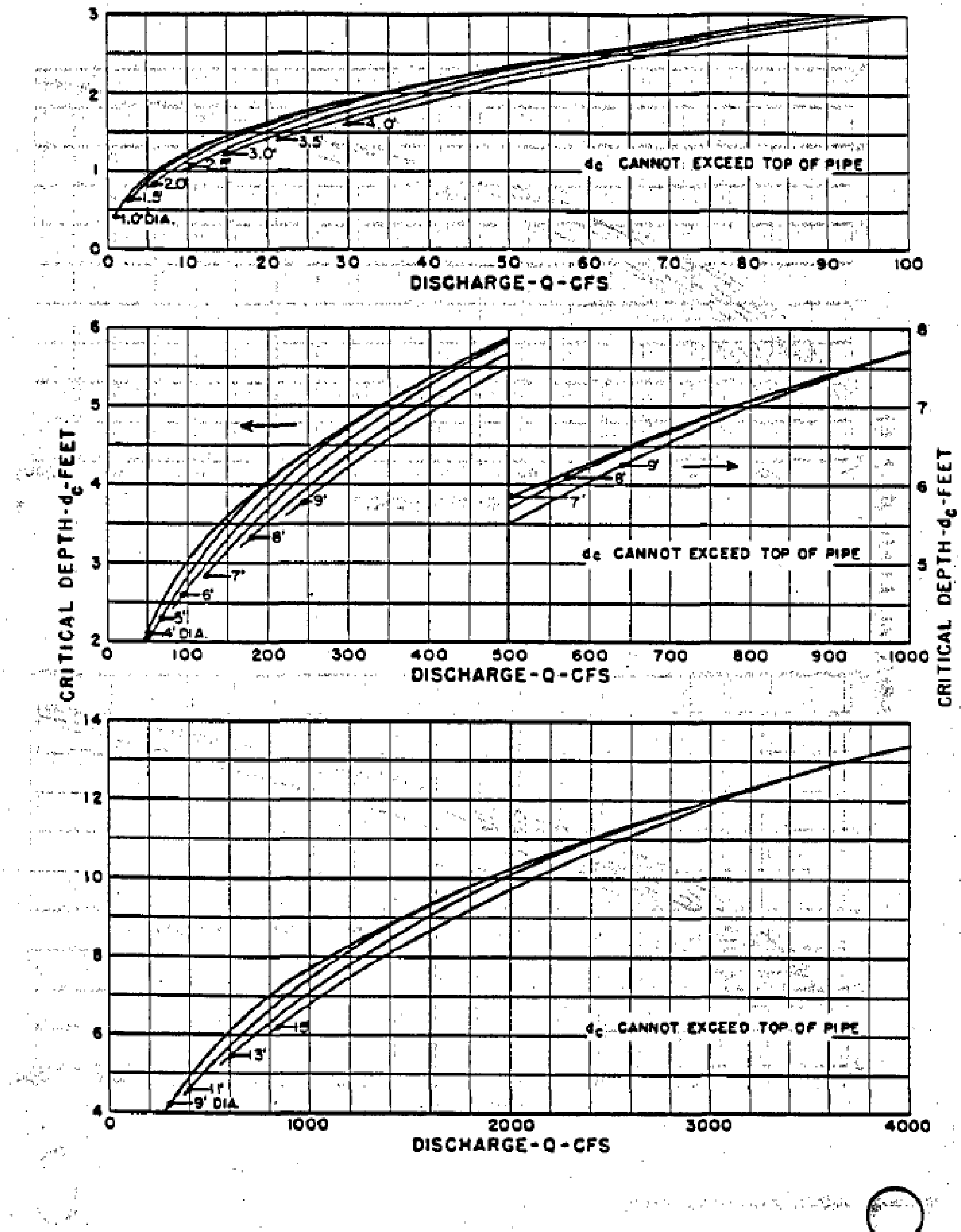
FIGURE 6-18

Outlet Control Chart for Arch Concrete Pipe Culverts Flowing Full



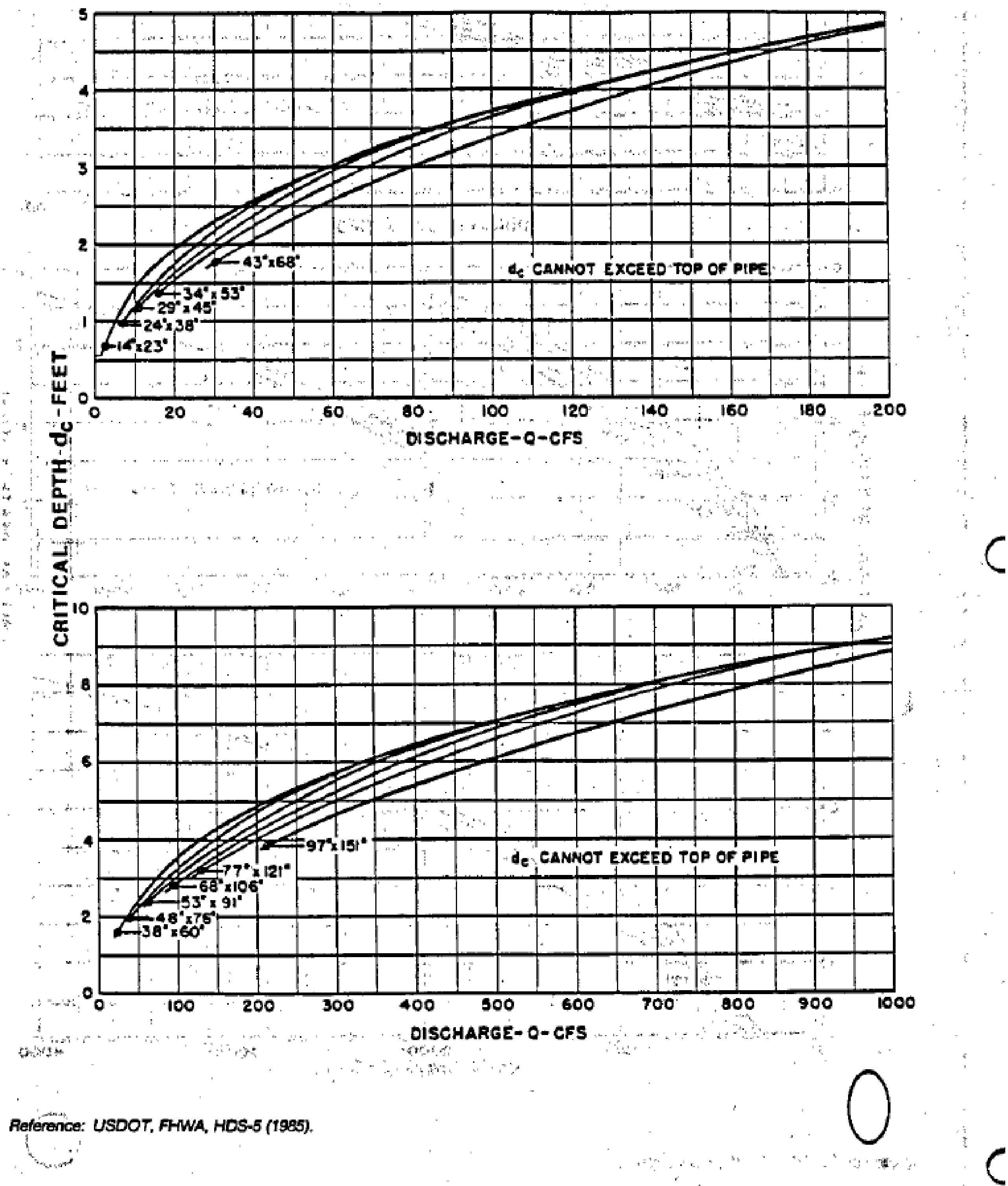
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-19
Critical Depth Chart for Rectangular Sections



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

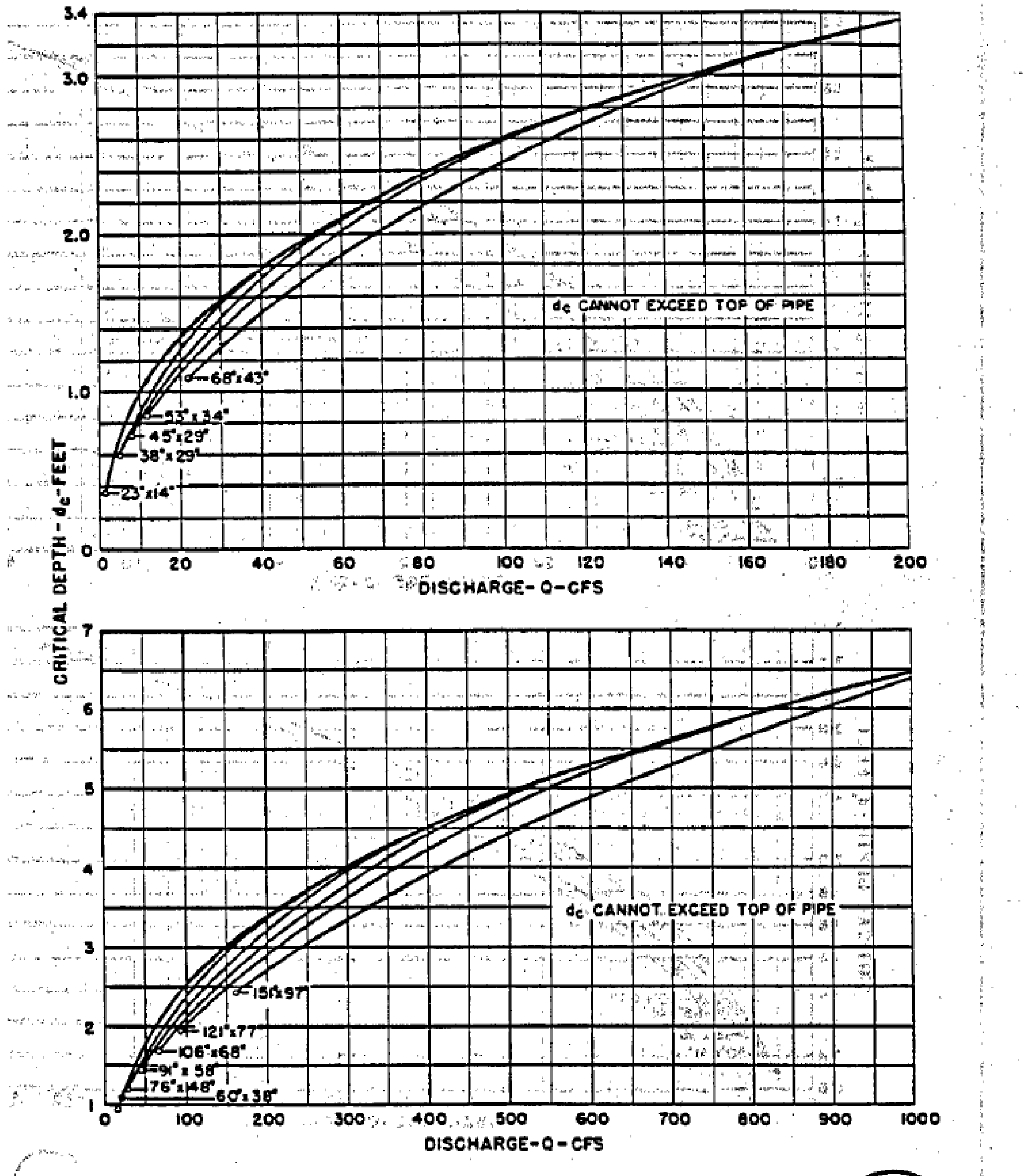
FIGURE 6-20
Critical Depth Chart for Circular Pipe



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

FIGURE 6-21

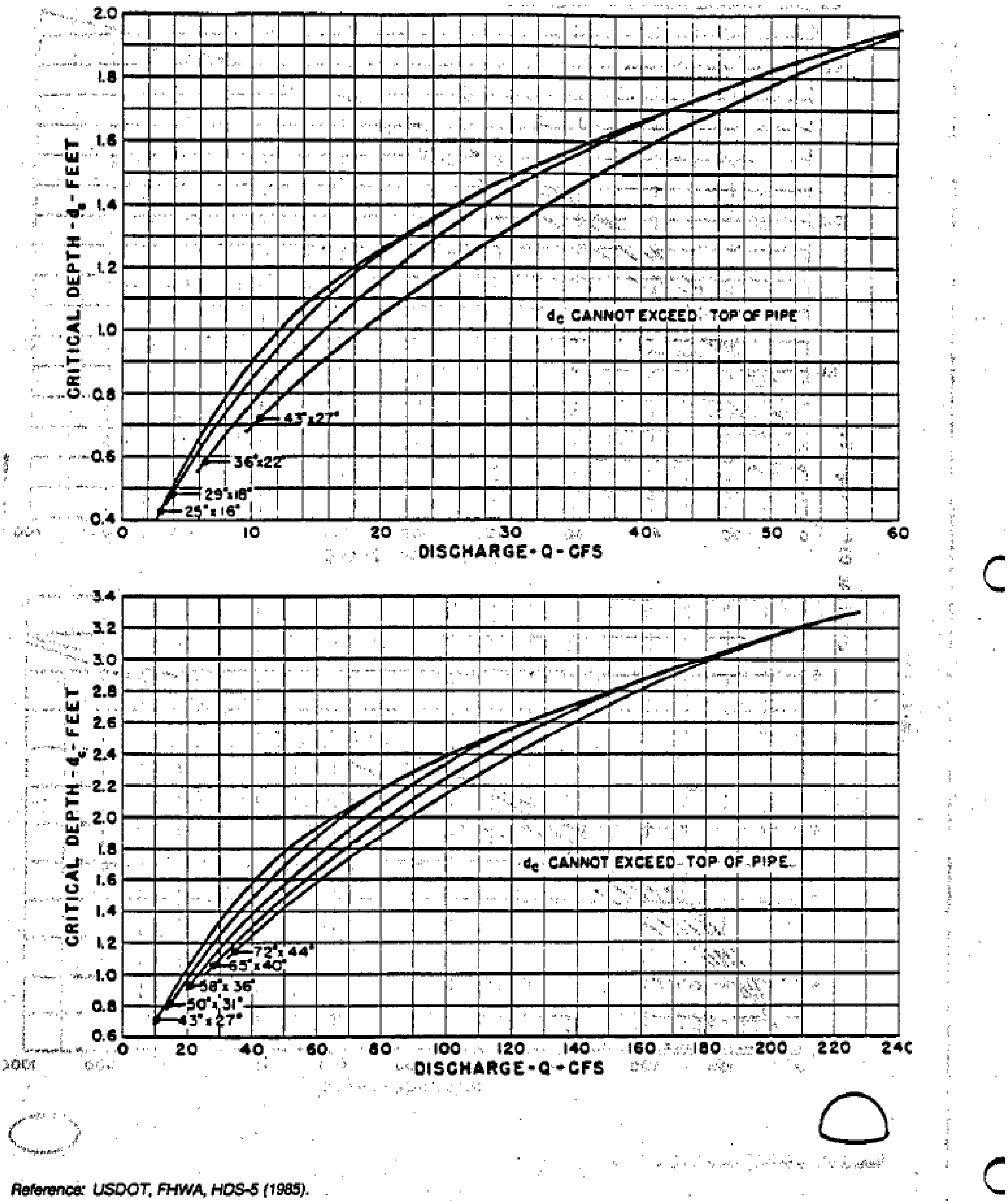
Critical Depth Chart for Oval Concrete Pipe—Long Axis Vertical



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

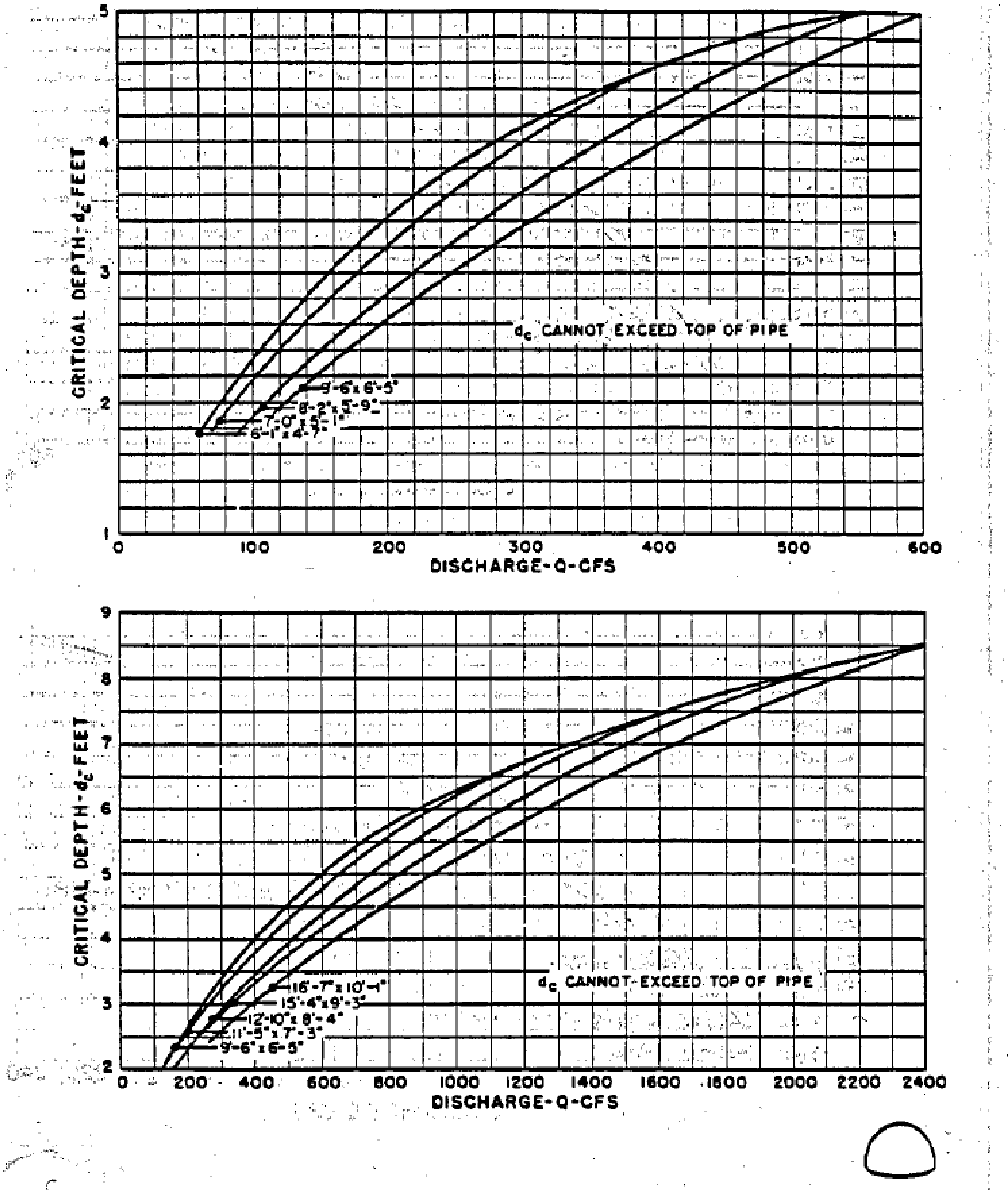
FIGURE 6-22

Critical Depth Chart for Oval Concrete Pipe—Long Axis Horizontal



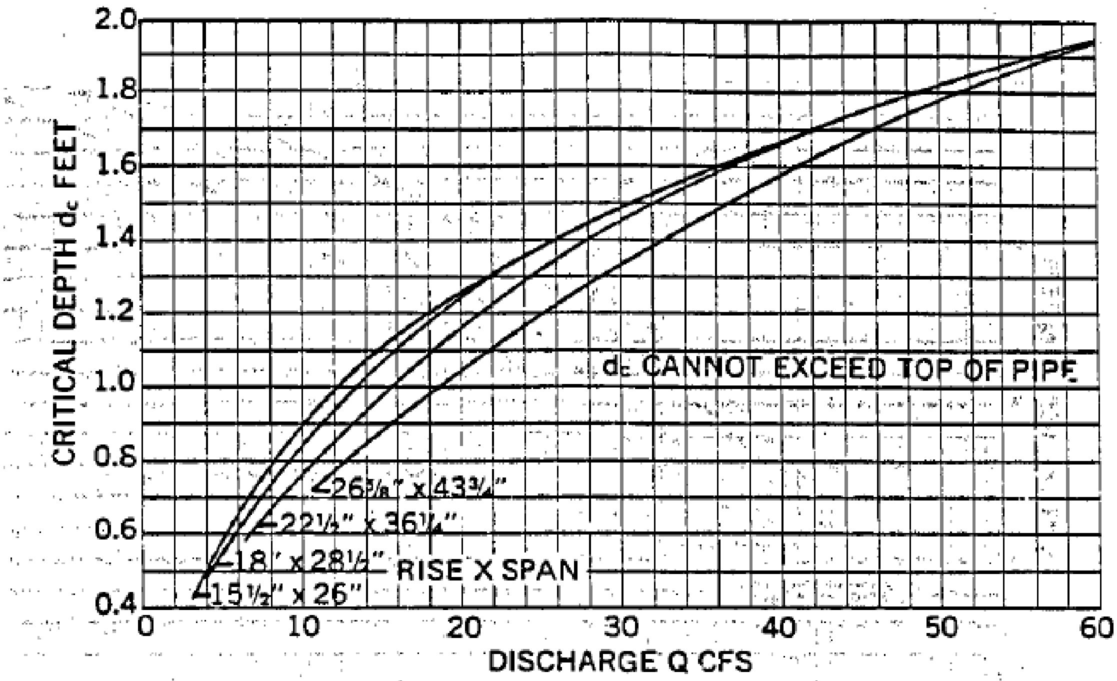
Reference: USDOT, FHWA, HDS-5 (1985).

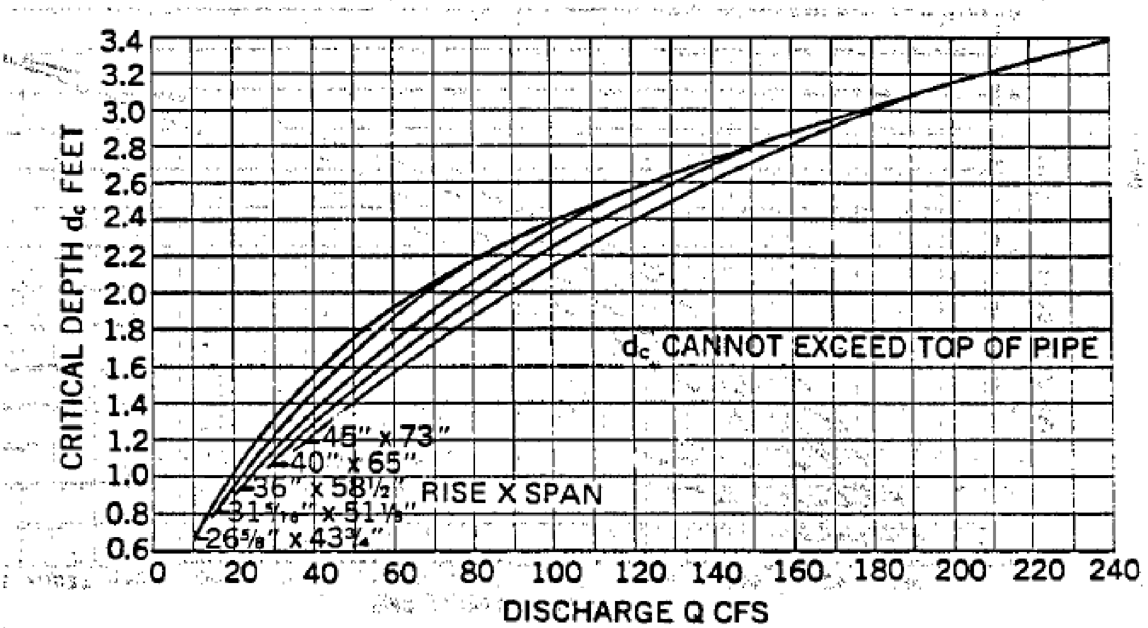
FIGURE 6-23
Critical Depth Chart for Standard CMP Arch

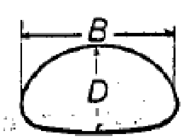


Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-24
Critical Depth Chart for Structural Plate CMP Arch

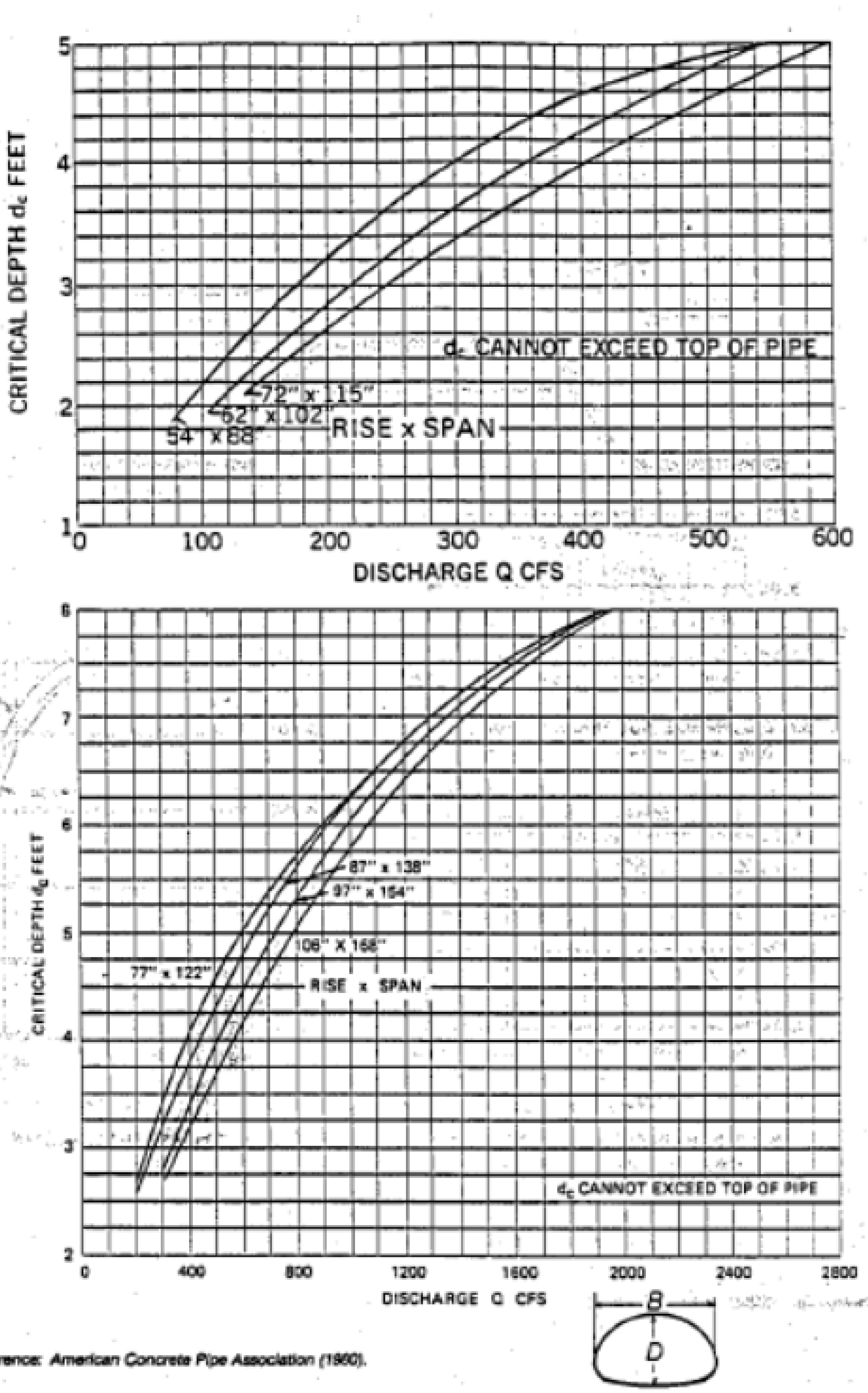






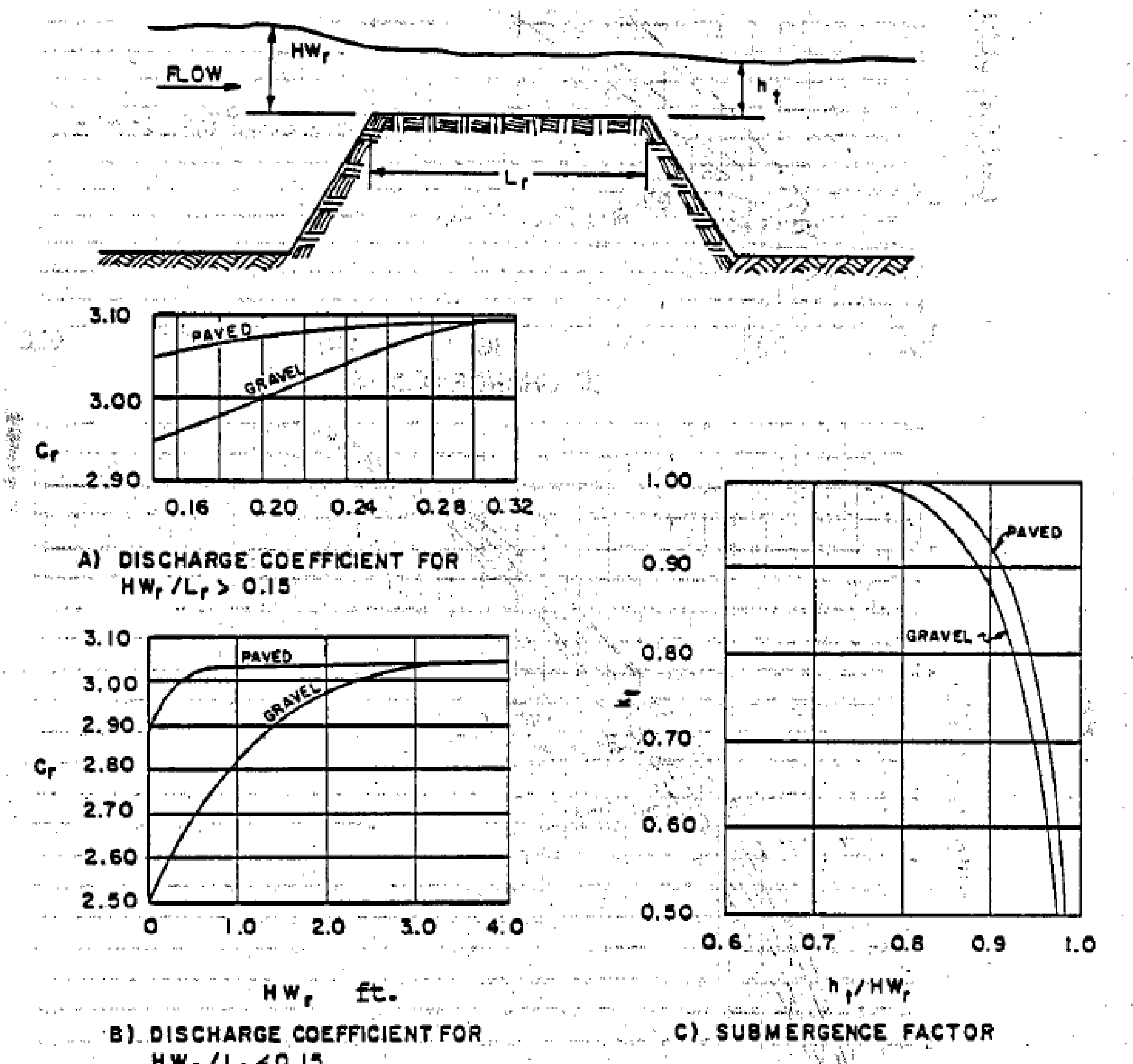
Reference: American Concrete Pipe Association (1980).

FIGURE 6-25
Critical Depth Chart for Arch Concrete Pipe
Sizes 15½" x 26" to 45" x 73"



Reference: American Concrete Pipe Association (1980).

FIGURE 6-26
Critical Depth Chart for Arch Concrete Pipe Sizes 54" x 88" to 106" x 168"



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

FIGURE 6-27
Discharge Coefficients for Roadway Overtopping

CHAPTER 7
GUTTER AND INLET HYDRAULICS

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GUTTER AND INLET HYDRAULICS

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GUTTER AND INLET HYDRAULICS

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CHAPTER 7
GUTTER AND INLET HYDRAULICS

7.1 DESIGN CRITERIA

The following design criteria are factors to consider for gutter and inlet capacity calculations:

1. Return period
2. Spread
3. Manning’s n values
4. Longitudinal slope
5. Cross slope
6. Curb and gutter sections
7. Inlet spacing
8. Roadside Ditches
9. Bridge Decks

7.1.1 RETURN PERIOD

The design storm return period for pavement drainage shall be consistent with the value selected for other components of the drainage system, as follows:

Street Classification	Return Period
Major and Minor Arterials	25-year
Major Collectors	25-year
Minor Collectors	10-year
Residential and Local	10-year

7.1.2 SPREAD

Spread is defined as the width of water transported on the pavement measured from the face of the curb. For roadways, a 10-foot spread shall not be exceeded during the design storm. Where streets are approved with a total lane width of less than 12 feet, an 8-foot maximum spread will be allowed.

7.1.3 MANNING'S n VALUES

Curb and gutter flow characteristics, including spread, shall be calculated using a minimum Manning's n value of 0.016. Higher design n values may be required by the Director of City Engineering where small slope, unusual roughness, or sediment are present.

7.1.4 LONGITUDINAL SLOPE

Curb and gutter longitudinal slopes shall not exceed 12 percent or fall below 1/2 of 1 percent.

7.1.5 CROSS SLOPE

The design of pavement cross slope on residential streets shall conform to the standard roadway sections provided in the standard drawings (see Appendix 3, bound separately). Shoulders should generally be sloped to drain away from the pavement, except with raised, narrow medians.

7.1.6 15" CURB AND GUTTER SECTIONS

Concrete curb and gutter at the outside edge of pavements is normal practice for urban roadway facilities. Combination curb and gutter sections may be 2.0 or 2.5 feet wide with a 1-inch per foot cross slope for the gutter portion of the section consistent with the standard drawings (see Appendix 3, bound separately). Curb and gutter capacities for conveying pavement drainage to the stormwater inlets must be considered in the design of the roadway. Allowable gutter flow rates shall be based on a gutter cross slope of 0.5 inch per foot to account for possible debris and sediment accumulation.

7.1.7 INLET SPACING

Inlets shall be located or spaced so that at least 80 percent of the gutter flow is intercepted (i.e., no more than 20 percent of the gutter flow may bypass the inlet). The maximum length of longitudinal curb and gutter section without a Type S inlet shall be 300 feet.

No flow will be allowed to cross intersecting streets unless approved by Director of City Engineering. Curb inlets will not be allowed within the radii at street intersections.

Yard inlets shall be designed to intercept the total design flow approaching the inlet. Standard grated structures may be used, but open-top structures will not be allowed.

7.1.8 ROADSIDE DITCHES

All new roadways will require curb and gutter to conform with the Subdivision Regulations of the City of Huntsville, unless curb and gutter installation, is, exempted by the Planning Commission. Where roadside ditches are approved along the roadway, by the Director of City Engineering, the design criteria shall conform to the standard roadway sections as shown in the standard drawings (see Appendix 3, bound separately). Where practicable the flow from upgradient areas drainage toward curbed highway pavements should be intercepted by ditches.

Roadside ditches can be used with uncurbed roadway sections to convey pavement runoff and upgradient area runoff that drains toward the pavement. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. Roadside ditches shall be sized using open channel hydraulic procedures presented in Chapter 5. Roadside drainage structures shall be designed to minimize hazards to vehicles that leave the traveled roadway (Chapter 3, Section 3.6).

7.1.9 BRIDGE DECKS

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect debris, and small inlets on scuppers have a higher potential for clogging than curb-opening inlets. Bridge deck constructability usually requires a constant cross slope, so the guidelines in Section 7.1.5 do not apply. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways shall be intercepted before it reaches a bridge.

7.2 GUTTER FLOW CALCULATIONS

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

$$Q = \frac{0.56}{n} S_x^{5/3} S^{1/2} T^{8/3} \tag{7-1}$$

where:

Q = Gutter flow rate, in cfs

n = Manning's roughness coefficient

S_x = Pavement cross slope, in feet/foot

S = Longitudinal slope, in feet/foot

T = Width of flow or spread, in feet

A nomograph for solving Equation 7-1 is presented in Figure 7-1. Manning's n values for various pavement surfaces are presented in Table 7-1.

7.2.1 UNIFORM CROSS SLOPES

The nomograph in Figure 7-1 is used with the following procedures find gutter capacity for uniform cross slopes:

Condition 1: Find spread given gutter flow.

1. Determine input parameters, including longitudinal slope, S, cross slope, S_x, g gutter flow, and Manning's n.

2. Draw a line between the S and S_x scales and note where it intersects the turning line.

3. Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1;

if not, use the product of Q and n.

4. Read the value of the spread, T at the intersection of the line from Step 3 and the spread scale.

Condition 2: Find gutter flow given spread.

1. Determine input parameters, including longitudinal slope, S, cross slope S_X, spread, T, and Manning's n.

2. Draw a line between the S and S_X scales and note where it intersects the turning line.

3. Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

4. For Manning's n values of 0.016, the gutter capacity, Q, from Step 3 is selected. For other Manning's n values, the gutter capacity times n, Qn, is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

7.2.2 STANDARD GUTTER SECTIONS

Allowable capacity data for Huntsville standard curb and gutter for residential pavement sections are presented in Table 7-2. Table 7-2 provides allowable gutter capacities for 2.5-foot curb and gutter section using a residential pavement cross slope of 0.0256 foot/foot and a maximum spread of 10 feet. These capacity data shall also be used for 2.0-foot curb and gutter section. When the roadway or longitudinal slope is known, the allowable gutter capacity can be estimated directly from Table 7-2.

7.2.3 COMPOSITE GUTTER SECTIONS

Figure 7-2 can be used to find the flow in any composite gutter section with width, W, less than the tota1 spread, T. The following steps are used to evaluate any composite gutter section:

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope, S, cross slope, S_X, depressed section slope, S_W, depressed section width, W, Manning's n, gutter flow, Q, and a trial value of the gutter capacity above the depressed section, Q_S.

2. Calculate the gutter flow in W, Q_W, using the equation:

$$Q_W = Q - Q_S \qquad (7\text{-}2)$$

3. Calculate the ratios Q_W/Q or E_0 and S_W/S_X and use Figure 7-2 to find an appropriate value of W/T from Step 3.

4. Calculate the spread, T, by dividing the depressed section width,W, by the value

of W/T from Step 3.

5. Find the spread above the depressed section, T_S, by subtracting W from the value of T obtained in Step 4.

6. Use the value of T_S from Step 5 along with Manning's n, S, and S_X^S to find the actual value of Q from Figure 7-1 (see Section 7.2.1, Condition 2).

7. Compare the value of Q_S from Step 6 to the trial value from Step l. If values are not comparable, select a new value of Q_S and return to Step 1.

Condition 2: Find gutter flow, given spread.

1. Determine input parameters, including spread, T, spread above the depressed section, T_S, cross slope, S_X, longitudinal slope, S, depressed section slope, S_W, depressed section width, W, Manning's n, and depth of gutter flow, d.

2. Use Figure 7-l to determine the capacity of the gutter section above the depressed section, Q_S. Use the procedure in Section 7.2.1, Condition 2, substituting T_S for T.

3. Calculate the ratios of W/T and S_W/S_X, and, from Figure 7-2, find the appropriate value of E_0 (the ratio of Q_W/Q).

4. Calculate the total gutter flow using the equation:

$$Q = Q_S / (1-E_0) \qquad (7\text{-}3)$$

where:

Q = Gutter flow rate, in cfs

Q_S = Flow capacity of the gutter section above the depressed section, in cfs

E_0 = Ratio of frontal flow to total gutter flow (Q_W/Q), obtained from Figure 7-3

5. Calculate the gutter flow in W, Q_W, with Equation 7-2.

7.3 CURB-OPENING INLETS

Curb-opening inlets are relatively free of clogging problems and offer little interference to traffic operation. They are preferable to grates in traffic lanes or where grates would be hazardous for pedestrians or bicyclists. Procedures to determine the capacity of curb-opening inlets placed on continuous grade and at sump locations are presented below.

7.3.1 CONTINUOUS GRADE—TYPE S

Table 7-3 provides allowable gutter flow, inlet intercept, and bypass estimates for Huntsville 2.5-foot curb and gutter on residential streets and Type S curb-opening inlet on a continuous grade. The estimates are based on the typical sections presented in the standard drawing (see Appendix 3, bound separately), using the general procedure for curb-opening inlets presented below.

Up to roadway slopes of 6 percent, the Type S inlet has sufficient capacity to handle allowable gutter flow with a maximum spread of 10 feet and a bypass of 20 percent for the 2.5-foot curb and gutter section. Above this slope, the allowable gutter flow is controlled a maximum of 20 percent bypass and the spread must be adjusted accordingly.

7.3.2 CONTINUOUS GRADE--GENERAL

The length of curb opening required for total interception of gutter flow can be estimated for general conditions using Figure 7-3. The efficiency of inlets that are shorter than the length required for total interception can be evaluated using Figure 7-4. Steps for using these figures are given below:

1. Determine input parameters, including cross slope, S_X, longitudinal slope, S, Manning's n, spread, T, opening length, L, and gutter flow, Q (see Section 7.2).

2. Draw a line between the n and S scales on Figure 7-3 and note where it intersects the left turning line.

3. Draw a line between the intersection point from Step 2 and the appropriate value on the S_X scale and note where it intersects the right turning line.

4. Draw a line between the intersection point from Step 3 and the appropriate value on the Q scale and obtain the value of curb opening length, L_T, required for total interception of gutter flow.

5. If L is less than L_T, find the ratio L/L_T and use Figure 7-4 to find the inlet efficiency, E.

6. Calculate the intercepted flow, Q_i, as the product of E and total gutter flow, Q

7.3.3 SUMP CONDITIONS

A curb-opening inlet located in sump conditions operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height, h. At depths between 1.0 and 1.4 times h, the flow is in a transition stage. Figures 7-5 and 7 6 provide graphical solutions to the weir and orifice equations for depressed and non-depressed curb-opening inlets, respectively. Steps for using these figures are given below.

Depressed Conditions

1. Determine input parameters, including opening length, L, depressed opening height, h, cross slope, S_X, depression, a, depression width, W, and gutter spread, T.

2. Determine the depth of gutter flow using the following equation:

$$d = TS_X \tag{7-4}$$

where:

d = Depth of gutter flow, in feet

T = Gutter spread, in feet

S_X = Cross slope, in feet/foot

3. Calculate the depth of flow above the bottom lip of the depressed opening, d_i, using the following equation:

$$d_i = d + a/12 \tag{7-5}$$

where:

d_i = Depth of flow above the bottom lip of the depressed opening, in feet

d = Depth of gutter flow, in feet, from Step 2

a = Depth of depression, in feet

4. If the value of d_i from Step 3 is less than or equal to the depressed opening height, h, use the weir flow curves at the bottom of Figure 7-5 to find inlet capacity, Q. Enter the y-axis with the value of d from Step 2, run a horizontal line to the right, and 1ocate the intersection with the appropriate value of P = L + 1.8w. From that point of intersection, run a vertical line downward to obtain the inlet capacity, Q.

5. If the value of d_i from Step 3 is greater than or equal to 1.4 times the depressed opening height, h, use the orifice flow curves at the top of Figure 7-5 to find the inlet capacity, Q. Enter the y-axis with the value of d_i from Step 3, run a horizontal line to the right, and locate the intersection with the appropriate values of A = hL and h. From that point of intersection, run a vertical line downward to obtain the inlet capacity, Q.

6. If the value of d_i from Step 3 is between 1.0 and 1.4 times the depressed opening height, h, transition flow exists. Draw a curve representing the transition between weir and

orifice flow for the appropriate values of P = L + 1.8W and A= hL and obtain the capacity, Q, from the intersection of that curve with the value of d_i.

7. To find the depth required to handle a desired inlet capacity, Q, enter the x-axis of Figure 7-5 with the value of Q and use the appropriate curve to find the depth on the y-axis.

Undepressed Conditions

1. Determine input parameters, including opening length, L, undepressed opening height, h, cross slope, S_x, and gutter spread, T.

2. Determine the depth of gutter flow, d, using Equation 7-4.

3. If the value of d from Step 2 is less than or equal to the undepressed opening height, h, use the weir flow curves at the bottom of Figure 7-6 to find inlet capacity, Q. Enter the y-axis with the value of d, run a horizontal line to the right, and locate the intersection with the appropriate value of L. From that point of intersection, run a vertical line downward to obtain the inlet capacity, Q.

4. If the value of d from Step 2 is greater than or equal to 1.4 times the undepressed opening height, h, use the orifice flow curves at the top of Figure 7-6 to find inlet capacity, Q. Enter the y-axis with the value of d, run a horizontal line to the right, and locate the intersection with the appropriate value of A=hL and h. From that point of intersection, run a vertical line downward to obtain the inlet capacity, Q.

5. If the value of d from Step 2 is between 1.0 and 1.4 times the undepressed opening height, h, transition flow exists. Draw a curve representing the transition between weir and orifice flow for the appropriate values of L and A and obtain the capacity, Q, from the intersection of that transition curve with the value of d.

6. To find the depth required to handle a desired inlet capacity, Q, enter the x-axis of Figure 7-6 with the value of Q and use the appropriate curve to find the depth on the y-axis.

7.4 GRATE INLETS

Grates are efficient for intercepting pavement drainage if clogging by debris is properly controlled. Grate inlets will intercept all of the gutter flow passing over the front of the grate if the gutter flow does not splash over the grate. The portion of side flow intercepted will depend on the cross slope of the pavement, length of grate, and flow velocity.

Procedures to determine the capacity of grate inlets placed on continuous grade and at sump locations are available from HEC-12 (USDOT, FHWA, 1984).

7.5 COMBINATION INLETS

7.5.1 CONTINUOUS GRADE

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. A combination inlet with the curb opening located upstream of the grate has an interception capacity equal to the sum of the two inlets, except that the frontal flow of the grate is reduced by the amount of flow intercepted by the curb opening. By placing the curb opening upstream, debris can generally be intercepted before it clogs the grate.

7.5.2 SUMP LOCATIONS

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, the capacity of a combination inlet at a sump shall neglect the grate inlet capacity.

Table 7-1
MANNING'S n VALUES FOFR STREET AND PAVEMENT GUTTERS

Type of Gutter or Pavement	Manning's n
Standard Huntsville design value (see Section 7.1.3)	0.016
Concrete gutter, troweled finish	0.012
Asphalt pavement	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement	
Smooth	0.013
Rough	0.015
Concrete pavement	
Float finish	0.014
Broom finish	0.016
For gutters where sediment may Accumulate, increase values of n by	0.002

Reference: USDOT, FHWA, HDS-3 (1961).

Table 7-2
HYDRAULIC CAPACITIES OF HUNTSVILLE STANDARD 2.5-FOOT
CURB AND GUTTER FOR RESIDENTIAL PAVEMENT SECTIONS

Roadway Slope (%)	Allowable Flow (cfs)	Roadway Slope (%)	Allowable Flow (cfs)
0.50	2.68	6.50	9.66
.075	3.28	6.75	9.85
1.00	3.79	7.00	10.03
1.25	4.24	7.25	10.20
1.50	4.64	7.50	10.38
1.75	5.01	7.75	10.55
2.00	5.36	8.00	10.72
2.25	5.69	8.25	10.89
2.50	5.99	8.50	11.05
2.75	6.29	8.75	11.21
3.00	6.56	9.00	11.37
3.25	6.83	9.25	11.53
3.50	7.09	9.50	11.68
3.75	7.34	9.75	11.83
4.00	7.58	10.00	11.99
4.25	7.81	10.25	12.13
4.50	8.04	10.50	12.25
4.75	8.26	10.75	12.43
5.00	8.47	11.00	12.57
5.25	8.68	11.25	12.71
5.50	8.89	11.50	12.85
5.75	9.09	11.75	12.99
6.00	9.28	12.00	13.13
6.25	9.48		

NOTES:

1. These capacity data shall also be used for 2.0-foot curb and gutter sections.
2. Gutter cross slope for allowable flows is based on 0.5 inch/foot instead of the 1 inch/foot required for construction.
3. Pavement cross slope is 0.0256 ft/ft.
4. Manning's n = 0.016.
5. Stormwater spread is 10 feet.
6. The maximum allowable flow equation for the above condition is $Q = 37.9\ S^{0.5}$.

Table 7-3
INTERCEPT OF TYPE S INLET WITH MAXIMUM ALLOWABLE GUTTER FLOW FOR RESIDENTIAL STREETS AND 2.5-FOOT CURB AND GUTTER

Roadway Slope (%)	Allowable Gutter Flow (cfs)	Inlet Intercept (cfs)	Bypass (cfs)
0.50	2.68	2.68	0.00
.075	3.28	3.28	0.00
1.00	3.79	3.79	0.00
1.25	4.24	4.24	0.00
1.50	4.64	4.64	0.00
1.75	5.01	5.01	0.00
2.00	5.36	5.36	0.00
2.25	5.69	5.68	0.00
2.50	5.99	5.95	0.05
2.75	6.29	6.17	0.12
3.00	6.56	6.35	0.21
3.25	6.83	6.51	0.32
3.50	7.09	6.65	0.44
3.75	7.34	6.77	0.56
4.00	7.58	6.88	0.70
4.25	7.81	6.98	0.83
4.50	8.04	7.07	0.97
4.75	8.26	7.16	1.10
5.00	8.47	7.23	1.24
5.25	8.68	7.30	1.38
5.50	8.89	7.37	1.52
5.75	9.09	7.42	1.66
6.00	9.28	7.48	1.80

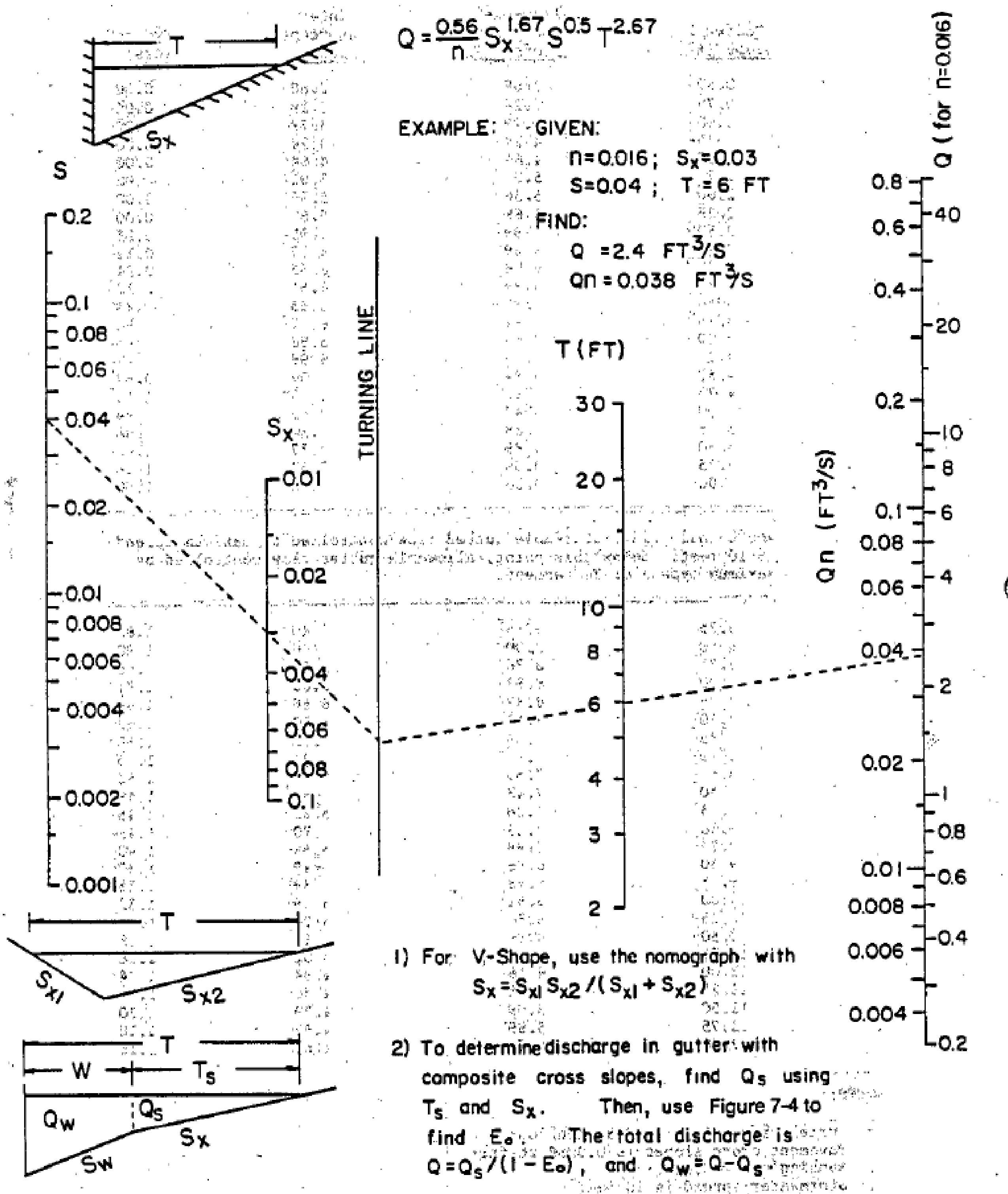
Table 7-3 (cont.)
INTERCEPT OF TYPE S INLET WITH MAXIMUM ALLOWABLE GUTTER FLOW FOR RESIDENTIAL STREETS AND 2.5-FOOT CURB AND GUTTER

Above this point, allowable gutter flow controlled by maximum spread of 10 feet. Below this point, allowable gutter flow controlled by maximum bypass of 20 percent.

6.25	9.25	7.40	1.85
6.50	9.00	7.20	7.80
6.75	8.76	7.01	1.75
7.00	8.53	6.83	1.71
7.25	8.32	6.66	1.66
7.50	8.12	6.50	1.62
7.75	7.93	6.35	1.59
8.00	7.76	6.21	1.55
8.25	7.59	6.07	1.52
8.50	7.42	5.94	1.49
8.75	7.28	5.82	1.45
9.00	7.13	5.70	1.43
9.25	6.99	5.59	1.40
9.50	6.86	5.49	1.37
9.75	6.73	5.39	1.35
10.00	6.61	5.29	1.32
10.25	6.50	5.20	1.30
10.50	6.39	5.11	1.28
10.75	6.28	5.02	1.26
11.00	6.118	4.94	1.24
11.25	6.08	4.86	1.22
11.50	5.99	4.79	1.20
11.75	5.89	4.72	1.18
12.00	5.81	4.65	1.16

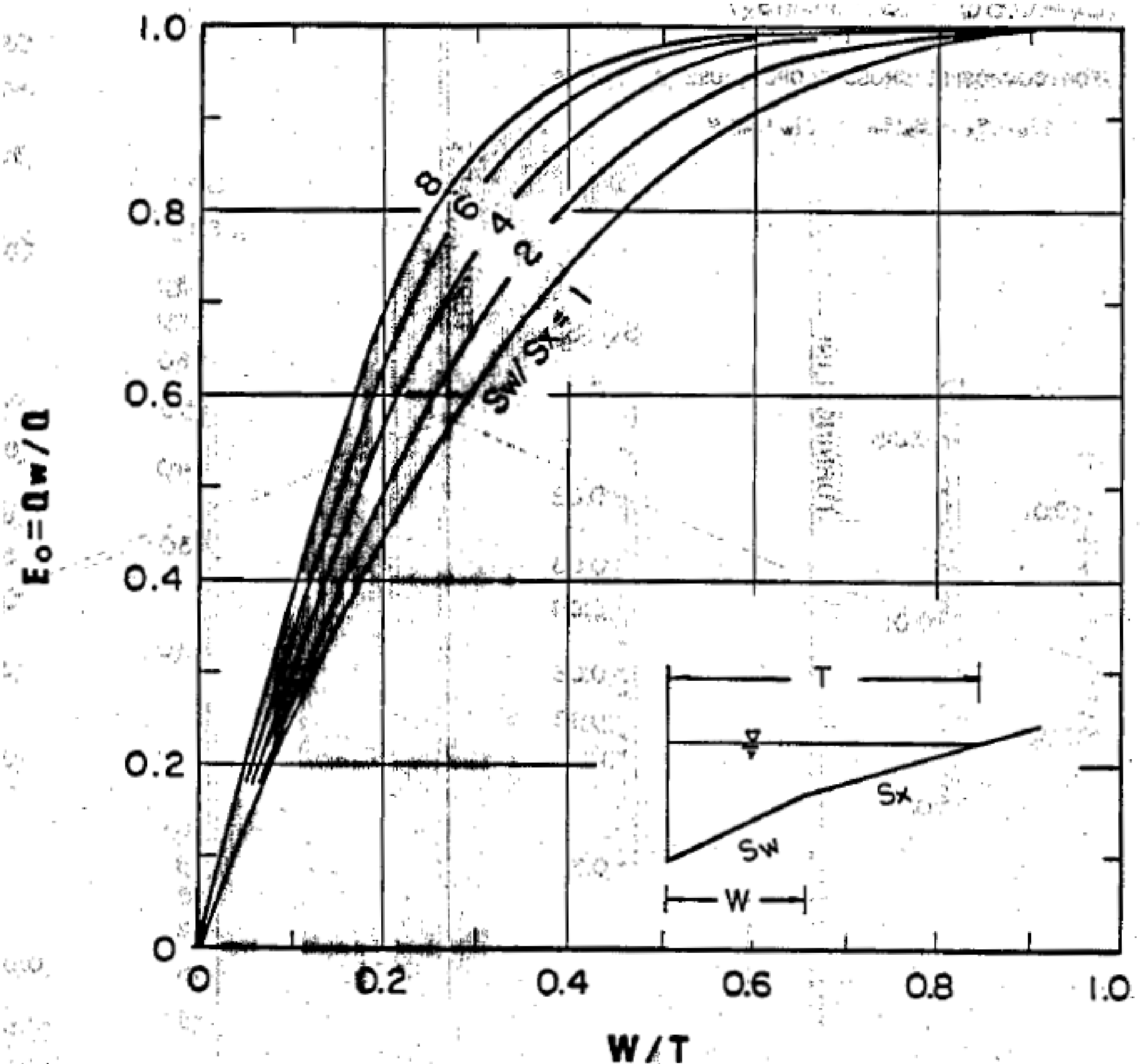
NOTES:

1. Gutter cross slope is 0.5 inch/foot.
2. Pavement cross slopes is 0.0256 ft/ft.
3. Manning's n = 0.016.
4. Stormwater spread is 10 feet.
5. See the standard drawings for details on curb and gutter and residential street sections.



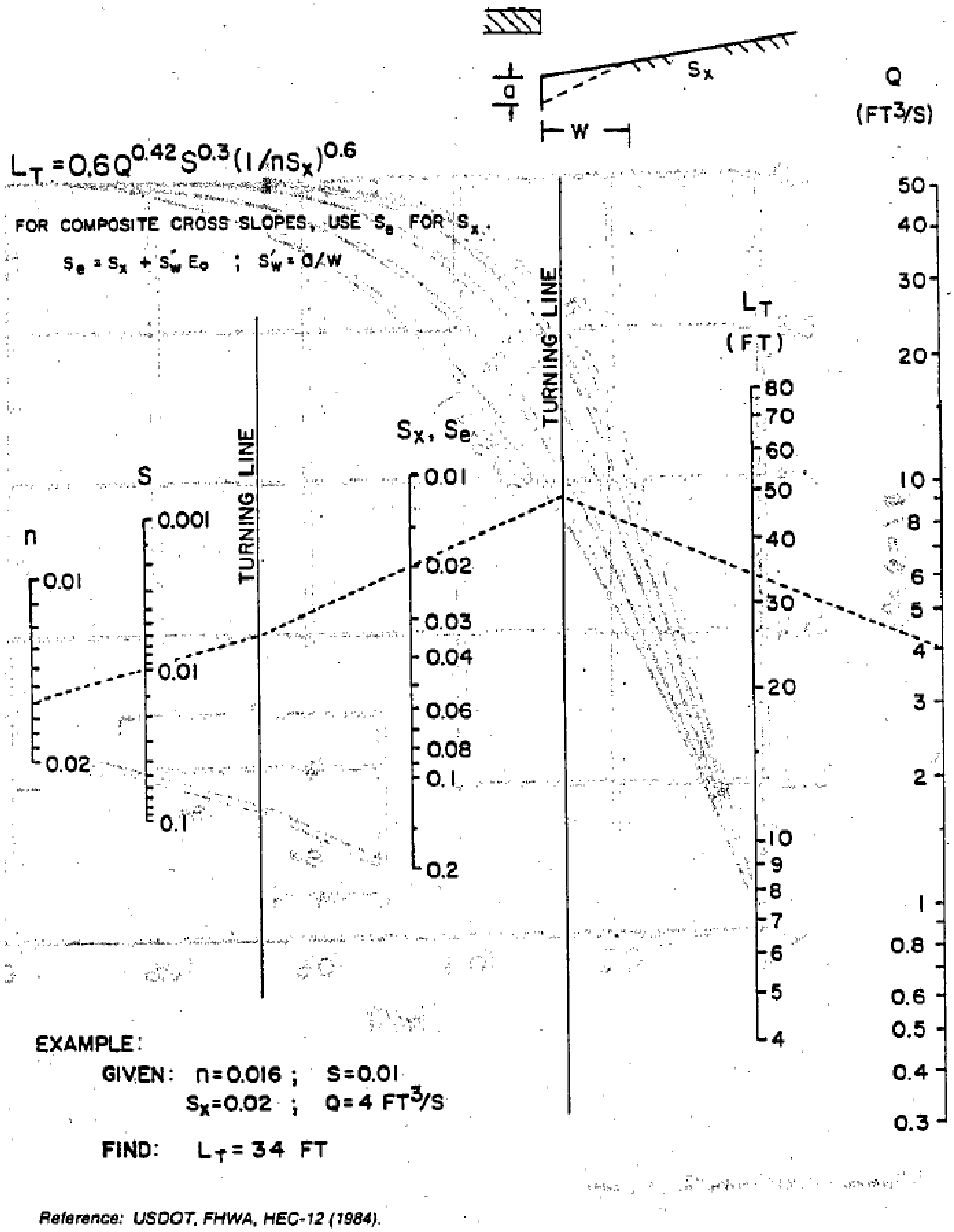
Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 7-1
Nomograph for Flow in Triangular Gutter Sections



Reference: USDOT, FHWA, HEC-12 (1984).

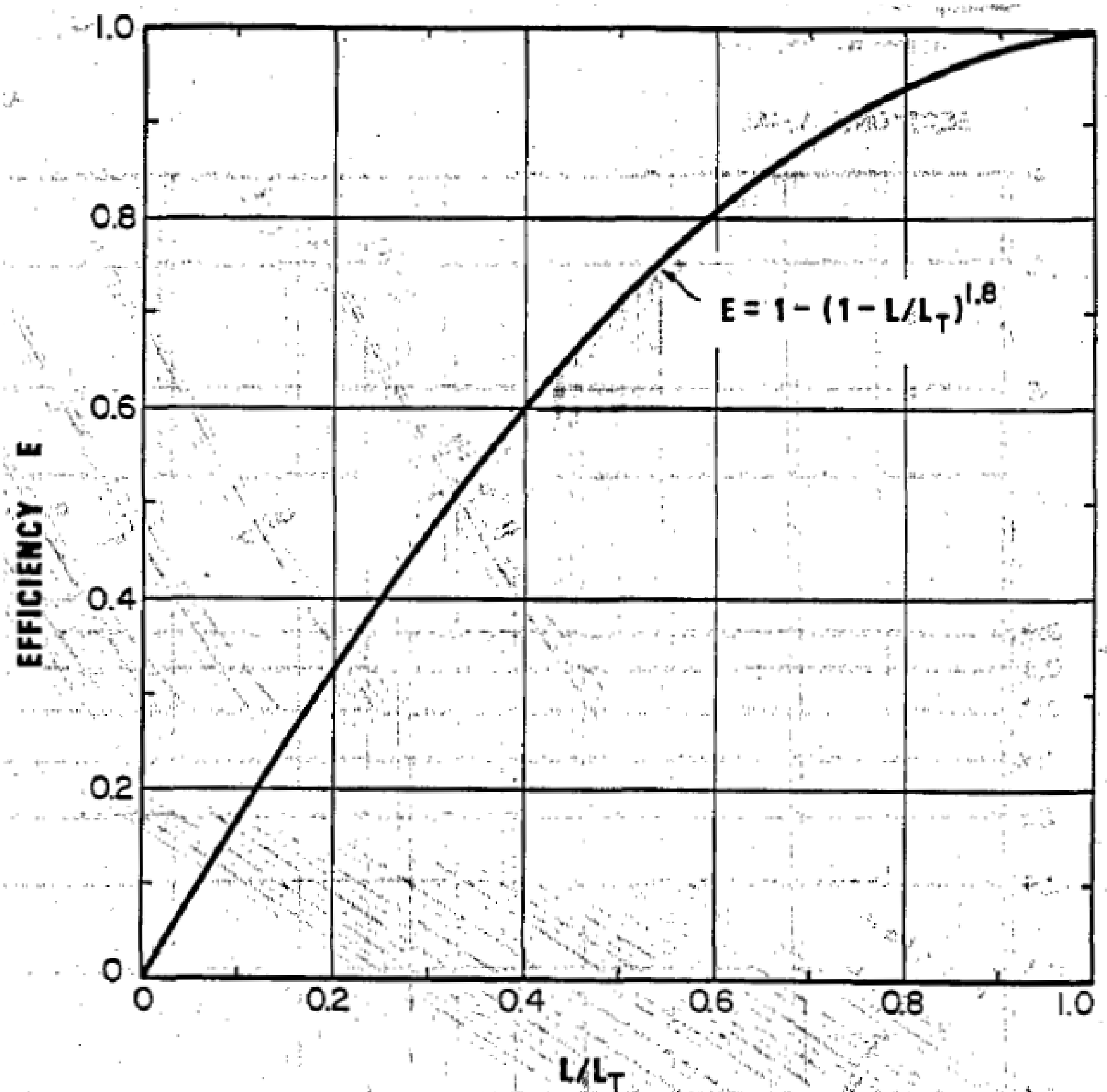
FIGURE 7-2
Ratio of Frontal Flow to Total Gutter Flow



Reference: USDOT, FHWA, HEC-12 (1984).

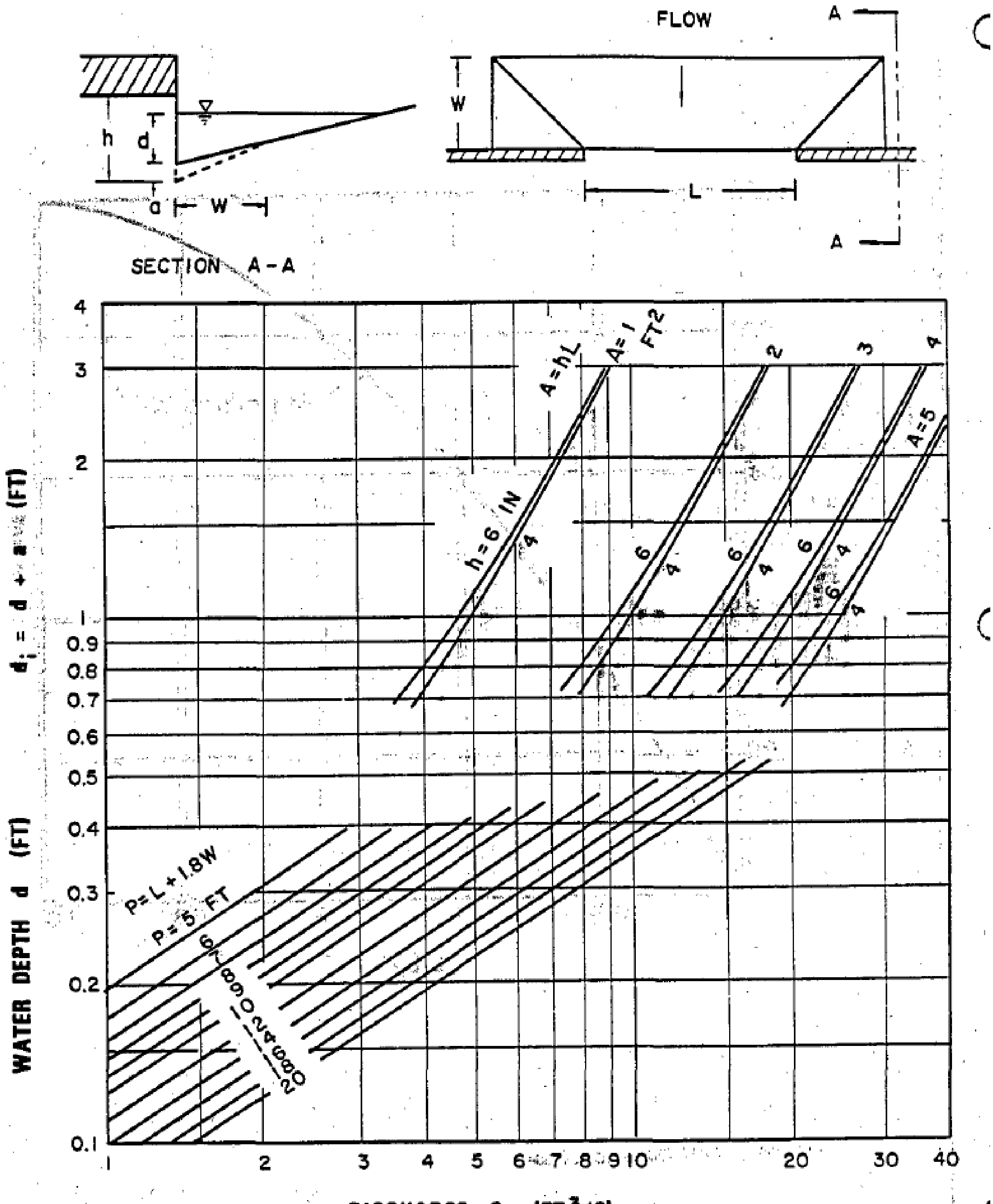
FIGURE 7-3

Curb-Opening and Slotted Pipe Inlet Length for Total Interception



Reference: USDOT, FHWA, HEC-12 (1984).

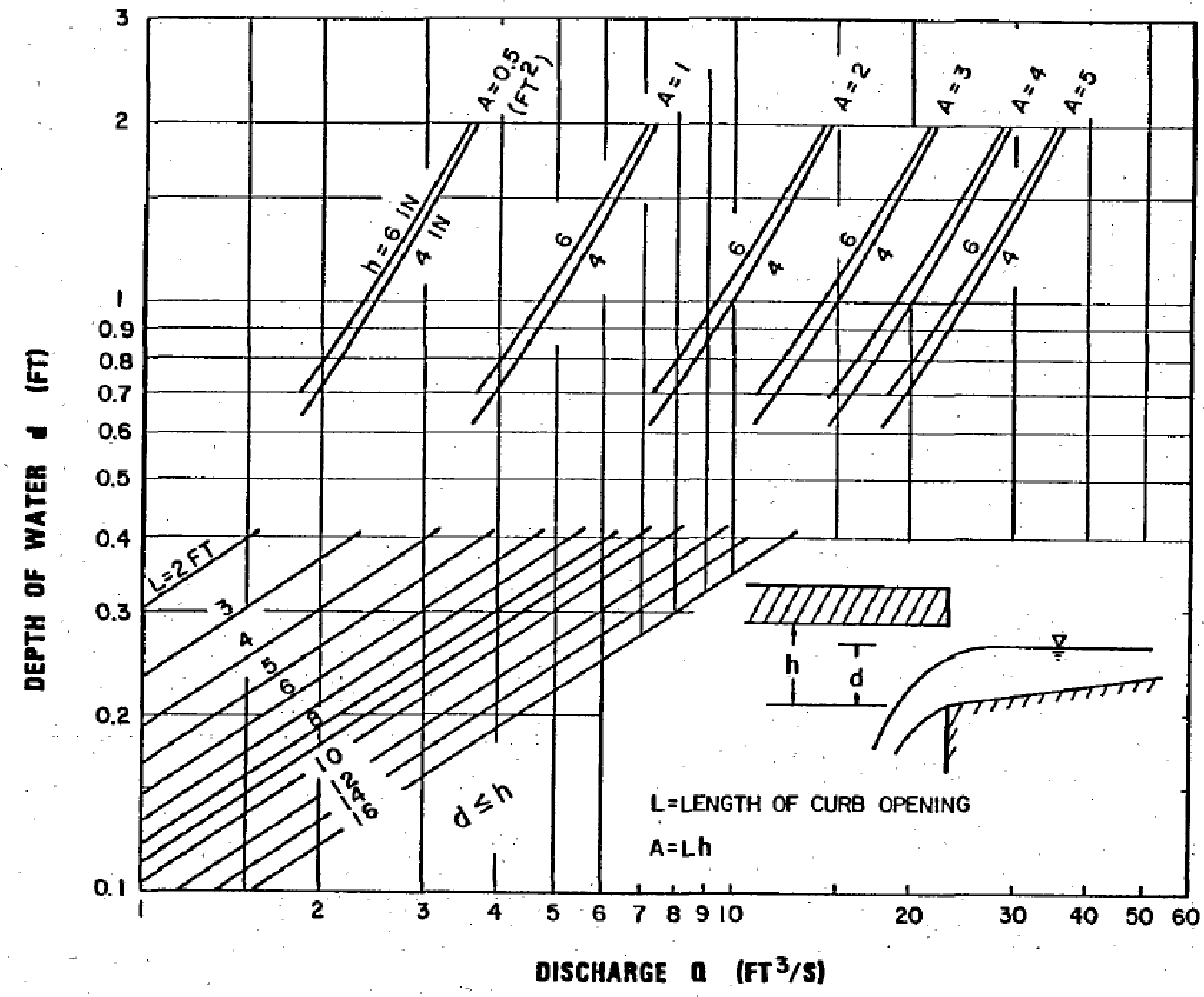
FIGURE 7-4
Curb-Opening and Slotted Pipe Inlet Interception Efficiency



Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 7-5

Depressed Curb-Opening Inlet Capacity in Sump Locations



Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 7-6
Undepressed Curb-Opening Inlet Capacity in Sump Locations

CHAPTER 8
STORM SEWER HYDRAULICS

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STORM SEWER HYDRAULICS

8.1 DESIGN CRITERIA

8.1.1 RETURN PERIODS

The following return period shall be used for storm sewer design:

Street Classification	Return Period
Major and Minor Arterials	25-year
Major Collectors	25-year
Minor Collectors	10-year
Residential and Local	10-year

The impact of a 100-year return period flood shall be analyzed as directed by the Director of City Engineering.

8.1.2 MANNING'S n VALUES

All storm sewer pipe shall be reinforced concrete unless alternate materials of equal or greater effectiveness and strength are approved by the Director of City Engineering. Values for Manning's roughness coefficient for concrete pipe, concrete box culvert, and corrugated metal pipe (CMP) are given below:

Concrete pipe	n = 0.012
Box culvert (cast-in-place)	n = 0.013
CMP (non-spiral flow, annular corrugations)	n = 0.024
CMP (full pipe spiral flow, helical corrugations)	
Sizes 15-24”	n = 0.017
Sizes 30-54”	n = 0.021
Sizes 60-96”+	n = 0.024

Full spiral flow occurs only for pipes with a length 20 times diameter, operating under full flow, and free of sediment buildup. Conditions where full spiral flow would be appropriate are down drains, retention outlet pipes, and free outlet or gravity storm sewer systems with a design velocity above 4 feet per second.

8.1.3 SLOPES AND HYDRAULIC GRADIENT

The maximum and minimum slopes for storm sewers shall conform to the following criteria:

1. The maximum hydraulic gradient shall not produce a velocity that exceeds 20 feet per Second. Higher velocities require approval from the Director of City Engineering.

2. The minimum desirable physical slope shall be that which will produce a velocity of 2.5 feet per second and have no less than 0.5% slope.

When hydraulic calculations do not consider minor energy losses, the elevation of the hydraulic gradient for design flood conditions shall be at least 1 foot below the gutter elevation. Minor losses shall be considered when the velocity exceeds 6 feet per second. If all minor energy losses are calculated, it is acceptable for the hydraulic gradient to reach the gutter elevations.

8.1.4 PIPE SIZE AND LENGTH

The minimum pipe size shall be 15 inches when access spacing is 50 feet or less. When access spacing exceeds 50 feet, the minimum pipe size shall be 18 inches. Standard pipe size increments of 6 inches shall be used for pipes larger than 18 inches.

The minimum box culvert size shall be 4 by 4 feet. Increments of 1 foot in the height or width shall be used above this minimum. The span by height format is used for reporting box culvert dimensions, e.g., in the dimension 10 by 7 feet, the span is 10 feet and the height is 7 feet.

Access spacing shall not exceed 400 feet for conduits less than 54 inches in diameter and shall not exceed 800 feet under any circumstances.

8.1.5 MINIMUM CLEARANCES

Minimum clearances for storm sewer pipe shall comply with the following criteria:

1. A minimum of 1 foot is required between the bottom of the road base material and the outside crown of the storm sewer, with 15 to 18 inches preferred.

2. For utility conflicts that involve crossing a storm sewer alignment, the recommended minimum design clearance between the outside of the pipe and the outside of any conflicting utility should be 0.5 foot if the utility has been accurately located at the point of conflict. If the utility has been approximately located, the minimum design clearance should be 1 foot. Electrical transmission lines or gas mains should never come into direct contact with the storm sewer.

3. Storm sewer systems should not be placed parallel to or below existing utilities in a manner that could cause utility support problems. Recommended clearance is 2 feet extending from each side of the storm sewer and 1:1 side slopes from the trench bottom. Where right-of-way limitations are inadequate, trench shoring is acceptable.

4. When a sanitary line or other utility must pass through a manhole, a minimum 1-foot clearance should be maintained between the bottom of the utility and the flow line of the storm main, and greater clearance is recommended. Flow will be less obstructed when the utility is placed above or as close as possible to the crown of the pipe.

8.1.6 INLET LOCATION AND SPACING

Curb inlets shall be located to facilitate the entrance of water from gutters into the storm sewer system. Consideration shall also be given to the movement of vehicles to and from adjacent property on turnouts and to the maintenance of safe pedestrian walkways. Curb inlets shall not be located within handicap ramps and shall be avoided within a 35-foot radius of turns.

The location and spacing of inlets shall be based on inlet capacity and width of spread calculations. Surface stormwater shall not be allowed to drain across street intersections. Procedures in Chapter 7 shall be used to establish inlet spacing such that a maximum of 20 percent bypass is allowed. The last inlet shall have no bypass.

8.2 GENERAL PROCEDURE

The design of storm sewer systems is usually an iterative process involving the following four steps:

1. System Layout: Selection of inlet locations and development of a preliminary plan and profile configurations consistent with design criteria in Section 8.1.

2. Hydrologic Calculations: Determination of design flow rates and volumes (see Section 8.3).

3. Hydraulic Calculations: Determination of pipe sizes required to carry design flow rates and volumes, as discussed in Section 8.4.

4. Outfall Design: Outlet protection or detention/retention may be required because of downstream constraints; see Chapter 9 for detention/retention or Chapter 10 for outlet protection.

8.3 HYDROLOGIC CALCULATIONS

The hydrologic determination of a peak runoff rate for sizing a storm sewer system shall be made using procedures presented in Chapter 4. In general, storm sewer systems are sized to carry stormwater intercepted by appropriate inlet facilities. However, if the intercepted runoff is transported through an extensive pipe network, channel storage within the storm sewers can modify the peak rate of the runoff as it travels along the system. The peak flow modification can be evaluation with hydrologic channel routing procedures (See Chapter 4).

For many projects, the Rational Method is well suited to performing hydrologic calculations for storm sewer systems. In general, as the time of concentration, drainage area, and variability in land use increase, more complex procedures are warranted. A rule-of-thumb is that flood hydrograph procedures should be considered when the time of concentration goes above 30 to 45 minutes. In addition, the size and complexity of the storm sewer system should be considered. See Chapter 4 for additional guidance on selecting hydrologic methods.

To apply the Rational Method at each design point, the following data are required:

1. Tributary area
2. Time of concentration
3. Rainfall intensity
4. Runoff coefficient

The time of concentration is the sum of the inlet travel time and the storm sewer travel time and must be calculated for each design point considered. Rainfall intensity is obtained from an IDF curve (see Chapter 4), based on the time of concentration and design return period. The runoff coefficient should be the composite factor based on tributary land use and soil conditions. The tabular data presented in Chapter 4 can provide a good basis for selecting the runoff coefficient, but if possible, a comparison of historical performance with the results of design calculations should be made.

The Rational Method as presented in Chapter 4 implicitly assumes that all runoff from the tributary area is intercepted by the storm sewer system. Therefore, because inlet efficiencies of 100 percent are unrealistic, bypass must be accounted for by adjusting the tributary drainage area.

The proper selection of the time of concentration indirectly accounts for situations in which peak runoff arrives at individual inlets at different points in time. However, this approach does not explicitly account for channel storage that can become important in the upstream portions of a large storm sewer system.

In some cases, only a portion of the area draining to a particular location in a storm sewer system will cause the peak discharge. In systems where a small subbasin of the total area has an unusually long travel time, another larger subbasin of the contributing area with a shorter time of concentration (and, therefore, a higher average rainfall intensity) can cause a higher peak discharge.

8.4 HYDRAULIC CALCULATIONS

Hydraulic calculations are used to size pipes to handle the design flows determined from hydrologic calculations (Section 8.3). The hydraulic capacity of a storm sewer pipe can be calculated for the two types of conditions typically referred to as open channel and pressure flow. Results should provide a balanced system in which all segments will be used to their full capacity consistent with the flood protection criteria for the project site.

Hydraulic procedures in this section represent a summary of information from publications by Brater and King (1976), Chow (1959), the American Society of Civil Engineers (1969), the

University of Missouri (1958), the American Iron and Steel Institute, (1980), and Marsalek (1985). These publications should be consulted if additional details are required.

8.4.1 PRESSURE VERSUS OPEN CHANNEL FLOW

Open channel flow occurs when a free water surface is exposed to the atmosphere as a boundary (see Figure 8-1). When the conduit is flowing full, the pipe is considered to be flowing under pressure (see Figure 8-2).

Guidance is presented in Figure 8-3 for determining whether pressure or open channel flow conditions occur in a storm sewer system. In general, if the hydraulic grade line is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the hydraulic grade line is below the crown of the pipe, open channel flow calculations are appropriate.

A special concern with storm sewers designed to operate under pressure flow conditions is that inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system.

As hydraulic calculations are performed, existence of the desired flow condition should be verified. Storm sewer systems can alternate between pressure and open channel flow conditions from one section to another.

The discharge point of the storm sewer system usually establishes a starting point for evaluating flow conditions. If the discharge is submerged, as when the tailwater established by the receiving waterbody is above the crown of the storm sewer, the exit loss should be added to the tailwater and calculations for head loss in the storm sewer system started from this point, as depicted in the submerged discharge conditions of Figure 8-3. If the hydraulic grade line is above the pipe crown at the next upstream manhole, pressure flow calculations are indicated; if it is below the pipe crown, then open channel flow calculations should be used at the upstream manhole.

When the discharge point is not submerged, a flow depth should be determined at some control section to allow calculations to proceed upstream. As shown in Figure 8-3, the hydraulic grade line is then projected to the upstream manhole. Pressure flow calculations may be used at the manhole if the hydraulic grade is above the pipe crown.

The assumption of straight hydraulic grade lines illustrated in Figure 8-3 is not entirely correct, because backwater and drawdown conditions can exist but is generally reasonable for typical pipe sizes. It is also usually appropriate to assume the hydraulic grade calculations begin at the crown of the outlet pipe for simple non-submerged systems. If additional accuracy is needed, as with very large conduits or where the result will have a very significant effect on design, backwater and drawdown curves should be developed.

8.4.2 ENERGY LOSSES

The following energy losses shall be considered for storm sewer systems:

1. Friction
2. Entrance
3. Exit

Additional energy loss parameters should be evaluated for complex or critical systems. The following losses are especially important when failure to handle the design flood has the potential to flood offsite areas:

1. Expansion
2. Contraction
3. Bend
4. Junction and manhole

Friction Loss

The energy loss required to overcome friction caused by conduit roughness is generally calculated as:

$$H_f = \left(\frac{29\, n^2\, L}{R^{1.33}}\right)\frac{v^2}{2g} \qquad (8\text{-}1)$$

where.:

H_f = Energy loss due to friction, in feet

n = Manning's roughness coefficient

L = Conduit length, in feet

R = Hydraulic radius of conduit, in feet

v = Average velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

Entrance, Exit, Expansion, Contraction, and Bend Losses
Head losses due to pipe form conditions are generally calculated as:

$$H_L = K\left(\frac{v^2}{2g}\right) \tag{8-2}$$

where:

H_L = Head loss due to pipe form conditions, in feet

K = Loss coefficient for pipe form conditions

v = Average velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

The loss coefficient, K, is different for each category of pipe form loss and should be based on operating characteristics of the system being considered. Values for the entrance loss coefficient are the same as those developed for culverts (see Chapter 6). The exit loss coefficient is generally assigned a value of l. Expansion and contraction loss coefficients for circular pipes can be selected based on the data from Brater and King (1976) presented in Tables 8-1 and 8-The bend loss coefficient for storm sewer systems can be evaluated using Figure 8-4, which provides various relationships between the angle of a bend and the loss coefficient. Relationships are presented for bends at manholes with and without deflectors, and for curved drain alignments with r/D values of 2 and greater than or equal to 6 (r = radius of bend and D = pipe diameter).

Junction and Manhole Losses

If losses associated with junctions and manholes are evaluated, procedures presented, in a report by the University of Missouri (1958) or Marsalek (1985) shall be used. Although details of the procedures are not duplicated, the implications of laboratory test results are discussed below and head loss coefficients for typical manholes and junctions are presented in Table 8-3.

For straight flow-through conditions, the pipes should be positioned vertically between the limits of inverts aligned or crowns aligned. An offset in the plan is allowable, provided that the projected area of the smaller pipe falls within that of the larger. It is most effective to align the pipe inverts, as the manhole bottom will then support the bottom of the jet issuing from the upstream pipe.

When two laterals intersect at a manhole, pipes should not be oppositely aligned, because the jets could impinge upon each other. If directly opposing laterals are necessary, the installation of a deflector (as shown in Figure 8-5) will significantly reduce losses. The research conducted on this type of deflector is limited to the ratios of outlet pipe to lateral pipe diameters equal to 1.25. In addition, lateral pipes should be located such that their centerlines are separated laterally by at least the sum of the two lateral pipe diameters.

Figure 8-5 depicts several types of deflectors that can be efficient in reducing losses at junctions and bends for full flow conditions. As a contrast, several inefficient manhole shapes are shown in Figure 8-6. Several of these inefficient devices would appear to be improvements, indicating that special shapings deviating from those in Figure8-5 should be used with caution.

8.4.3 OPEN CHANNEL FLOW

Under non-pressure conditions, the capacity of closed conduit can be analyzed by applying Manning 's Equation to evaluate frictional loses for uniform flow. As shown in Figure 8-1, the hydraulic grade line is the free water surface elevation and is parallel to the energy grade line under uniform flow conditions.

For circular conduits flowing full, Manning's Equation can be expressed as:

$$v = \frac{0.592}{n} d_s^{2/3} S_o^{1/2} \qquad (8\text{-}3)$$

or

$$Q = \frac{0.465}{n} d_s^{8/3} S_o^{1/2} \qquad (8\text{-}4)$$

where:

v = Full flow velocity, in feet/second

Q = Design discharge in cfs

n = Manning’s roughness coefficient

d_s = Diameter of the circular conduit, in feet

S_o = Pipe slope, in feet/foot

Non-circular and non-full flow conditions can be evaluated using the standard form of Manning's Equation discussed in Chapter 5.

Given the appropriate peak runoff rate for the design point in question, the conduit is sized to carry this peak rate as an open channel using Manning's Equation. For a condition in which pressure is allowed to develop in storm sewers with a design based on open channel flow conditions, the design capacity of the system will be greater than that determined using Equation 8-4 and can be evaluated as discussed below.

8.4.4 PRESSURE FLOW

If the hydraulic grade line, as illustrated in Figure 8-2 can be increased above the crown of the pipe, pressure flow occurs. The capacity of storm sewers designed to operate under pressure flow conditions can be sized using inlet and outlet control nomographs developed for the evaluation of culverts (see Chapter 6). A more general procedure involves the application of the energy and continuity equations, which can be developed to consider unsteady flow conditions.

The energy equation between upstream and downstream locations can be evaluated by considering velocity head, pipe form, and friction losses, expressed as:

$$H = H_v + H_L + H_f \tag{8-5}$$

or

$$H = \left(1+K_L + \frac{29n^2L}{R^{1.33}}\right)\frac{v^2}{2g} \tag{8-6}$$

where:

- H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, in feet.
- H_v = Velocity head, in feet
- H_L = Head loss due to pipe form conditions, in feet
- H_f = Head loss due to friction, in feet
- K_L = Loss coefficient for pipe form losses
- n = Manning's roughness coefficient
- L = Length of storm sewer segment, in feet
- R = Hydraulic radius, in feet
- v = Average velocity of flow, in feet/second
- g = acceleration due to gravity, 32.2 feet/second^2

If H can be determined, the storm sewer capacity is calculated by rearranging Equation 8-6 as follows:

$$v = \left\{2gH \div \left(1 + K_L + \frac{29n^2L}{R^{1.33}}\right)\right\}^{1/2} \tag{8-7}$$

or

$$Q = A\left\{2gH \div \left(1 + K_L + \frac{29n^2L}{R^{1.33}}\right)\right\}^{1/2} \tag{8-8}$$

Table 8-1

VALUES OF K_2 FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN EXPANSION IN PIPES, FROM THE FORMULA

$$H_2 = K_2(v_1^2/2g)$$

d_2/d_1 = Ratio of larger pipe to smaller pipe

v_1 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_1, in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

Reference: Brater and King (1976).

Table 8-2

VALUES OF K_3 FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN CONTRACTION IN PIPES, FROM THE FORMULA

$$H_3 = K_3(v_2^2/2g)$$

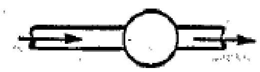
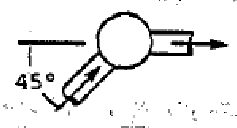
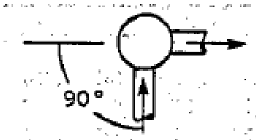
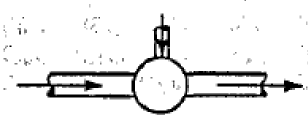
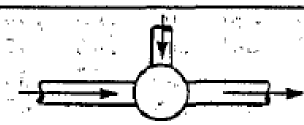
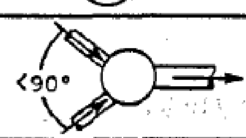
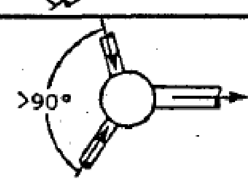
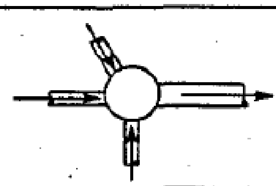
d_2/d_1 = Ratio of larger to smaller diameter

v_2 = Velocity in smaller pipe

d_2/d_1	Velocity, v_2, in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04.	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41.	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

Reference: Brater and King (1976).

Table 8-3
HEAD LOSS COEFFICIENTS FOR MANHOLES/JUNCTIONS

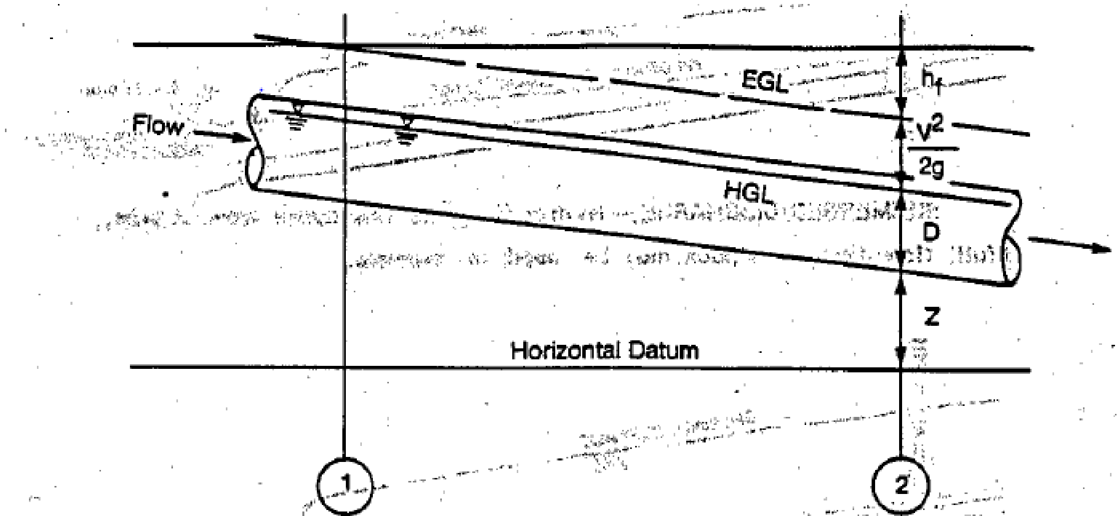
Single Pipe Junctions	
Type of Manhole/Junction	**Head Loss Coefficient (K)**
Trunkline only with no bend at junction 	0.5
Trunkline only with 45° bend at junction 	0.6
Trunkline only with 90° bend at junction 	0.8
Multiple Pipe Junctions	
Type of Manhole/Junction	**Head Loss Coefficient (K)**
Trunkline with one small lateral 	0.6
Trunkline with one large lateral 	0.7
Two roughly equivalent entrance lines with angle of <90° between lines 	0.8
Two roughly equivalent entrance lines with angle of >90° between lines 	0.9
Three or more entrance lines 	1.0

Reference: Golding (1987).

Note: Above values of K are to be used to estimate energy or head losses through surcharged junctions/manholes in pressure flow portions of a storm sewer system. The energy loss equation is

$$hj(ft)=K\frac{[v(ft/sec)]^2}{64.4}$$

with v = larger velocity in main entrance or exit line of junction/manhole.



FIGUF
Open Channel Flow in a Closed Conduit

Z = Distance above horizontal datum
D = Depth of flow
P = Pressure head
$\frac{V^2}{2g}$ = Velocity Head
h_f = Friction loss between Section 1 and Section 2
EGL = Energy Grade Line
HGL = Hydraulic Grade Line

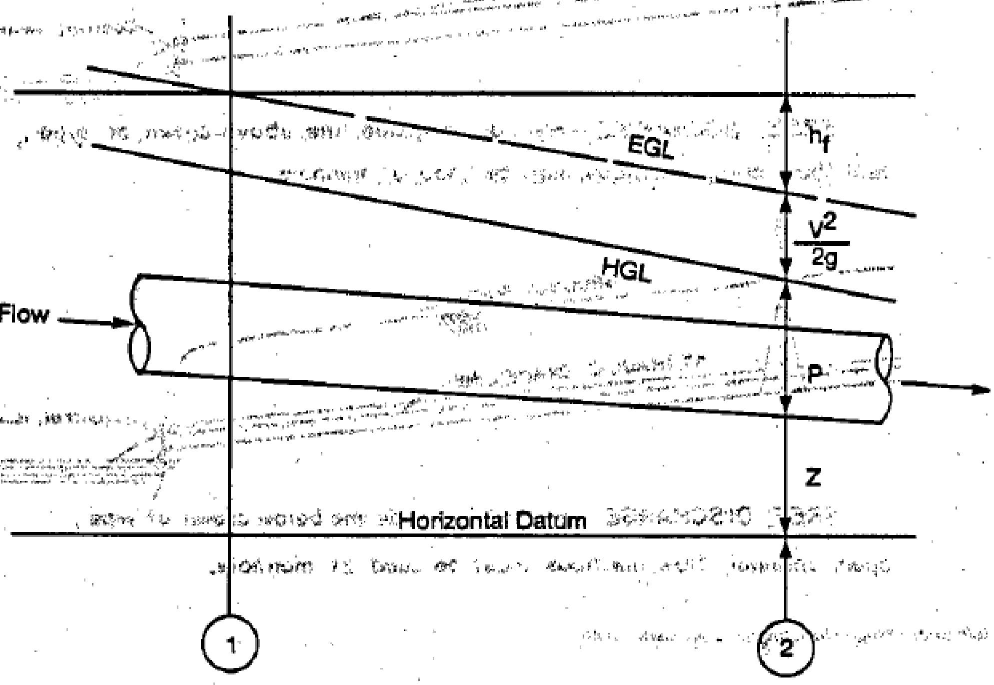
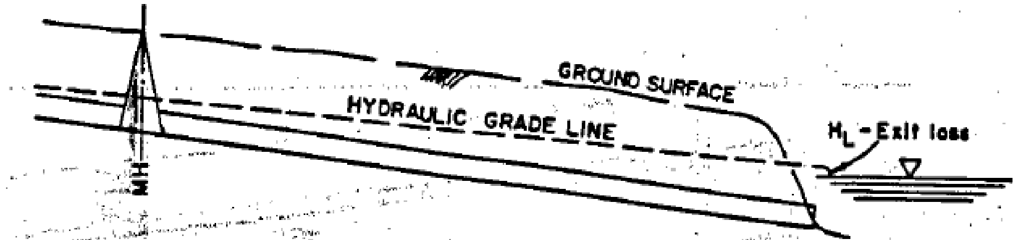
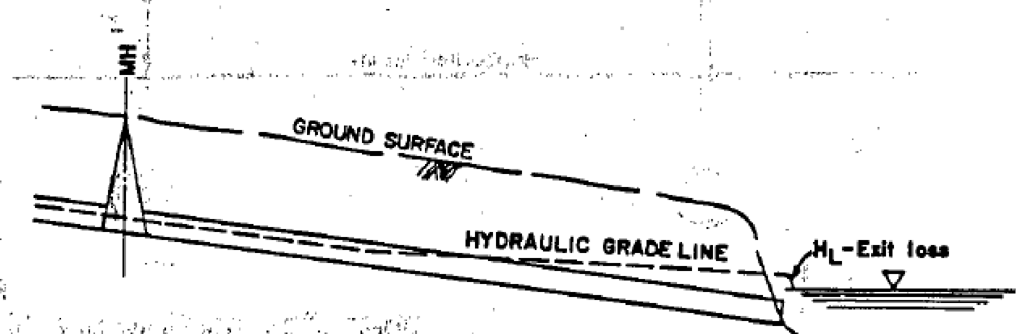


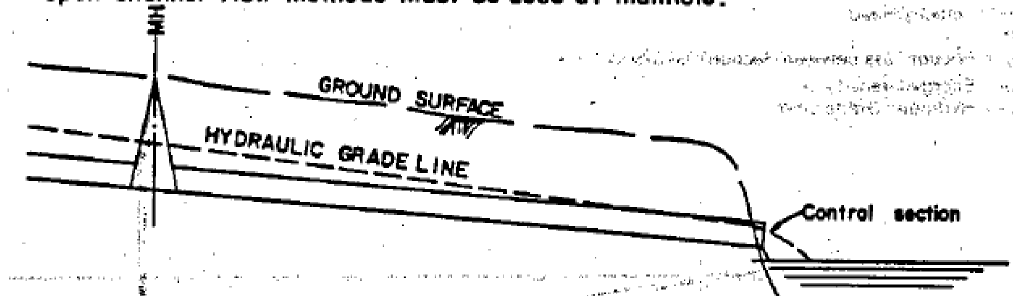
FIGURE 8-2
Pressure Flow in a Closed Conduit



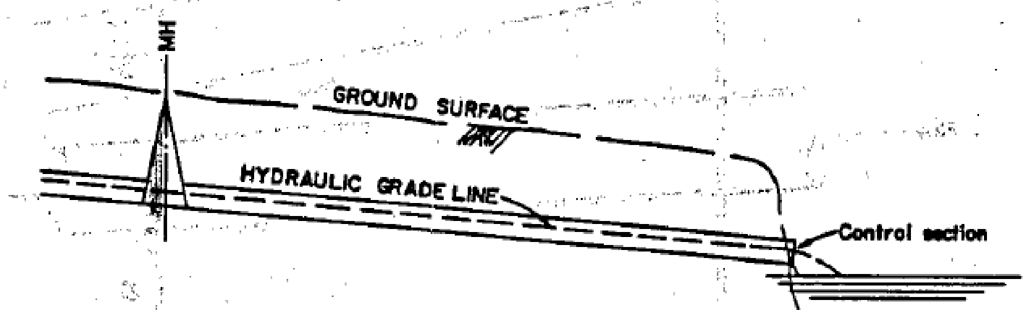
SUBMERGED DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



SUBMERGED DISCHARGED – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.



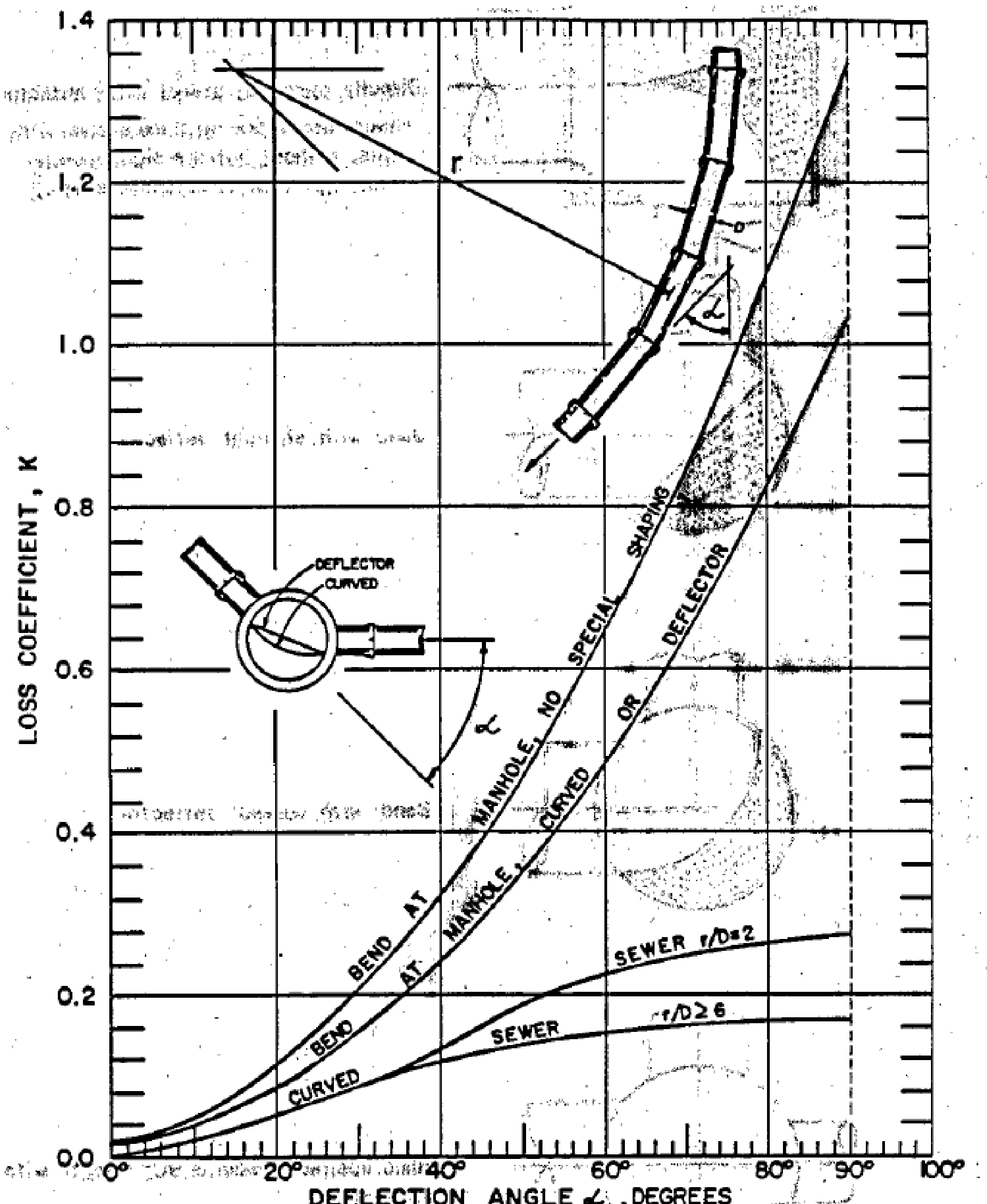
FREE DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



FREE DISCHARGE – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

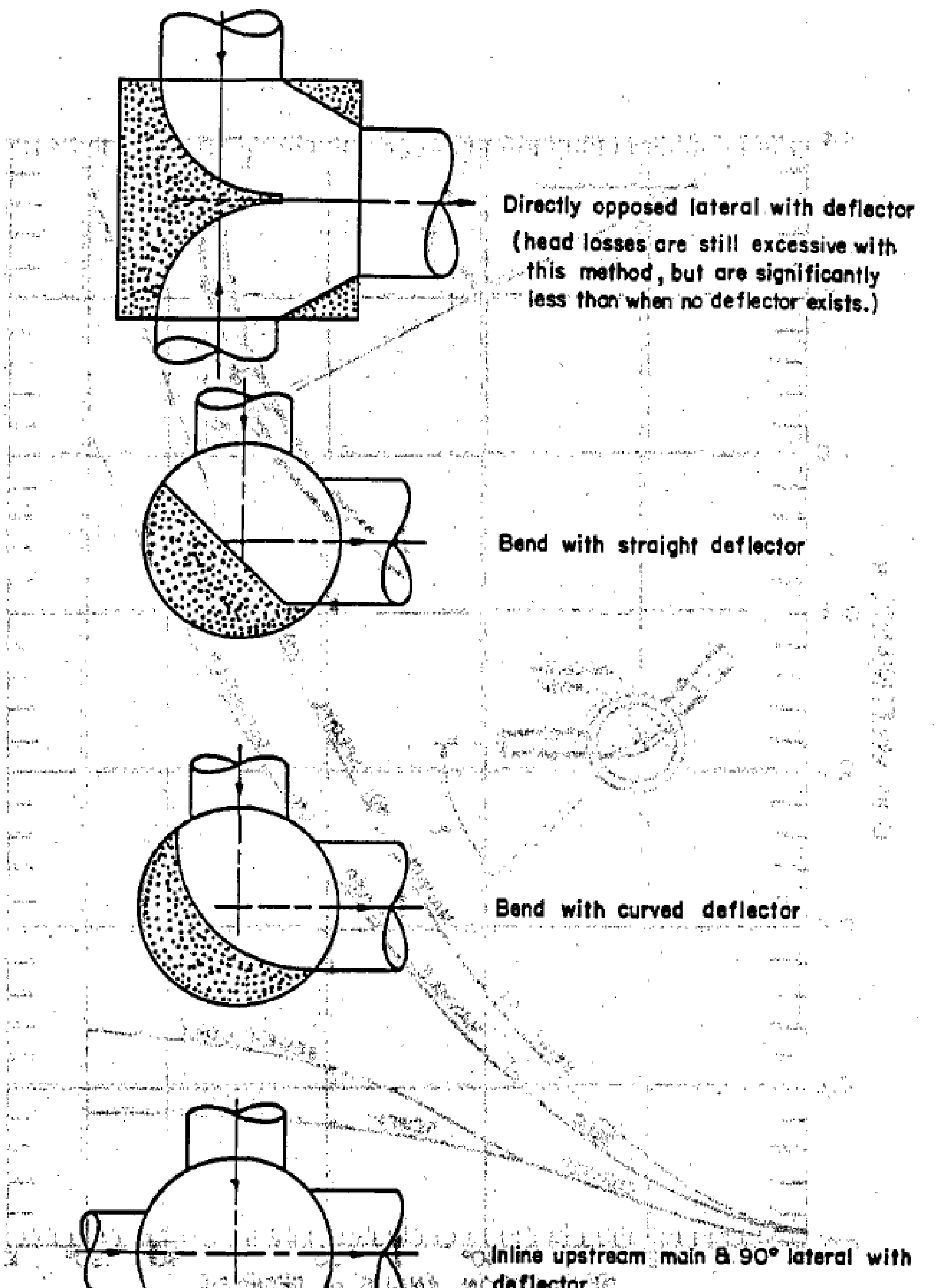
Reference: Wright-McLaughlin Engineers (1969).

FIGURE 8-3
Determination of Pressure vs. Open Channel Flow Conditions in Storm Sewer Systems



ence: *Wright-McLaughlin Engineers (1969).*

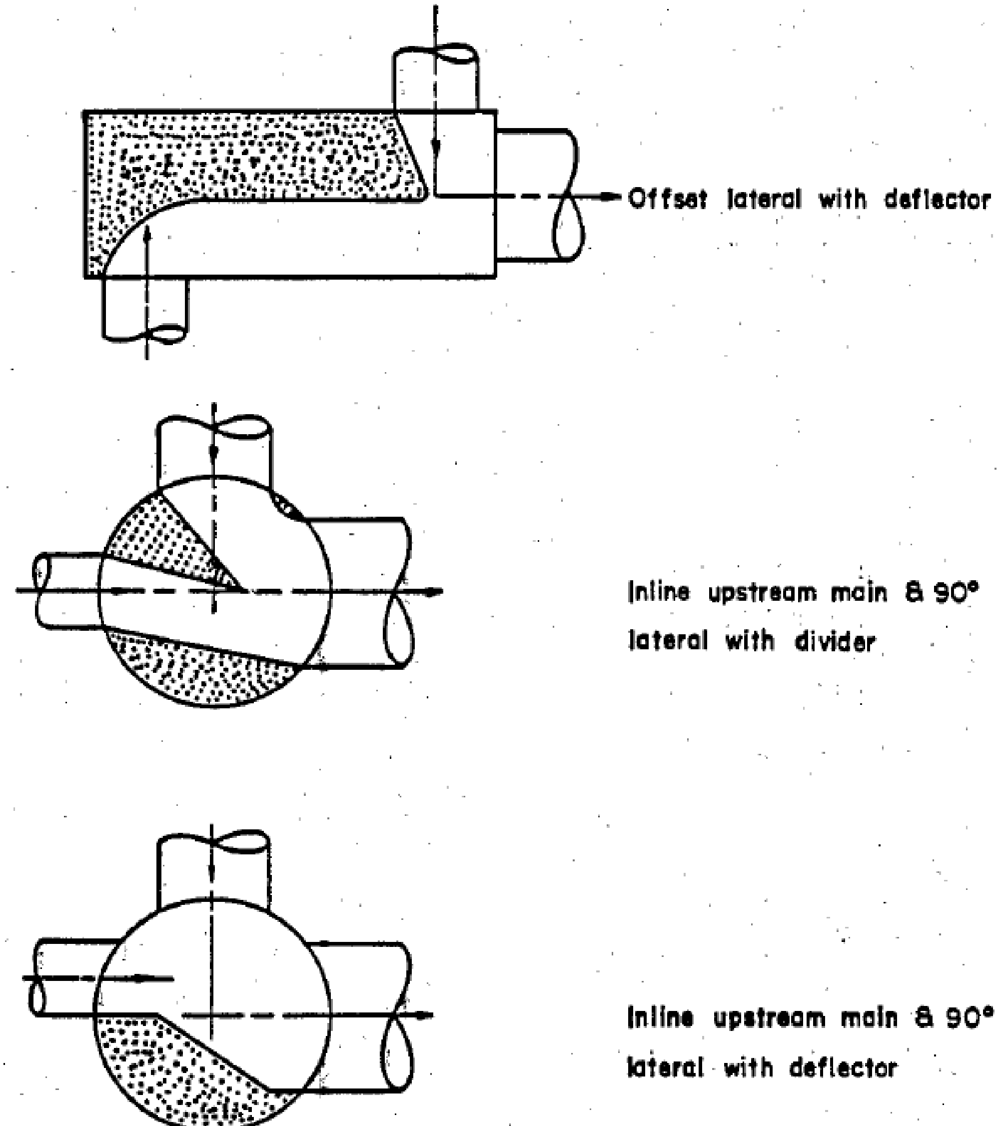
FIGURE 8-4
Storm Sewer Bend Loss Coefficient



These methods of shaping the interior of a manhole were found efficient in University of Missouri (1958) tests.

Reference: Wright-McLaughlin Engineers (1969).

FIGURE 8-5
Efficient Manhole Shaping



These methods of shaping the interior of a manhole were found efficient in University of Missouri (1958) tests, either due to increased head loss or tendency to plug with trash.

Reference: Wright-McLaughlin Engineers (1969).

FIGURE 8-6
Inefficient Manhole Shaping

CHAPTER 9
POST-CONSTRUCTION STORMWATER MANAGEMENT

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CHAPTER 9
POST-CONSTRUCTION STORMWATER MANAGEMENT

9.1 TERMINOLOGY

For stormwater management purposes, retention refers to storage without access to a positive outlet, while detention facilities offer temporary storage accompanied by controlled release of the stored water. Wet detention typically has a pool of water below the outlet elevations; dry detention is typically placed with the basin bottom above the seasonal high water table. Retention and detention can be used separately or together in storage basins as site conditions and management objectives require.

Low Impact Development (LID) and Green Infrastructure (GI) refer to post-construction measures that mimic natural conditions and encourage infiltration of stormwater, which results in the removal of pollutants from runoff, in addition to reduction to runoff volumes and peak runoff rates.

9.2 FUNDAMENTALS

The purpose of post-construction stormwater management is to reduce the discharge of pollutants and mimic the pre-construction hydrology. Post-construction stormwater measures are intended to:

1. Ensure that post-construction runoff mimics pre-construction hydrology of the site;
2. Remove suspended solids and other pollutants associated with activities occurring during and after development;
3. Encourage the use of Low Impact Development and Green Infrastructure techniques in managing post-construction stormwater runoff;
4. Decrease the erosive potential of increased runoff volumes and velocities associated with development; and,
5. Preserve natural systems including in-stream habitat, riparian areas, and wetlands.

9.3 DESIGN CRITERIA

The standard for determining the water quality treatment volume is the post-construction runoff volume resulting from the 1.1-inch rainfall over a 24-hour period, preceded by a 72-hour antecedent dry period. The treatment volume must be 100% managed on-site via infiltration, evapotranspiration, reuse, or treatment using post-construction stormwater controls, including LID practices where practicable.

The standard for determining the requirement for retention and/or detention facilities shall be that the calculated peak rate of stormwater runoff shall be no greater after development of a site than before development of a site (See Sections 2.1.1 and 3.2.5).

Each development plan will be reviewed by the Director of City Engineering to determine if detention and/or retention will be required. When required, design criteria for detention/retention facilities shall include the following items:

1. Release rate
2. Detention volume
3. Grading and depth requirements
4. Outlet works

9.3.1 WATER QUALITY TREATMENT AND RUNOFF REDUCTION

Design strategies to reduce post-construction runoff and/or provide water quality treatment may include, but are not limited to:

1. Minimizing impervious surfaces
2. Providing vegetated buffers
3. Detention ponds
4. Retention ponds
5. Bioretention
6. Constructed stormwater wetlands
7. Grassed swales, infiltration swales, and wet swales
8. Rain gardens
9. Rainwater harvesting
10. Permeable pavement

Design criteria for LID practices should be determined from the most recent version of the *Low Impact Development Handbook for the State of Alabama* (Alabama LID Handbook). LID practices shall be designed to drain within 72 hours.

To assist with selection and design of appropriate LID practices, infiltration testing must be performed in the location of any proposed infiltration practice. Infiltration testing must be performed in accordance with ASTM 3385 (Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer) or an alternative method approved by the Director of Engineering. Infiltration practices must include an underdrain when the infiltration rate is less than 0.5 inch per hour.

If site conditions will not support the inclusion of LID practices, management of the water quality treatment volume may be incorporated in detention ponds by designing the pond to collect the water quality volume and releasing that runoff over a minimum 24-hour and a maximum 72-hour period.

9.3.2 RELEASE RATE

Release rates for managing stormwater quantity by detention shall be based on limiting peak runoff rates, to match the following value:

The post development and pre-development calculated peak rate of stormwater resulting from the two, five and ten-year return period, twenty-four (24) hour duration, type II storm distribution (as defined by the Natural Resources Conservation Service, U.S, Department of Agriculture) shall be equal.

9.3.3 DETENTION VOLUME

Detention volume shall be adequate to provide attenuation of the post-development peak discharge rates to the allowable release rates set according to provisions in Section 9.3.2.

Routing calculations shall be consistent with procedures in Section 9.6. If siltation during construction causes loss of detention volume, design dimensions shall be restored prior to submitting as-built certification. Detention volume shall be drained within 72 hours.

9.3.4 GRADING AND DEPTH

To obtain sufficient storage volume, the construction of dry detention/retention facilities usually requires either excavation or the placement of earthen embankments. If vegetated embankments are used, they shall be less than 10 feet in height and shall be placed with side slopes no steeper than 3:1 (horizontal: vertical). Slopes for riprapped earthen embankments should be no steeper than 3:1. A slope stability evaluation is generally appropriate for embankment configurations with impoundment depths greater than 3 feet.

The maximum depth of stormwater detention/retention facilities should be consistent with safety and aesthetic considerations for the system. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage the growth of attached weeds (without creating undue potential for anaerobic bottom water) should be considered. A depth of from 6 to 8 feet is generally reasonable.

Other considerations when setting detention/retention facility depths include flood elevation requirements, public safety, land availability, land values, present and future land use, water table fluctuations, soil profile characteristics, maintenance, and freeboard. Aesthetically pleasing features are also an important consideration and can often be provided with little extra cost or loss of capacity. A minimum freeboard of 1 foot above the high water elevation of the 100-yr storm event shall be provided, with higher values considered when addressing special concerns.

9.3.5 OUTLET WORKS

Outlet works shall be designed to prevent overtopping of the embankment based on a 100-year discharge rate considering total watershed development. Outlet works can take the following forms:

1. Drop inlets with pipes
2. Weirs
3. Orifices
4. Curb openings for parking lot storage

The principal outlet shall convey the 10-year discharge rate for total watershed development without allowing flow to enter the emergency outlet. An emergency outlet, often referred to as an emergency spillway, shall be provided to convey the 100-year discharge rate through the spillway while providing a minimum of 1 foot of freeboard before overtopping of the embankment begins. A smooth and stable transition to downstream facilities shall be provided. The sizing of a particular outlet work shall be based on results of hydrologic routing calculations (see Section 9.6), consistent with criteria in Section 9.7.1 and 9.7.2.

9.3.6 WET PONDS

The bottom of a detention/retention pond intended to have a permanent pool of water shall be lined or sealed according to sustain a permanent pool of water at the indicated design depth. Designer shall provide supporting geotechnical and materials information for synthetic liners. Clay liners shall meet the following minimum criteria:

1. Compact the subgrade to 95 percent (Standard Proctor Test) with an optimum moisture content of 2 percent.

2. Place two 6-inch layers of clay (plasticity index greater than 20) above the compacted subgrade. Each 6-inch layer shall be compacted to 95 percent (Standard Proctor Test) with and optimum moisture content of 2 percent.

Side slopes for wet ponds shall be no greater than 3:1 and riprap or engineered slope armoring shall be installed to protect slopes from wave attack. A five foot wide bench shall be constructed at a depth of four feet below normal pool elevation with a 4:1 maximum slope at a depth. Riprap shall be at least two feet below the control elevation and up to the 100-year flood elevation plus one foot of freeboard.

9.4 GENERAL PROCEDURE

The following three relationships shall be considered when sizing a stormwater detention facility:

1. Inflow hydrograph for an appropriate design storm (see Chapter 4)

2. Stage-storage curve for basin (see Figure 9-1 for an example)

3. Stage-discharge curve for basin outlet control structure (see Figure 9-2 for an example)

In most cases, a trial and error design procedure is required, because only the inflow hydrograph is known. A general procedure for evaluating these variables is presented below:

1. Compute inflow hydrographs for appropriate design storms using procedures form Chapter 4. In most cases, both pre- and post- development hydrographs are required.

2. Perform preliminary calculations to select a reasonable detention storage configuration (see Section 9.6) to handle the hydrographs from Step 1.

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The estimated volume is the maximum storage requirement calculated from Step 2.

4. Size the outlet structure. The estimated peak state will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

5. With inflow hydro-graphs from Step 1, perform routing calculations to check the design using the Storage Indication Method (see Section 9.7). If the routed peak discharges exceed the allowable peak discharges, or if the peak stage varies significantly from the estimated peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

6. If appropriate evaluate the downstream effects of dentition out-flow to ensure that the recession limb of the outflow hydrograph does not cause downstream flooding problems

7. Evaluate the control structure outlet velocity and provide stabilization if necessary.

Because this procedure can involve a significant number or reservoir routing calculations, a computer procedure is useful for conducting final routing computations. Additional information on retention/detention basin design computations can be found in articles by Mason and Rhomberg (1982, 1983) McKinnon (1984), Mein (1980), and Rossmiller (1982).

9.5 OUTLET HYDRAULICS

Sharp-crested weir flow equations for no end contractions two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, and orifices. If culverts are used as outlet works, procedures presented in Chapter 6 should be used to develop stage-discharge data.

9.5.1 SHARP-CRESTED WEIRS-NO END CONTRACTION

A sharp-crested weir with no end contractions is illustrate in Part A of Figure 9-3. The discharge equation for this configuration (Chow, 1959) is expressed as:

$$Q = \left(3.27 + 0.4\ \frac{H}{Hc}\right) L\ H^{1.5} \qquad (9\text{-}1)$$

where:

Q = Discharge, in cfs

H = Head above the weir crest excluding velocity head, in feet (see Figure 9-3, Part C)

H_c = Height of weir crest above channel bottom in feet (see Figure 9-3, Part C)

L = Horizontal weir length, in feet

9.5.2 SHARP-CRESTED WEIRS-TWO END CONTRACTIONS

A sharp-crested weir with two end contractions is illustrated in Part B of Figure 9-3. The discharge equation for this configuration (Chow, 1953) is expressed as:

$$Q = \left(3.27 + 0.4\,\frac{H}{Hc}\right)(L - 0.2h)\,H^{1.5} \qquad (9\text{-}2)$$

where:

Q = Discharge, in cfs

H = Head above the weir crest excluding velocity head, in feet (see Figure 9-3, Part C)

H_c = Height of weir crest above channel bottom, in feet (see Figure 9-3, Part C)

L = Horizontal weir length, in feet

9.5.3 SHARP-CRESTED WEIRS-SUBMERGED DISCHARGE

The effect of submergence on a sharp-crested weir should be considered when applying Equations 9-1 and 9-2. When the tailwater rises above the weir crest elevation, the discharge over the weir will be reduced. To account for the submergence effect, the free discharge obtained by Equation 9-1 or 9-2 should be modified using the equation:

$$Q_s = Q_f\left(1 - \left(\frac{H2}{H1}\right)^{1.5}\right)^{0.385} \qquad (9\text{-}3)$$

where:

Q_s = Submergence flow, in cfs

Q_f = Free flow, in cfs

H_1 = Upstream head above crest, in feet

H_2 = Downstream head above crest, in feet

9.5.4 BROAD-CRESTED WEIRS

The general form of the broad-crested weir equation (Brater and King, 1976) is expressed as:

$$Q = C\ L\ H^{1.5} \qquad (9\text{-}4)$$

where:

Q = Discharge, in cfs

C = Broad-crested weir coefficient

L = Broad-crested weir length, in feet

H = Head above weir crest (measured at least 2.5H upstream of the weir), in feet

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction, and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087 for a broad-crested weir. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional data from Brater and King (1976) on selecting c values for broad-crested roughness and slope on weir discharge can be computed by applying Manning's Equation (see Chapter 5).

9.5.5 V-NOTCH WEIRS

The discharge through a v-notch weir can be evaluated using the following Equation (Brater and King, 1976):

$$Q = 2.5 \tan(r/2)\ H^{2.5} \qquad (9\text{-}5)$$

where:

Q = Discharge, in cfs

r = Angle of v-notch, in degrees

H = Head on vortex of notch, in feet

9.5.6 ORIFICES

The discharge through an orifice can be evaluate using the following equation:

$$Q = CA(2gH)^{0.5} \qquad (9\text{-}6)$$

where:

Q= Discharge, in cfs

C = Orifice coefficient (a value of 0.6 is usually appropriate; see Brater and King [1976] if additional information is desired)

A = Area of orifice, in square feet

q = Acceleration due to gravity, 32.2 feet/second

H = Head above orifice centroid, in feet

9.6 PRELIMINARY DETENTION CALCULATIONS

9.6.1 STORAGE VOLUME

The following steps provide a graphical procedure for developing a preliminary estimate of the storage volume required to meet allowable discharge requirements:

1. Develop an inflow hydrograph and an allowable peak outflow rate, Q_o. Plot the hydrograph on graph paper.
2. On the graph from Step 1, draw an estimated outflow hydrograph with an allowable peak outflow of Q_o occurring on the recession limb of the inflow hydrograph. A simple estimate can be made by drawing a straight line form the point at which discharge begins to Q_o on the recession limb of the inflow hydrograph.

3. Calculate the area bounded by the line from Step 2 and the inflow hydrograph. This area represents a preliminary estimate of the required storage volume. Assuming triangular inflow and outflow hydrographs, a preliminary estimate of the storage volume can be made using the following equation:

$$S = 0.5b\,(Q_i - Q_o) \qquad (9\text{-}7)$$

where:

S = Volume of storage, in cubic feet

Q_i = Inflow peak flow, in cfs

Q_o = Outflow peak flow, in cfs

b = Duration of runoff, in seconds

4. Confirm that the preliminary estimate is adequate, using the Storage Indication Method (see section 9.7.1).

9.6.2 PEAK FLOW REDUCTION

A preliminary estimate of the potential peak flow reduction for a selected detention volume can be obtained as follows:

1. Determine input data, including the peak flow rate of the inflow hydrograph, Qi, the storage volume, S, and the duration of runoff, b.

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation:

$$Q_o = bQ_i - S \qquad (9\text{-}8)$$

where:

Q_o = Outflow peak flow, in cfs

b = Duration of runoff, in seconds

Q_i = Inflow peak flow, in cfs

S = Volume of storage, in cubic feet

3. Confirm that the preliminary estimate is adequate, using the Storage Indication Method (see Section 9.7.1).

9.7 ROUTING CALCULATIONS

The storage Indication Method is recommended for performing reservoir routing calculations for final design of detention facilities.

9.7.1 STORAGE INDICATION METHOD

The following procedure is used to perform a reservoir routing by the Storage Indication Method:

1. Develop an inflow hydrograph, a stage-discharge curve, and a stage storage curve for the proposed detention facilities. Example stage-storage and Figures 9-1 and 9-2 respectively.

2. Select a routing time period, W_t, to provide at least five points on the rising limb of the inflow hydrograph.

3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S_2 + \frac{O_2}{2}Wt$ versus stage. A curve data is presented in Table 9-2 and Figure 9-4.

4. For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1 - \frac{O_1}{2}$ Wt can be determined for the appropriate storage characteristics curve (e.g., Figure 9-4).

5. Determine the value of $S_2 + \frac{O_2}{2}$Wt from the following relationship:

$$S_2 + \frac{O_2}{2}W_t = \left[S_1 - \frac{O_1}{2}W_t\right] + \left[\frac{I_1+I_2}{2}W_t\right] \quad (9\text{-}9)$$

where:

S_2 = Storage volume at time 2, in cubic feet

O_2 = Outflow rate at time 2, in dvs

W_t = Routing time period, in seconds

S_1 = Storage volume at time 1, in cubic feet

O_1 = Outflow rate at time 1, in cfs

I_1 = Inflow rate at time 1, in cfs

I_2 = Inflow rate at time 2, in cfs

6. Enter the appropriate storage characteristics curve (e.g., Figure 9-4) at the value of $S_2 + \frac{O_2}{2}$Wt determined in Step 5 and read off a new depth of water, H_2.

7. Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage discharge curve (e.g., Figure 9-2).

8. Repeat Steps 1 through 7 setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_e, S_2, and H_2, and using new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

9.7.2 EXAMPLE PROBLEM

Example 9-2. Reservoir Routing Using Storage Indication Method

An example application of the Storage Indication Method using data presented in Figure 9-2, 9-2 and 9-4 is presented in Table 9-3. The inflow hydrograph is given in columns 1 and 2 of Table 9-3. The objective is to find the outflow using the Storage Indication Method. A step-by-step discussion of the calculations summarized in Table 9-3 is presented below:

a. Using the data tabulated in Column 2 of Table 0-3, calculated:

$$\left(\frac{I_1 + I_2}{2}\right) W_t$$

And tabulate these values in Column 3 of Table 9-3.

b. Given that $S_1 - \frac{O_1}{2} W_t = 0.05$ acre-foot for $H_1 = 0$ foot, find $S_2 + \frac{O_2}{2} W_t$ by adding 0.05 + 0.01 (Column 5 value plus Column 3 value) and tabulate 0.06 acre-foot in Column 6 of Table 9-3.

c. Enter the $S_2 + \frac{O_2}{2} W_t$ storage characteristics curve in Figure 9-4 and rad the stage at the value of 0.06 acre-foot. This value is 100.10 feet and is tabulated as stage H_2 in Column 7 of Table 9-3.

d. Using the stage of 100.10 feet found in Step 4, enter the stage-discharge curve (Figure 9-2) and find the discharge corresponding to that stage. In this case O is approximately 1 cfs and is tabulated in Column 8 of Table 9-3.

e. Assign the value of H_2 to H_1, find a new value of $S_1 - \frac{O_1}{2} W_t$ from Figure 9-4, and repeat the calculations for Steps 2, 3, and 4. Continue repeating these calculations until the entire inflow hydrograph has been routed through the storage basin.

f. The Storage Indication Method calculations give a peak outflow of 220 cfs. The inflow hydrograph has a peak rate of 360 cfs, so a reduction of approximately 40 percent is calculated.

9.8 PROTECTIVE TREATMENT

Protective treatment may be required to prevent entry to facilities that present a public safety hazard. A six (6) foot high security type chain link fence shall be provided for detention areas where one or more of the following conditions exist:

1. Rapid stage changes that would make escape practically impossible for small children

2. Water depths that either exceed 2.5 feet for more than 24 hours or are permanently wet and have side slopes steeper than 4:1 (horizontal: vertical)

3. Passage through the detention area of a low-flow watercourse or ditch with a depth greater than 5 feet or a flow velocity greater than 5 feet per second

4. Side slopes that equal or exceed 1.5:1 (horizontal: vertical)

5. Depth that equals or exceeds 3 feet and maintenance will be the responsibility of the City of Huntsville.

    a. Guards or grates may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

    b. Fencing should be considered for dry retention areas with design depths in excess of 2.5 feet for 24 hours, unless the area is within a fenced, limited access facility.

Table 9-1
BROAD-CRESTED WEIR COEFFIENT C VALUES
AS A FUNCTION OF WEIR CREST BREADTH AND HEAD

Measured Head, H[a] (ft)	Weir Crest Breadth (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	2.65	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	2.72	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

Reference: Brater and King (1976).

[a]Measured at least 2.5H upstream of the weir.

Table 9-2
EXAMPLE TABULATION OF STORAGE CHARACTERISTICS CURVES

Stage (ft above NGVD)	Storage[a] (acre-ft)	Discharge[b] (cfs)	(acre-ft/hr)[d]	$S_2 + \frac{O_2}{2}\Delta t$[c] (acre-ft)	$S_2 + \frac{O_2}{2}\Delta t$[c] (acre-ft)
100	0.05	0	0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	7.85	2.15	3.45
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10.0	275	22.73	8.11	11.89

[a]Obtained from Figure 9-1.

[b]Obtained from Figure 9-2.

[c]Δt = 10 min = 0.167 hour.

[d]1cfs = 0.0826 acre-ft/hr.

Table 9-3
STORAGE INDICATION METHOD-EXAMPLE CALUCLATIONS

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time, t (min)	Inflow, I (cfs)	$\frac{(I_1 + I_2)\,\Delta t}{2}$ (acre-ft)	Upstream Head, H_1 (ft)	$S_1 - \frac{O_1}{2}\Delta t$ (acre-ft)	$S_2 + \frac{O_2}{2}\Delta t$ (acre-ft)	Downstream Head, H_2 (ft)	Outflow, O (cfs)
0	0						
10	2	0.01	0.0	0.05	0.06	100.10	1
20	27	0.20	100.100	0.06	0.26	101.10	16
30	130	1.08	101.10	0.21	1.29	102.20	41
40	300	2.96	102.20	0.61	3.57	104.10	100
50	360	4.55	104.10	2.20	6.75	105.60	175
60	289	4.47	105.60	4.40	8.87	106.25	217
70	194	3.33	106.25	5.80	9.13	106.30	220
80	133	2.25	106.30	5.90	8.15	106.05	205
90	91	1.54	106.05	5.30	6.84	105.65	177
100	61	1.05	105.65	4.50	5.55	105.10	147
110	37	0.67	105.10	3.60	4.27	104.50	116
120	20	0.39	104.50	2.70	3.09	103.80	87
130	11	0.21	103.80	1.90	2.11	103.05	64
140	5	.11	103.05	1.18	1.30	102.25	43
150	1	0.04	102.25	0.63	0.67	101.40	22
160	0	0.0	101.40	0.35	0.35	100.70	10

Note:

$S_2 + \frac{O_2}{2}\Delta t = [S_1 - \frac{O_1}{2}\Delta t] + [\frac{I_1+I_2}{2}\Delta t]$ Equation 9-9
(column 6) = (column 5) + (column 3)

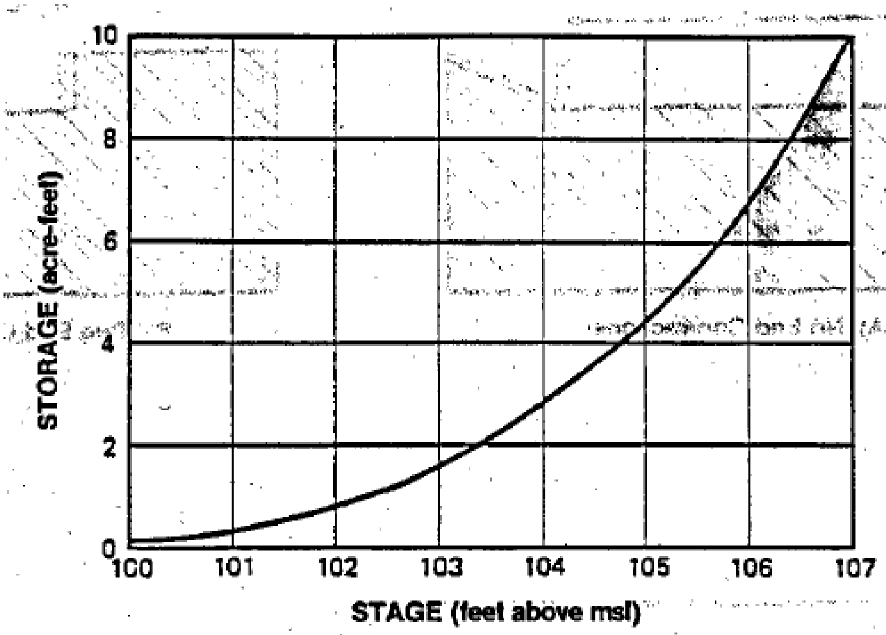


FIGURE 9-1
Example Stage-Storage Curve

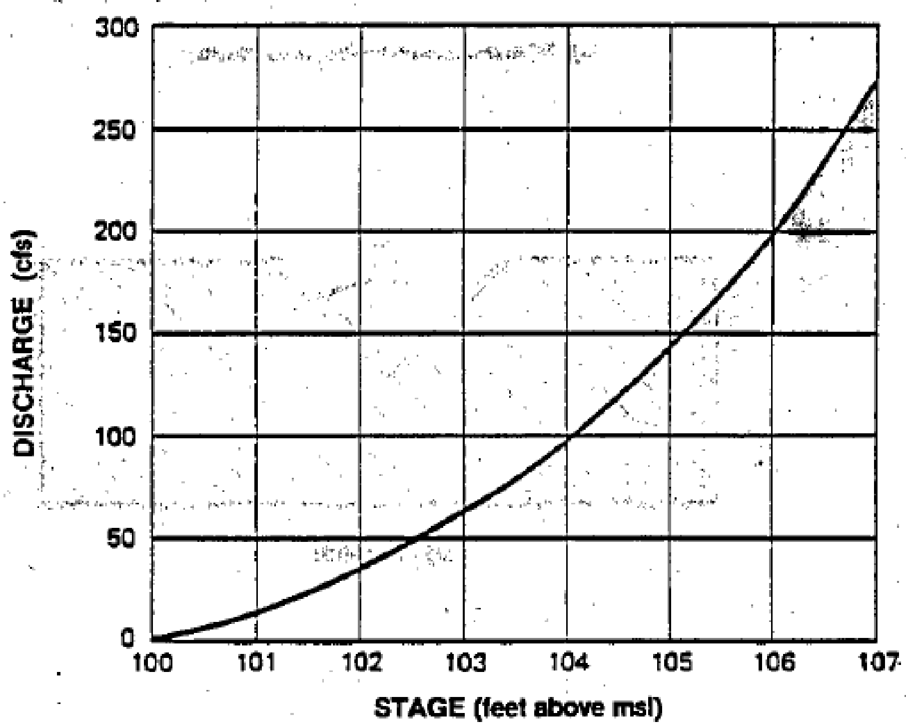


FIGURE 9-2
Example Stage-Discharge Curve

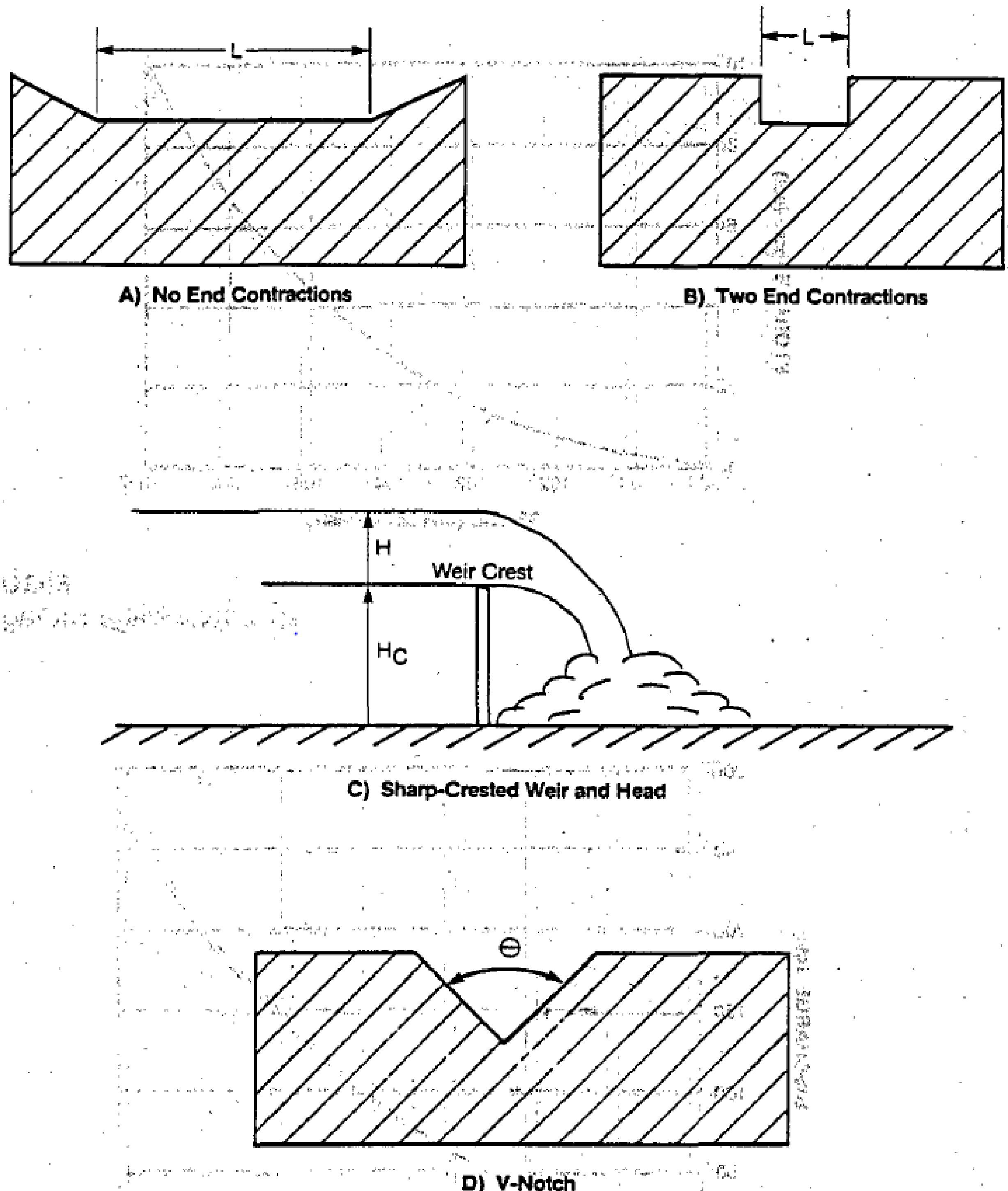
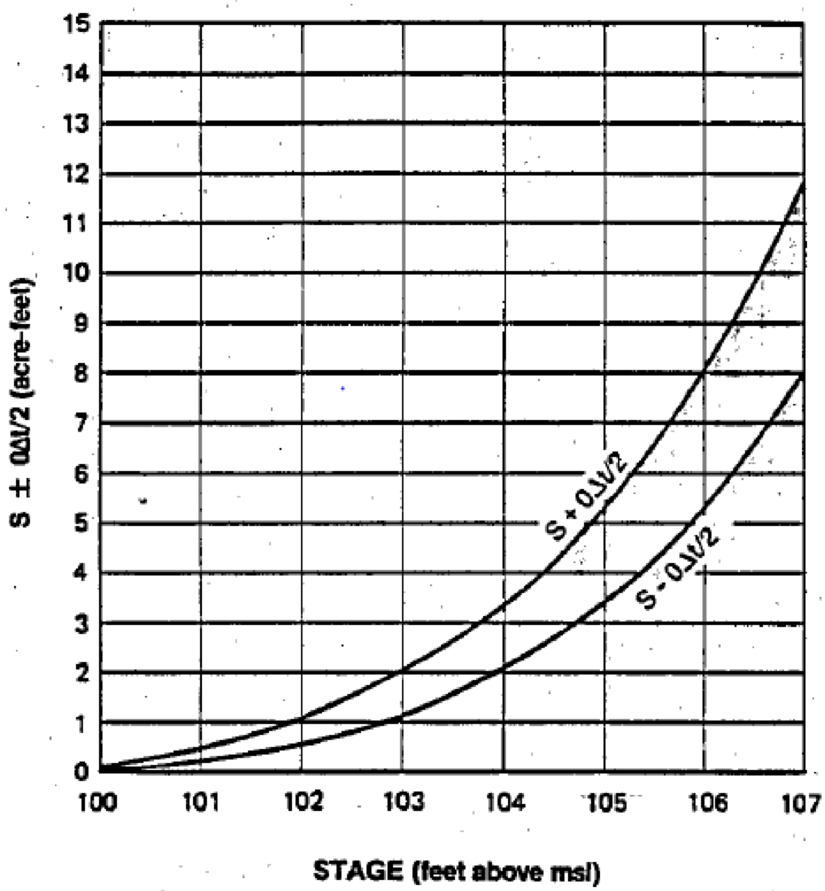


FIGURE 9-3
Illustrations of Weir Flow Control Structures



NOTE: Data presented in Table 9-2.

FIGURE 9-4
Storage Characteristics Curves

CHAPTER 10
EROSION AND SEDIMENT CONTROL

CHAPTER 10
EROSION AND SEDIMENT CONTROL

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CHAPTER 10
EROSION AND SEDIMENT CONTROL

The erosion and sediment control requirements established in this manual are consistent with those required by the Alabama Construction NPDES General Permit ALR100000 (Alabama CGP) and by the Huntsville MS4 Storm Water Management Program Plan (SWMPP). Where the requirements established herein conflict with the requirements of the Alabama CGP or Huntsville SWMPP, the more restrictive requirements shall take precedence.

10.1 EROSION AND SEDIMENT CONTROL PLANS

10.1.1 SUBMITTAL REQUIREMENTS

As discussed in Sections 2.4.2 and 3.2.6, the submittal of an Erosion and Sediment Control Plan (ESC Plan) is required for all land alteration activities, unless specifically exempted. ESC Plans are required in order to obtain a grading permit, building permit, or subdivision approval.

The Director of City Engineering shall require the following information on the ESC Plan:

1. A complete plan of the proposed development at a scale no less than 1 inch = 100 feet. This plan is to include existing and proposed contours at intervals no greater than 2 feet with at least one benchmark required.

2. Existing and proposed buildings, impervious surfaces, and drainage structures, including inlets, catch basins, manholes, junction boxes, driveway pipes, culverts, cross drains, headwalls, outlet facilities, and sanitary sewers.

3. Any proposed drainage improvements, such as channels, ditches, drainage pipes, and structures.

4. All fill areas indicated as such, with the limits and elevations indicated.

5. Drainage arrows indicating the existing and proposed direction of runoff throughout the plan.

6. Locations of existing and proposed locations where stormwater runoff will exit the site (stormwater outfalls).

7. Temporary erosion and sediment control measures to be implemented during construction, including an implementation schedule.

8. Locations of construction entrances and exits, as well as temporary access drives or haul roads.

9. Locations of designated fueling areas and concrete washout areas, and the associated containment measures.

10. Measures the builder or contractor will use to keep construction materials, including dirt, mud, and debris, off the streets.

11. Locations of proposed vegetative buffers and vegetated areas to be preserved, as well as methods to delineate such areas during construction.

12. Proposed construction phasing, with total disturbance for each phase identified in acres.

10.1.2 BASIC PRINCIPLES

The design of erosion and sediment control systems involves the application of common-sense planning, scheduling, and control actions that will minimize the adverse impacts of soil erosion, transport, and deposition. Five basic principles govern the development and implementation of a sound erosion and sediment control plan:

1. Plan the project to take advantage of the topography, soils, waterways, and natural vegetation at the site.

2. Expose the smallest practical area for the shortest possible time

3. Apply on-site erosion control measures to reduce the gross erosion from the site.

4. Place sediment control measures to trap eroded material and prevent off-site damage.

5. Implement a continuous maintenance and inspection program.

The practices considered in this chapter are classified as either erosion control or sediment control. In general, erosion control practices are designed to prevent soil particles from being detached, whereas sediment control practices prevent the detached particle from leaving the site or from entering a receiving water. Sediment control measures are generally structural in nature; erosion control measures can be either structural (e.g., diversions) or non-structural (e.g., mulches).

Planned erosion and sediment control measures should be designed in accordance with the most recent version of the *Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas* (Alabama Handbook) or the Alabama Department of Transportation (ALDOT) Special Drawings. Additional erosion and sediment control measures not included in the Alabama Handbook or ALDOT Special Drawings may be designed and certified by an individual meeting one or more of the following requirements:

1. Professional Engineer registered in the State of Alabama
2. Alabama Natural Resources Conservation Service professional designated by the State Conservationist

3. Certified Professional in Erosion and Sediment Control (CPESC)
4. Registered landscape architect
5. Registered Land Surveyor
6. Professional Geologist
7. Registered Forester
8. Registered Environmental Manager as determined by the National Registry of Environmental Professionals (NREP)
9. Certified Professional Soil Scientist (CPSSc) as determined by ARCPACS

10.2 ALABAMA CONSTRUCTION GENERAL PERMIT

The Alabama Construction NPDES General Permit ALR100000 (Alabama CGP) applies to the following types of construction activity:

1. Land disturbance equal to or greater than one (1) acre

2. Land disturbance involving less than one (1) acre, but which is part of a common plan of development or sale equal to or greater than one (1) acre

The Alabama CGP specifically exempts agricultural and silvicultural practices.

When calculating land disturbance to determine applicability of the Alabama CGP, construction specifically includes:

1. Land disturbance associated with or resulting from building, excavation, land clearing, grubbing, placement of fill, grading, blasting, and reclamation
2. Areas in which construction materials are stored in association with a land disturbance or handled above ground
3. Construction site vehicle parking
4. Equipment or supply storage areas
5. Material stockpiles
6. Temporary office areas
7. Access roads

10.2.1 <u>SUBMITTAL REQUIREMENTS</u>

The engineer of record is responsible for determining if the site is in a priority watershed. Qualifying land disturbance activities must include the following in the application for a grading permit or subdivision approval:

1. Construction Best Management Practices Plan (CBMPP), as applicable
2. Proof of NPDES permit coverage
3. Proof of eNOI coverage

The narrative CBMPP must be prepared in accordance with the most recent version of the Alabama CGP and the *Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

Proof of NPDES permit coverage shall consist of the ADEM cover letter stating that coverage under the Alabama CGP has been granted and identifying the permit number and effective date of coverage.

10.3 EROSION CONTROL

The ESC Plan shall effectively control and reduce soil erosion from disturbed and filled land. At a minimum, the ESC Plan shall include appropriate measures from the following categories:

1. Preservation of existing vegetation, including the preservation or creation of vegetated buffers at surface waters

2. Surface stabilization, including, but not limited to, mulches, seeding and vegetation, chemical binders and tacks, synthetic fabrics, and riprap

3. Runoff control, including, but not limited to, diversions, slope and down drains, level spreaders, check dams, and outlet protection

4. Exposure scheduling, including planned phasing

10.3.1 TEMPORARY AND PERMANENT CONSIDERATIONS

Construction erosion control measures are usually temporary, intended to function for only the duration of construction. To the maximum extent possible, surface stabilization measures shall provide permanent protection once construction is complete. In addition, the layout for temporary runoff control measures shall be consistent with the layout of permanent drainage facilities.

10.3.2 PRESERVATION OF EXISTING VEGETATION

Where practical, existing vegetation and trees should be preserved and protected. The ESC Plan must clearly identify vegetated areas that will be preserved, as well as the measures proposed to identify the protected areas during construction.

A vegetated buffer must be created or maintained around surface waters adjacent to the site or contained within the site boundary. The buffer must be a minimum of 25 feet in width and should be preserved between the top of the stream bank and the disturbed area. The planned buffer must be clearly identified on the ESC Plan, as well as the measures proposed to delineate the buffer during construction.

Where a 25-foot vegetated buffer is not feasible, the ESC Plan must include additional erosion and sediment controls that will achieve a sediment load reduction equivalent to a 25-foot buffer.

10.3.3 SURFACE STABLILIZATION

Surface stabilization measures will be required when land is cleared and existing conditions of the soil surface are altered. Surface stabilization control measures include mulches, seeding and vegetation, chemical binders or tacks, erosion control blankets, and riprap.

Temporary and permanent surface stabilization must be undertaken using the procedures included in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Standard Specifications.

Additionally, the following Alabama CGP requirements must be incorporated into the ESC Plan:

1. Temporary stabilization must be initiated immediately on any disturbed area not undergoing active construction for more than 13 calendar days.

2. Final stabilization must be initiated immediately when any land disturbance activities have permanently ceased on any portion of the site.

Mulches (Temporary)

A mulch is a layer of material applied to the soil surface for temporary stabilization and to help establish plant cover by holding in moisture and preventing seed loss. Common types of mulching material include straw or wood chips. Hydraulic Erosion Control Products (HECPs) may also be used. The choice of mulch material should be based on soil conditions, season, type of vegetation, and the size of the application area. Mulches are not intended for use in swales or other areas of concentrated flow. Guidelines for mulch selection and application are included in the Alabama Handbook.

Mulches are generally practical for 6 months or less on graded or cleared areas when the growing season is not suitable to produce an erosion-resistant cover from seedings. Final grading is not required before mulching, but mulch may be applied after final grade is reached. Any structural erosion control features should be installed prior to mulching.

Seeding and Vegetation (Temporary and Permanent)

Surface stabilization by vegetation includes temporary seeding, permanent seeding, sod, vines, shrubs, and trees. Sod is preferred in situations where immediate protection is required, such as in swales or ditches. The type of seeding and vegetation, application rates, site preparation, and fertilizer and water requirements should be consistent with the growing season and duration of protection required.

Mulching should be employed along with all seeding operations. To establish permanent vegetation, soil testing results should be evaluated by an agronomist to determine soil treatment requirements for parameters such as pH, nitrogen, phosphorus, potassium, or other factors. Vegetative cover must be irrigated during dry periods until vegetation is fully established.

Guidelines for selecting and placing vegetation are presented in the Alabama Handbook.

Chemical Binders and Tacks

Chemical binders and tacks are sprayed on bare soils or mulches to bind soil particles or mulch material, reduce moisture loss, and enhance plant growth. A chemical binder or tack is a temporary erosion control measure and may be applied with seed, lime, and fertilizers. Chemical binders and tacks provide a viable alternative to seeding if construction occurs at a time when seeding is not feasible.

Erosion Control Blankets

Erosion Control Blankets (ECBs) may be used in place of mulch in areas where vegetation is likely to grow too slowly to provide adequate protection, such as areas subject to concentrated flow and/or steep grades. ECBs protect exposed soils from erosion, provide moisture retention, and may assist in establishing vegetation. ECB installations may be temporary or permanent.

Guidelines for selecting and installing ECBs are presented in the Alabama Handbook.

Riprap

Dumped riprap may also be used for permanent erosion control on slopes with a ratio of 2:1 (horizontal to vertical) or less. Dumped riprap consists of stone or broken concrete dumped in place on a geotextile underlayment or prepared slope to form a well-graded mass with a minimum of voids.

Riprap is not recommended for use along stream banks, unless a riparian buffer cannot be established.

10.3.4 RUNOFF CONTROL

Runoff control measures will be required whenever natural and/or existing land conditions result in an increase and/or change in runoff. This will include when land is cleared and/or vegetation removed to install utility systems, such as storm and sanitary sewers, gas, water, electric, and telephone. Runoff control measures include diversions, slope and down drains, level spreaders, check dams, and outlet protection.

Diversions

Any structure that slows or diverts runoff away from exposed areas can reduce erosion. Types of diversion structures include dikes, swales, and channels, any of which can function as temporary or permanent facilities.

A diversion dike is a ridge of compacted soil placed above, below, or around a disturbed area to intercept runoff and divert it to a stable area. Generally, a diversion dike is the least durable diversion structure and is best used to provide protection for short periods and relatively small amounts of runoff. A dike is often appropriate above a newly constructed cut and fills lose to prevent excessive erosion until more permanent control features are established. Where the ground slope is not steep, a dike is also appropriate above graded slope to divert sediment-laden runoff into sediment traps or basins. Once the slope is stabilized, the diversion dike should be removed.

A diversion swale is an excavated, temporary drainageway used above and below disturbed areas to intercept runoff and divert it to a safe disposal area. A diversion swale can be constructed at the perimeter of a disturbed area to transport sediment-laden water to a sediment trap or sediment basin. The swale should be left in place until the disturbed area is permanently stabilized. A diversion swale can be constructed in conjunction with a dike to prevent stormwater from entering a disturbed area. Although generally a temporary feature, a carefully planned swale could also become part of the permanent drainage system.

A diversion channel is a permanent or temporary drainageway constructed by excavating a ditch along a hillside and often building a dike with the excavated soil along the downhill of a ditch and a dike. Although diversion channels can be used in place of temporary structures like dikes and swales, their primary application is to provide permanent runoff control on long slopes subject to heavy flow concentrations. In addition to usage on or above long spoil slopes, diversion channels are practical for intercepting runoff upgradient of a roadway to prevent off-site flow from entering gutter and inlet facilities.

Diversions must be designed and installed according to the specifications and procedures outlined in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Special Drawings.

Slope and Down Drains

Down drain structures (pipes) and slope drains (paved or sodded channels) can be used as temporary or permanent structures to conduct concentrated runoff safely down a slope. Such structures are often used to help dispose of water collected by diversion structures.

A paved slope drain is a channel placed to extend from the top to the bottom of a slope and lined with bituminous concrete, portland cement concrete, or comparable nonerodable material (such as grouted riprap). This structure is generally used to convey a concentrated flow of surface runoff down a slope without causing erosion. When flow is supercritical, a pipe downdrain should be used to prevent flow from leaving the channel.

A down drain generally consists of either corrugated metal pipe or flexible tubing, together with a prefabricated entrance section, and is temporarily placed to extend from the top to the bottom of a slope.

Temporary slope drains must be designed and installed according to the specifications and procedures outlined in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Special Drawings.

Level Spreaders

A level spreader is generally a permanent outlet constructed at zero percent grade across the slope to convert a concentrated flow of sediment-free runoff (e.g., from diversion outlets) into sheet flow and to discharge it at nonerosive velocities onto undisturbed area stabilized by existing vegetation. The level spreader should be used only where the following conditions are found:

1. The spreader can be constructed on undisturbed soil.
2. The area directly below the level lip is stabilized by existing vegetation.
3. The drainage area above the spreader is stabilized by existing vegetation.
4. The materials used are rigid and nonerodable, such as concrete or asphalt, and a fixed grade can be maintained.
5. The water will not be re-concentrated immediately below the point of discharge.

Check Dams

A check dam is a barrier constructed across an area of concentrated flow for the purpose of reducing velocities and therefore, channel erosion. Check dams are typically installed in series and may be constructed from rock, silt fencing, sand bags, or straw wattles. Check dams are not intended to function as sediment traps, as sediment retention is generally minimal.

Check dams must be designed and installed according to the specifications and procedures outlined in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Special Drawings. Installation considerations include the following:

1. For installation on exposed soil, geotextile underlayment should be used to prevent erosion below the check dam and to prevent undermining.
2. Sand bags should be used in paved ditches.
3. The center of the dam must be lower than the sides.
4. The check dam must extend the full width of the ditch.

Outlet Protection

Outlet protection entails providing de-energizing devices and erosion-resistant channel sections between drainage outlets and stable downstream channels. The channel sections may be rock-lined, vegetated, paved with concrete, or otherwise erosion-resistant. The purpose of outlet protection is to convert pipe flow to channel flow and reduce the velocity of the water consistent with the channel lining. The flow of water can then be conveyed to a stable existing downstream channel without causing erosion.

This practice is applicable to storm sewer outlets, road culverts, and paved channel outlets discharging into natural or constructed channels that, in turn, discharge into existing streams or drainage systems. The appropriate treatment should be provided along the entire length of the flow path from the end of the conduit, channel, or structure to the point of entry into an existing stream or publicly maintained drainage system.

Outlet protection must be designed and installed according to the specifications and procedures outlined in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Special Drawings.

10.3.5 EXPOSURE SCHEDULING

The season and length of time cleared land is exposed to the erosive forces of rainfall can have a significant effect on gross soil loss. In general, the goal is to select the season and duration of exposure so vegetation can be established before heavy rainfall. A quantitative evaluation of exposure scheduling on the gross soil loss form a site can be performed using the monthly distribution of the rainfall factor, R, which can be obtained from data presented in Wischmeier and Smith (1978). This evaluation involves determining the appropriate surface stabilization factors for the sequence of land covers being considered.

10.4 SEDIMENT CONTROL

Sediment control measures will be required to trap eroded material form exposed areas and to prevent off-site damage. At a minimum, the ESC Plan shall include appropriate measures from the following categories:

1. Sediment barriers (e.g., silt fence, wattles, sand bags, etc.)
2. Construction entrance/exit stabilization
3. Sediment traps (e.g., inlet protection, rock filter dams)
4. Sediment basins

10.4.1 SEDIMENT BARRIERS

Sediment barriers are temporary construction measures placed to intercept sheet flows and retain sediment from disturbed areas of limited extent. The barriers intercept stormwater flows to form ponds that allow sediment to settle out of the runoff. Sediment barriers are typically placed on the contour and should not be used in areas of concentrated flow.

Various types of sediment barriers are identified in the Alabama Handbook and the ALDOT Special Drawings. Additional sediment barrier designs may be accepted, provided they are designed by a Qualified Credentialed Professional as defined in the Alabama CGP.

Common sediment barriers used in the Huntsville area are identified below.

Silt Fence

Silt fencing must be installed according to the specifications and procedures outlined in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Special Drawings.

Installation considerations include the following:

1. Type C silt fence should only be used for short-term installation, typically less than 3 months.
2. Silt fence must be trenched in at the bottom to prevent undermining.
3. Silt fence sections must be either overlapped to the next post or rolled.
4. Silt fence is intended for use where the area below the barrier is undisturbed and vegetated.
5. Silt fences are not suitable for areas of concentrated flow.

Straw Wattles

Straw wattles may be used as sediment barriers along the tops of curbs or on the contours of slopes to shorten slope length.

Installation considerations include the following:

1. Wattles must be securely staked to prevent floating or displacement.
2. Wattles shall not be used on pavement, base, or any other surface where they cannot be properly staked.
3. Wattle ends must be overlapped to ensure no gaps are present.
4. Wattles are not suitable for areas of concentrated flow.

Sand/Gravel Bag Barriers

Sand bags or gravel bags may be placed and/or stacked to form a sediment barrier on pavement, base, or in other areas where silt fence or wattles cannot be staked in place.

Installation considerations include the following:

1. Sand or gravel bag sediment barriers should be installed on the contour.
2. Sand or gravel bags must be tightly abutted to prevent flow between or under the bags.
3. flow between or under the bags.

10.4.2 CONSTRUCTION EXIT PAD (TEMPORARY)

All points of access to a construction site shall be stabilized to reduce or eliminate the tracking or flowing of sediment onto public rights-of-way. A stabilized pad of crushed stone is to be placed at entrances and exits of construction sites for this purpose. The construction exit pads must be installed according to the specifications and procedures outlined in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Special Drawings.

Installation considerations include the following:

1. The construction exit pad shall be underlain by non-woven geotextile.
2. The stone for the construction entrance/exit pad shall meet the specifications for ASTM D448 Coarse Aggregate Gradation No. 1.
3. In no case shall any construction exit pad be less than 6 inches in depth.

4. Construction exit pad width shall not be less than 15 feet and shall be wide enough so that the largest construction vehicle will fit with room to spare.

Construction exit pads must be properly maintained to be effective. Maintenance activities shall include periodic top dressing with additional stone as conditions demand or cleanout of any facilities used to trap sediment.

In some cases, wheels of construction vehicles should be cleaned prior to leaving the construction site. When appropriate, a stabilized area that drains into a sediment trap or basin shall be provided.

10.4.3 SEDIMENT TRAPS

A sediment trap is a temporary basin formed by an excavation and/or an embankment to intercept sediment-laden runoff and trap and retain the sediment. In so doing, drainageways, properties, and right-of-way below the trap are protected from sedimentation. A trap can also be formed at storm sewer inlets by placing a raised barrier around the inlet.

Inlet Protection

Inlet protection is a temporary barrier installed around an inlet to slow runoff to encourage settlement of suspended particles prior to entering the inlet. Inlet protection is most effective for coarse particles and typically does not reduce turbidity.

Inlet protection may be a manufactured product or may be field-constructed using silt fencing, block and gravel, sand bags, or straw wattles. Inlet protection must be installed according to the specifications outlined in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Special Drawings.

Rock Filter Dam

A rock filter dam is a temporary stone embankment constructed across a drainageway to capture sediment. This practice may also be used as a forebay to a sediment basin. Rock filter dams are intended for use in flowing water.

Rock filter dams must be designed and installed according to the specifications outlined in the most recent version of the Alabama Handbook.

10.4.4 SEDIMENT BASINS

A sediment basin is usually a temporary facility constructed along a flow path to capture sediment carried by runoff from a cleared construction site. The basin is formed by placing and earthen dam across the watercourse, by excavating a depression, or by a combination of the two. Sediment basins are intended to capture sediment from areas that are not treated adequately by other sediment control measures. Basins act as a last line of defense against off-site sediment damage and, if maintained by periodic cleaning, can sometimes be re-used as permanent facilities.

Sediment basins shall be designed by a QCP and installed according to the specifications and procedures outlined in the most recent version of the Alabama Handbook or the most recent version of the ALDOT Special Drawings. Sediment basins shall not be constructed under the following conditions:

1. Failure would result in loss of life, damage to buildings, or interruption of service from public roads or utilities.

2. The tributary drainage area is more than 10 acres.

3. The basin will become a maintenance problem for the City of Huntsville.

Installation considerations include the following:

1. Basin location should be selected so that failure of the dam will not result in property damage or loss of life.
2. The basin should be constructed with a minimum length to width ratio of 2:1.
3. The basin floor should generally be level, although a reverse grade may be included between the inlet and the first baffle to improve sediment trapping.
4. Porous baffles must be installed across the basin, perpendicular to the flow path between the inlet and outlet. Baffles are intended to spread the flow across the full width of the basin and to reduce turbulence.
5. The embankment, spillway, basin bottom, and basin sides must be stabilized using either vegetation or a non-woven geotextile.
6. Sediment basins should be dewatered using a skimmer or other dewatering device that draws water from the water surface.

Sediment basins require scheduled inspection and maintenance to function properly and shall be located for easy access. In addition to routine maintenance, sediment basins shall be inspected after all major storm events (0.5 inch or greater within 24 hours) to ensure that spillway structures are not clogged and that the sediment storage volume has not been exceeded.

A sediment basin shall be cleaned out when it has reached 50 percent of its sediment storage capacity. A red pole shall be placed in the sediment storage area to establish the depth at 50 percent of the storage volume. When the red pole disappears, the sediment shall be removed. The method of sediment disposal shall be clearly identified on the ESC Plan prior to construction.

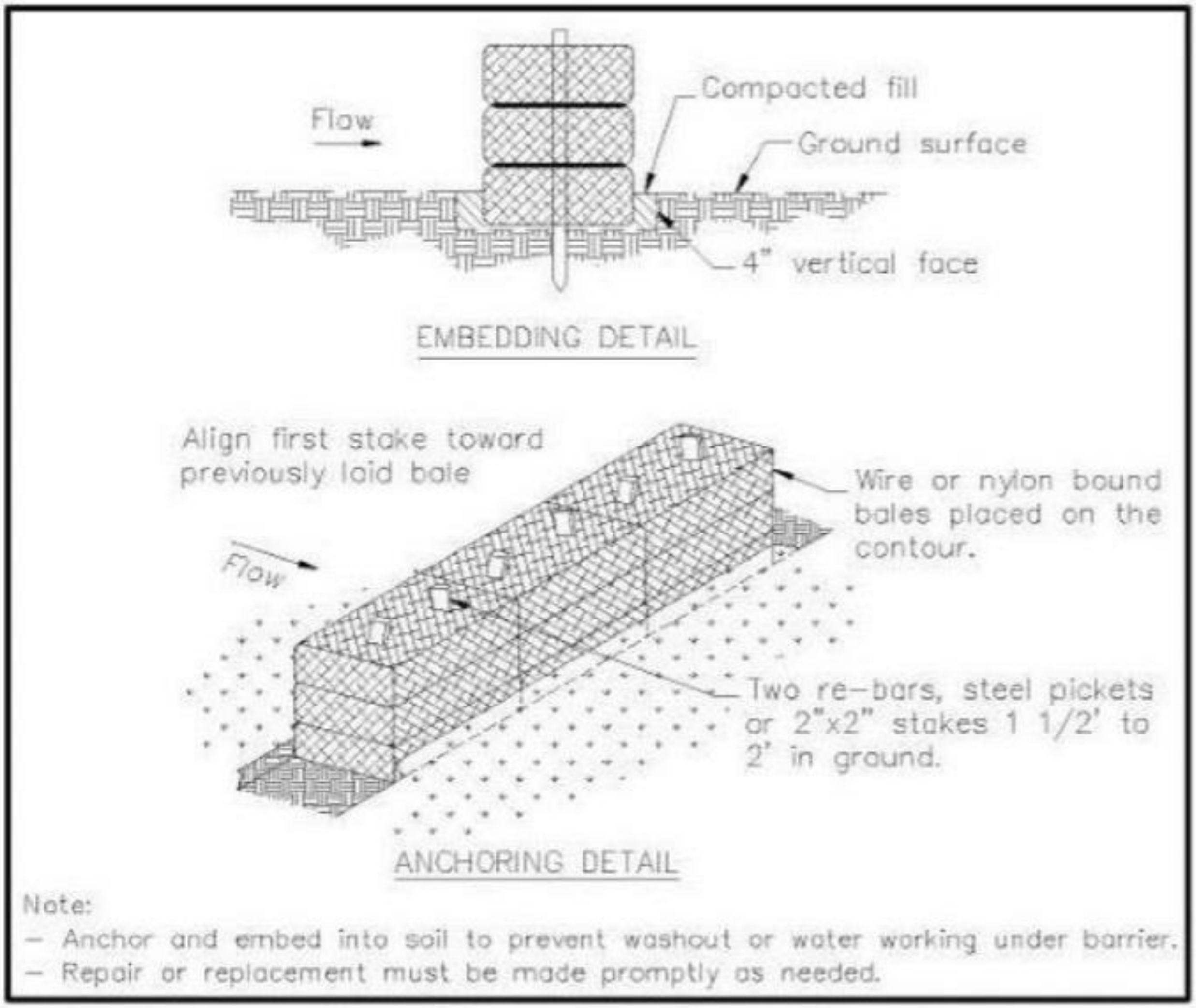
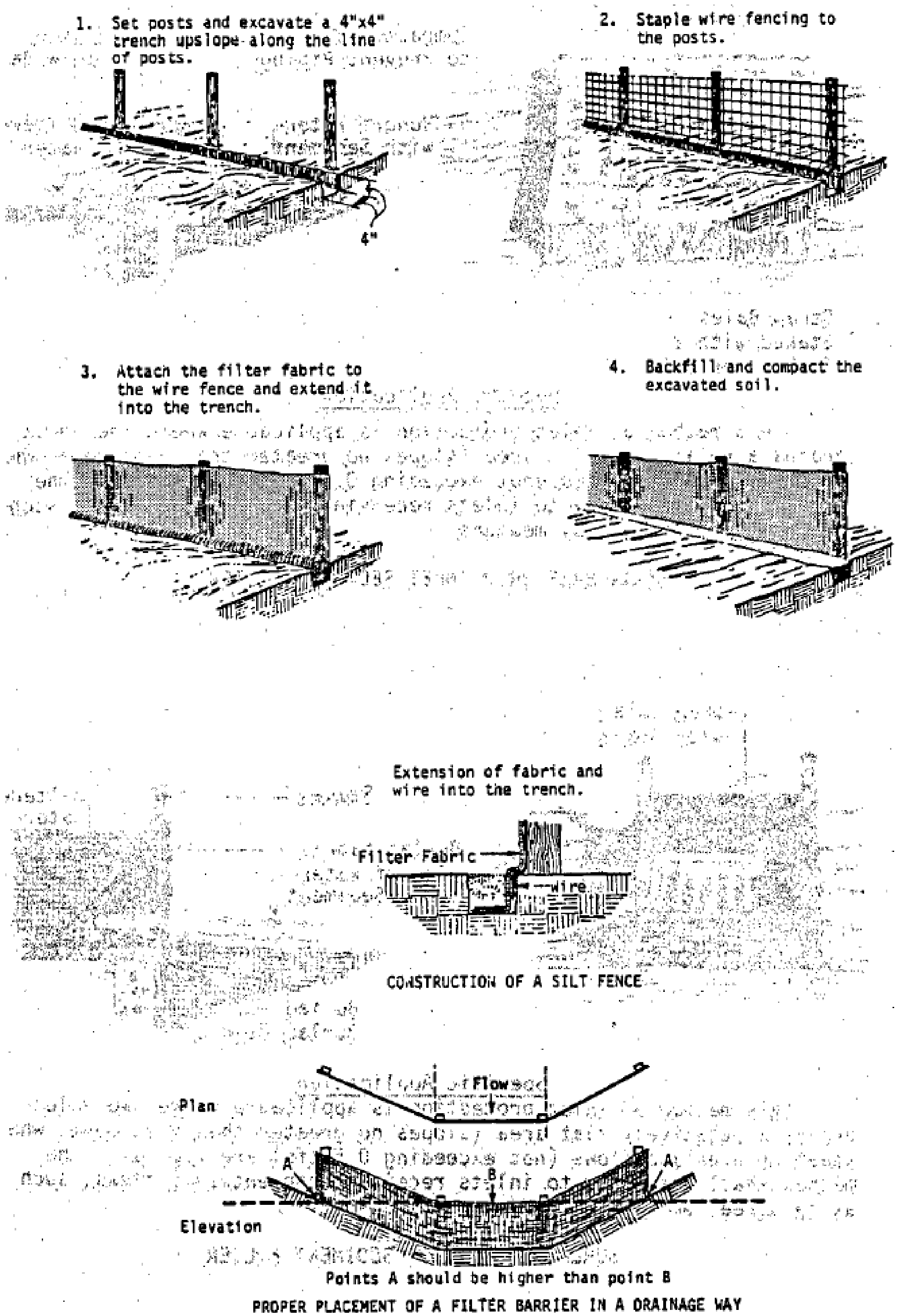
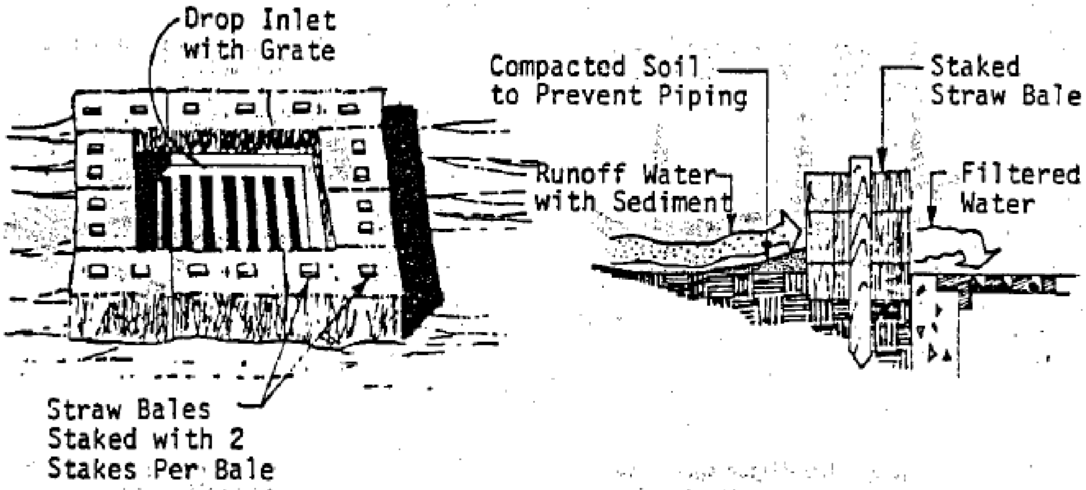


FIGURE 10-1
DETAILS FOR PLACING STRAW BALE BARRIER



Reference: Virginia Soil and Water Conservation Commission (1980).

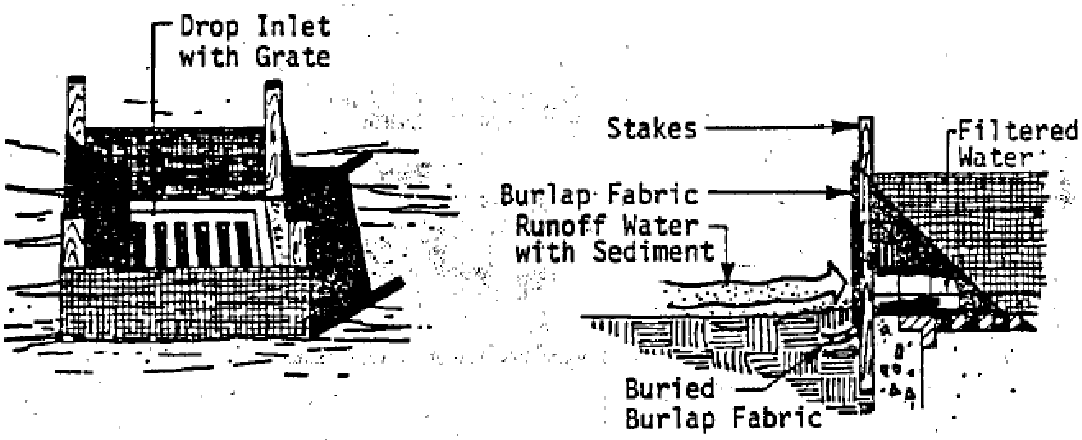
FIGURE 10-2
Details for Placing Silt Fences



Specific Application

This method of inlet protection is applicable where the inlet drains a relatively flat area (slopes no greater than 5 percent) where sheet or overland flows (not exceeding 0.5 cfs) are typical. The method shall not apply to inlets receiving concentrated flows, such as in street or highway medians.

STRAW BALE DROP INLET SEDIMENT FILTER



Specific Application

This method of inlet protection is applicable where the inlet drains a relatively flat area (slopes no greater than 5 percent) where sheet or overland flows (not exceeding 0.5 cfs) are typical. The method shall not apply to inlets receiving concentrated flows, such as in street or highway medians.

BURLAP DROP INLET SEDIMENT FILTER

Reference: Virginia Soil and Water Conservation Commission (1980).

FIGURE 10-3
Details for Placing Temporary Inlet Sediment Traps

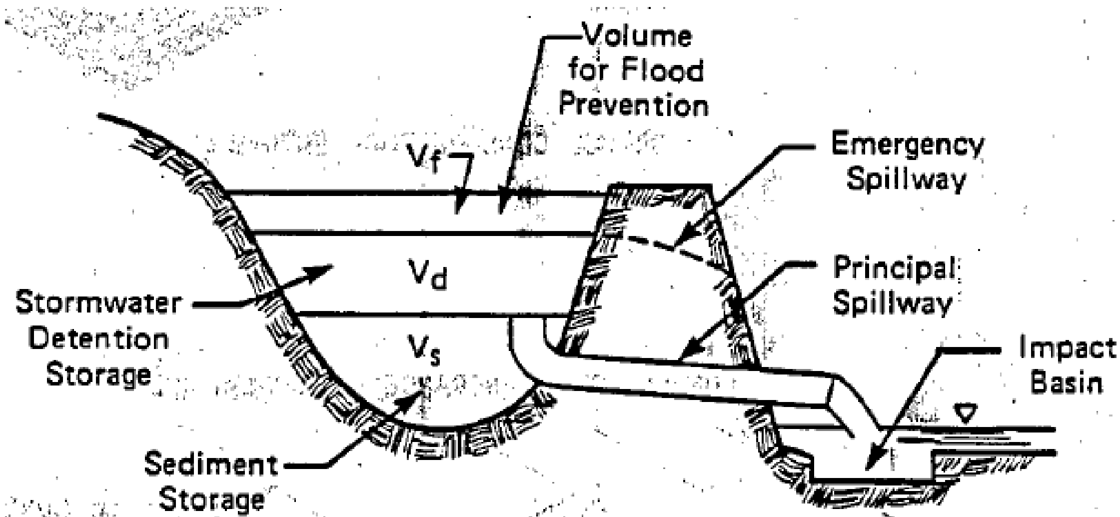
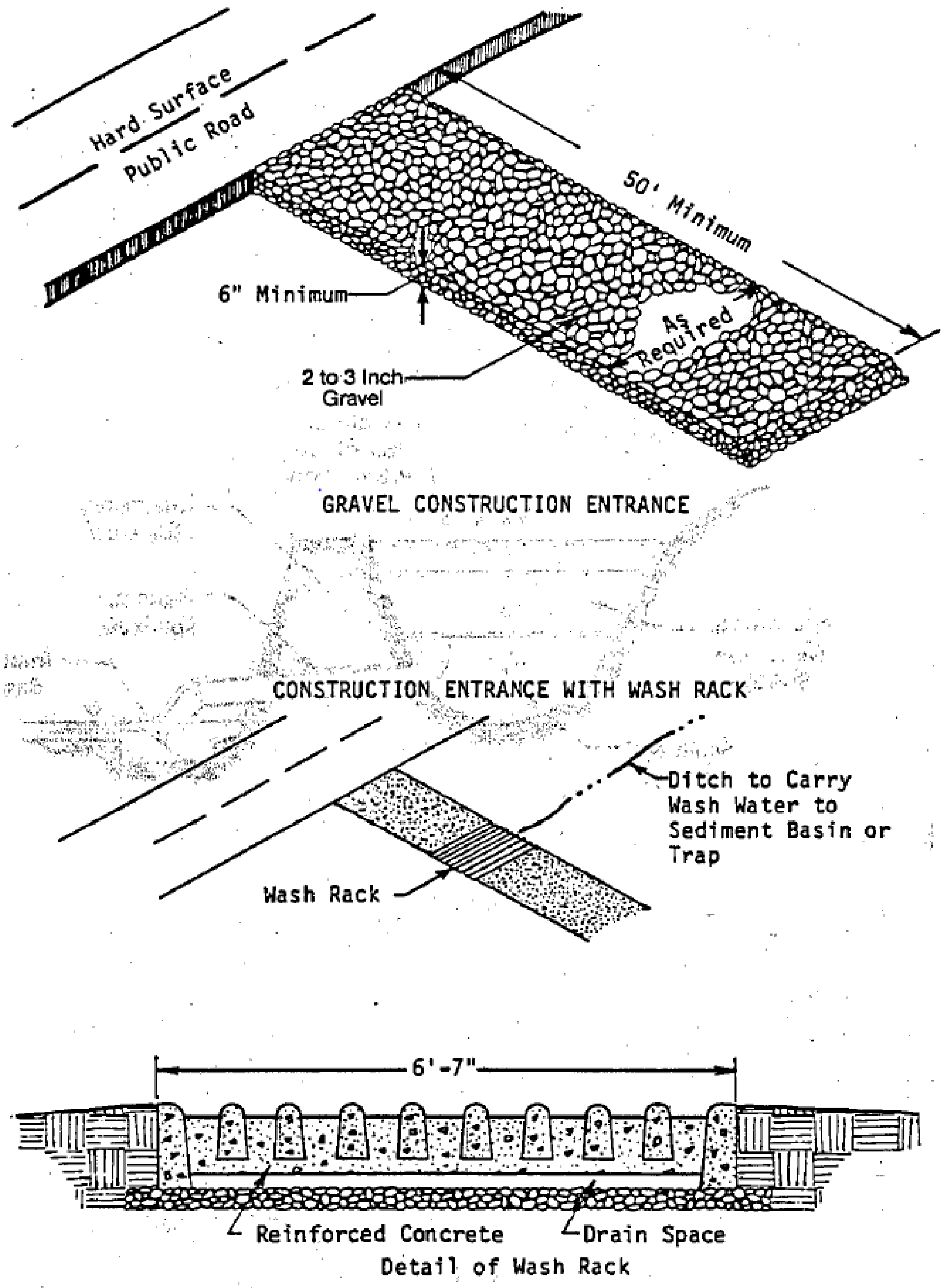


FIGURE 10-4
Conceptual Sketch of Components for Typical Sediment Basins



Reference: Virginia Soil and Water Conservation Commission (1980).

FIGURE 10-5
Details for Construction Entrance Treatment

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